Manitoba
INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

KGS GROUP
CONSULTING ENGINEERS

Final Report

12-0300-011 | January 2016

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January 12, 2016

File No. 12-0300-011

Manitoba Infrastructure and Transportation
2nd Floor, 280 Broadway
Winnipeg, Manitoba
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ATTENTION: Mr. Eugene Kozera
Director, Water Management, Planning & Standards

RE: Assiniboine River & Lake Manitoba Basins Flood Mitigation Study

Dear Mr. Kozera:

Please find enclosed 30 copies of the Final Report for the Assiniboine River & Lake Manitoba Basins Flood Mitigation Study, as well as one electronic copy on a DVD.

We appreciate the opportunity to have worked with MIT on this project and look forward to any future projects. If you have any questions or comments regarding the enclosed report, please contact the undersigned.

Yours truly,

<original signed by>

Dave MacMillan, P.Eng.
Principal

DBM/nf
Enclosures

cc: C. Siepman, P. Leclercq, R. Carson, KGS Group

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FINAL REPORT

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EXECUTIVE SUMMARY

1. BACKGROUND

The flood of 2011 was the largest flood in more than one hundred years in both the Assiniboine and Souris Rivers. Similarly, extreme water levels were experienced on Lakes Manitoba and St. Martin. The flood of 2011 highlighted several potential weak links in the existing flood control systems. The shortcomings of the system were emphasized again in 2014.

Studies by the 2011 Flood Review Task Force and the Lake Manitoba and Lake St. Martin Regulation Review have both recently provided broad perspectives on this unprecedented flood event and offered a variety of opinions and recommendations on possible improvements that could reduce damages in the future. The Province of Manitoba retained KGS Group to undertake engineering analyses of a wide range of measures that could mitigate floods of a similar magnitude to 2011 and larger. The study was requested to be at a conceptual level of analysis, and intended to identify and assess potential options to reduce flood risk for communities and major infrastructure along the Assiniboine River, Souris River, Qu'Appelle River, Dauphin Lake, Lake Manitoba, Fairford River, Lake St. Martin, Dauphin River and Shoal Lakes. This report provides a detailed account of the work carried out, the findings, and the conclusions / recommendations.

The study process involved the deployment of the usual Steering and Technical Committees comprised of staff from MIT, Manitoba Conservation and Water Stewardship (C&WS), University of Manitoba, and Manitoba Agriculture, Food and Rural Development (MAFRD). It also included input from a Study Liaison Committee (SLC) that had a broader perspective than the engineering focus of the Steering and Technical Committees. In addition to the representation previously identified, the SLC also included representatives from various key towns, cities and municipalities (e.g., Winnipeg and Brandon), as well as from other organisations and local citizen groups (e.g., Emergency Measures Organization (EMO), Manitoba Métis Federation (MMF), Ducks Unlimited, Association of Lake Manitoba Stakeholders (ALMS), and more).

The study commenced with review and familiarization of a large compendium of previous reports. Over 185 reports were examined, dating back as far as the 1950s. A summary of all the reviewed reports is provided in this report. **Since the 1950 Flood, Manitoba has developed an extensive flood protection system. This integrated strategy has worked well and has avoided billions of dollars in damages.**

The study terms of reference required identification of all major flood vulnerabilities throughout the study area. The process to confirm vulnerabilities that were initially identified in this study was based on a technical workshop developed jointly by the Manitoba Infrastructure & Transportation (MIT) Steering Committee and KGS Group. Approximately 75 technical representatives with diverse experience in Manitoba floods were invited from various government and non-government sectors. The purpose of the workshop was to provide a mechanism for input from the attendees prior to KGS Group completing their comprehensive conceptual-level studies.

2. VULNERABILITIES

A wide range of vulnerabilities were identified and are described in detail in this report. In the context of the process of identification, the threshold of minimum acceptable protection against floods was defined by the three criteria promoted by the Province of Manitoba, which were:

1. The 1:200 year event, as recommended by the Lake Manitoba and Lake St. Martin Regulation Review Committee and the 2011 Flood Review Task Force, which was greater than the previous 1:100 year standard as discussed further below.
2. The highest flood on record if greater than the 1:200 year event.
3. A greater flood event that could be justified in an economic analysis that considered costs and benefits.

More than 100 vulnerabilities were identified and classified by a rating system that varied from low to high. The three most important vulnerabilities are summarized as follows:

1. The maximum safe discharge capacity of the Lower Assiniboine River from Portage la Prairie to Winnipeg combined with the Portage Diversion is less than the minimum criteria for flood protection promoted by the Province. This vulnerability became evident during the 2011 Flood when extensive emergency measures were necessary on the Portage Diversion and on the Lower Assiniboine River to convey peak flows.

2. The shorelines of Lakes Pineimuta and St. Martin are vulnerable to flooding. This has been a chronic problem for decades and requires urgent attention.

3. Although the response to the 2011 Flood effectively addressed major problems, and decisions were made in a timely and purposeful fashion, many decisions were made under duress and were based on less than full knowledge of optimum reactions. Existing emergency preparedness and response plans to address extreme floods have been pushed to the limits and would greatly benefit from being updated and expanded.

The flood mitigation alternatives analysed in the study addressed a significant number of the vulnerabilities that were identified including the first two most important vulnerabilities mentioned above. The remaining vulnerabilities, including the third most important vulnerability identified above, were not specifically addressed in this study as they were outside the study scope. Nevertheless, these were also provided to the Province of Manitoba with the report.

3. OPEN HOUSES

Three rounds of public open house events were held for the study. The first round occurred from June 18 to 20, 2013 in Dauphin, Brandon and Portage la Prairie and communicated as much information as possible regarding the scope of the study. Another open house event was held in Ashern on September 18, 2014 and provided information specific to the Lake Manitoba and Lake St. Martin outlet channels. The final round occurred from December 8 to 10, 2014 in Dauphin, Brandon and Portage la Prairie and provided an opportunity for feedback on the preliminary results of the study prior to the development of the final report. Feedback received at the open house has been incorporated into the report.

4. BASIN HYDROLOGY

The hydrology of the Assiniboine River and Lake Manitoba drainage basins has been studied extensively as a basis for addressing possible improvements in the flood protection works. Flood frequency analyses were provided by MIT for most relevant rivers and streams in the region, and were supplemented with other analyses as required for specific study needs. The statistics of all rivers were focussed on the characteristics derived from the full hydrometric record for each site, which for example, dates back more than 100 years for the Assiniboine River at Holland. Flood probabilities were identified across the region as a basis for assessing the need and justification for upgrades in flood protection. Use of the full historical period in the statistical analyses reflects an assumption that the hydrological characteristics of the watersheds will remain essentially the same in the future as they have persisted in the past. For example, 1:200 year flood peaks were estimated to be the following for key locations in the watershed, based on the existing published records of river flow:

- Assiniboine River near Holland – 1,617 m^3/s (57,100 cfs).
- Assiniboine River at Brandon – 1,198 m^3/s (42,300 cfs).
- Souris River at Melita – 790 m^3/s (27,900 cfs).
- Lake Manitoba still water level – El 248.9 m (El 816.6 ft).

It was recognized in this study that there is a growing belief that climate change is already affecting the statistical characteristics of flood potential in the region, compounded in some areas by changes in land use. However, the consensus of the engineering community represented at the Workshop held on February 6, 2013, was that there is no solid understanding of the means to predict future trends at this point in time. It was agreed at the Workshop and subsequently confirmed with MIT that the full historical period of hydrometric records should be used as a basis of analysis. However, it was also agreed that sensitivity analyses should be conducted to explore what the effects would be if shorter records using only recent high flood peaks were considered. This is discussed further later in this summary, and details are provided in the report.

The drainage basin of the study area is vast, covering more than 246,000 km^2 (95,000 mi^2). The Assiniboine River is also a tributary of the Red River and forms more than 82,000 km^2 (32,000 mi^2) of the total area of 285,000 km^2 (11,000 mi^2) at Winnipeg. It was recognized that floods in one region do not necessarily correspond to simultaneous floods in another region. For example, floods in the Assiniboine River may or may not occur in parallel with inflow floods to Lake Manitoba. Similarly, floods in the Assiniboine River may or may not occur simultaneously with floods in the Red River. This diversity within the overall watershed had to be understood and reflected appropriately in the statistical characteristics of flood events. Special statistical techniques were devised to address the diversity of the streams and were used to quantify flood probabilities with and without upgrades to flood protection works, as well as to form a basis for economic calculations of benefits due to flood reduction. Details of this approach have been provided within the report, but the key basis for the studies of diversity across the watershed was the application of the principles of synthetically generated flood peaks and volumes for a large number of hypothetical years in the future. The statistical generation of this data followed in all cases the existing known statistical characteristics of individual rivers and streams within the study area. This technique, often called a Monte Carlo approach by practitioners, permits examination of the inter-dependencies of flood flows within the region. It is widely accepted by practitioners and has formed the basis for studies of the basin-wide flood potential, particularly for the Assiniboine / Lake Manitoba / Red River catchments.

The assembled statistical characteristics of floods throughout the study area permitted the identification of zones where the flood potential exceeded the existing capability of flood protection works and, therefore, formed the basis of the vulnerability to floods. The only zone where flood protection does not meet the basic standard of either the flood of record or the 1:200 year flood was found to be the Lower Assiniboine River between Portage la Prairie and Headingley. **The level of protection for the Lower Assiniboine River area, including the Portage Diversion, is deficient with an estimated level of protection of 1:55 years**.

In other areas of the basin, there have already been significant strides towards upgrading the flood protection system. **A number of related flood mitigation studies and projects are ongoing or have been recently completed within the Assiniboine River and Lake Manitoba Basins. Many areas are, therefore, already protected to the highest flood on record.**

5. FLOOD PROTECTION MEASURES AND ECONOMIC ANALYSES

A wide variety of flood protection measures were considered as potential upgrades to existing facilities, as well as new works that could assist in averting flood damages. **A massive amount of data and technical input has been assessed and over 70 mitigation options have been evaluated**. The options included all the classic measures such as dikes, reservoirs, diversion channels, channel improvements, modifications to land use, as well as the purchase of vulnerable properties where protection measures were not practical. The report provides considerable detail on each option considered, and the improvements in flood protection that could be provided by the options, both as stand-alone schemes, or in some cases selected combinations.

The economics of the various options were assessed using guidelines that were developed for the study and vetted by representatives of MIT and the Study Liaison Committee. The details of the guidelines are extensive and are described in the report. Key features included:

- Benefits were estimated on the basis of the averted flood damages in the future. These damages would include:
  - Cost of pre-emptive works or actions that would be deployed if the protection measure were not implemented.
  - Repairs / replacements for direct physical damages to residential, commercial and industrial buildings and contents, as well as damages to adjacent developments.
  - Temporary relocation and subsistence costs for affected residents.
  - Temporary services that may be required until direct physical damages are repaired (e.g., hauled water or septic).
  - Repair / replacement of damaged vehicles that could not be removed from the inundated zone in time.
  - Agricultural losses.
  - Miscellaneous damage to crops or arable land.
  - Miscellaneous impacts on farm infrastructure including, for example:
    - Effects on timely access to non-flooded fields.
    - Loss of income from off-farm jobs.
    - Relocation costs of the affected farm.
    - Emergency care of livestock, if located within the ring dike.
    - Repair / replacement of damaged public infrastructure.

- A depth-damage relationship was used to estimate damages on partially submerged buildings, and was modelled after similar relationships devised in previous extensive studies of flood protection in the City of Winnipeg.
- Expected Annual Damages (EAD) were estimated with and without a protection option and the calculation incorporated the full spectrum of flood probabilities up to and exceeding the 1:500 year flood.
- A time horizon of 50 years was the basis of the economic calculations, and present valuations were based on this period.
- Costs of the various options were based on preliminary estimates of the construction details, using a set of pre-selected unit prices that would represent conditions in the study area in 2014 (i.e., all cost estimates are in 2014$).
- Rates of interest and inflation assumed the following basic values:
  - Interest – 7% per year.
  - Inflation – 3 % per year.
  - Future Growth of Population and Economic Activities in the region – 1% per year.
- Sensitivity analyses were undertaken in cases where the economic indicators would be affected significantly by the assumed parameters listed above.

6. NUMERICAL MODELS

A broad spectrum of numerical models were developed and applied during the course of the study. These included:

- Flood routing models for a range of key lakes and river reaches.
- Backwater models were either developed from cross sectional data or based on existing models provided by MIT.
- Wind and wave generation models for Lake Manitoba.
- Models to permit synthetic generation of river flows in the Upper and Lower Assiniboine River, inflow to Lake Manitoba, and flows in the Red River.
- Flood damage models that incorporated depth-damage relationships / topographic information and water levels with and without flood protection works under consideration.

All major models were documented and prepared for transfer to MIT as “legacy tools” for use in future studies of the region.

7. FLOOD PROTECTION MEASURES

A summary of the measures and issues that were considered for each of the geographic areas of the region follows.

A. UPPER ASSINIBOINE RIVER

Agricultural land located along the Upper Assiniboine River at the bottom of the valley is prone to flooding and at times has been negatively impacted by operation of the Shellmouth Dam. The 2011 Flood Review Task Force recommended that three alternative means to prevent or reduce flood damages on the Assiniboine River downstream of Shellmouth Dam should be investigated. These options were evaluated for the study and are summarized below:

1. The effects of the installation of gates on the spillway at the Shellmouth Dam, and specifically whether they could be relied on to reduce flood potential was reviewed in

considerable detail. A previous numerical model developed by the Province of Manitoba (Assiniboine River Water Management Model) was mobilized and used to assess the merits of gates in the context of flood mitigation. **It has been concluded that the gates on the Shellmouth Dam could have moderating effects at modest flood magnitudes, however, they would essentially provide no significant improvement for above normal to extreme floods** without changes to the existing operating guidelines for the structure. Even the modest benefits for minor floods would only be available locally near the Shellmouth Dam and would attenuate significantly further downstream.

2. Options for protecting agricultural interests along the Assiniboine River Valley between the Shellmouth Dam and Brandon were considered. Diking proved to be very expensive, ranging between approximately $73 million to $471 million depending on the options, which provided a flood protection level of 1:5 year to 1:50 year, and was not practical in comparison with the estimated benefits that could be gained. It was also recognized, but not evaluated specifically, that diking along the valley would exacerbate flooding in Brandon and downstream due to an increase in flow from the loss of floodplain storage in the protected area.
3. An alternative option in the Upper Assiniboine River is the purchase of flood prone land. The option consisted of purchasing land affected by flooding between the Shellmouth Dam and St. Lazare at a flow of 85 m^3/s (3,000 cfs). The cost of the option was estimated to equal approximately $24 million. **Due to various issues related to the linear diking scheme identified in the report and the lower cost of the land purchase option, it was concluded that purchase of the flood prone land option was preferred over linear diking and that it should be considered further.**

A selection of possible large reservoirs including the Holland Dam and the Alexander Dam on the Assiniboine River, the Zelena Dam on the Shell River, and the Victor Dam on the Qu'Appelle River were studied and found to provide varying extents of reduction in flood conditions, but with considerable cost, ranging between $104 million for the Zelena Dam Project, to $591 million for the Alexander Dam project. The environmental effects on the surrounding area would be substantial, ranging from loss of agricultural land, loss of First Nation reserve land, to the blocking of fish passage, altering of fish habitat associated with the change in flow regime and impeding navigation of the rivers. These impacts are expected to render these options difficult to implement. **Therefore, it has been concluded that large dams on the Upper Assiniboine River should not be considered further due to the significant environmental and operational concerns, as well their relatively high costs compared to other flood mitigation solutions**. These conclusions are consistent with the comments and advice received at the technical workshop that was undertaken at the beginning of this study.

The City of Brandon is currently in the process of upgrading their flood protection system to be able to withstand a repetition of the 2011 and 2014 floods. **It has been concluded that linear diking is the most feasible means to increase the level of protection for Brandon**. Current hydrological analyses that include both the 2011 and 2014 floods suggest that the 2011 Flood is approximately a 1:140 year flood event. The study assessed requirements to increase that protection to a 1:300 year flood event. The proposed diking now underway would have to be raised by 0.8 m (2.6 ft), at an estimated additional cost of $11 million.

The communities of St. Lazare and Sioux Valley are protected to the highest flood on record, which exceeds the provincial flood protection level target of a 1:200 year flood event.

B. LOWER ASSINIBOINE RIVER

The existing provincial linear diking system on the Assiniboine River downstream of Portage la Prairie was able to convey a flow of about 510 m^3/s (18,000 cfs) for two months in 2011, and again for approximately two weeks in July 2014. Although the dikes held in 2011, significant repairs and ongoing inspection and maintenance were required, with temporary top up, to safely pass the flood during the extended flood period. **Based on an assessment of the existing diking system and experience from the 2011 Flood, the current reliable flow capacity of the provincial Assiniboine River linear dikes has been estimated to be 425 m^3/s (15,000 cfs)**.
Conceptual designs and cost estimates were prepared for three improvement schemes for the existing dikes between Portage la Prairie and approximately Baie St, Paul. The schemes consisted of:

- Option 1a – Upgrading the existing dikes generally in their current location.
- Option 1b – A major setback of the dikes to adjacent roadways.
- Option 1c – A combination of existing dike upgrades and major setbacks to adjacent roadways.

Different capacities were considered for the options, which when combined with upgrades to the Portage Diversion, would provide a flood protection level of at least 1:200 years. The cost for Option 1a was estimated to range between approximately $153 million and $392 million for design flows of 510 m^3/s (18,000 cfs) to 793 m^3/s (28,000 cfs). **Option (1a) was considered the most feasible option to increase the capacity of the Assiniboine River between Portage la Prairie to approximately Baie St. Paul as the estimated costs for Option 1b and Option 1c were estimated to be two to three times more.**

Downstream of the provincial linear diking system, from approximately Baie St. Paul to the City of Winnipeg limits in Headingley, minimal damages occurred during the 2011 and 2014 floods, although temporary flood mitigation measures were implemented to protect vulnerable properties from flooding. The following two options were considered for permanent protection in this area:

1. Option 2a – Extend the existing dike to the City of Winnipeg limits in Headingley.
2. Option 2b – Combination of individual flood proofing, neighbourhood diking and the purchase of vulnerable properties severely affected by flooding.

The most feasible option to increase the capacity of the Assiniboine River between approximately Baie St. Paul and the City of Winnipeg limits in Headingley is Option 2b, where the cost was estimated to range between approximately $21 million to $123 million for design flows of 510 m^3/s (18,000 cfs) to 793 m^3/s (28,000 cfs). The cost for option 2a is higher and estimated to range between $114 million and $197 million for the same flows.

Diversion of excess flood waters in 2011 at the Hoop and Holler Bend of the Assiniboine River was achieved by excavating a portion of the south bank of the Assiniboine River to release some flow into the La Salle River watershed. Review by the Task Force on the 2011 Flood led to a recommendation that the overflow could be a viable future emergency measure if more permanent works could be developed to contain the overflow and avoid excessive flooding along the diversion route.

The study examined several options for the capability of having a permanent mechanism for release of excess floodwaters at Hoop and Holler. It was found that, to affect a significant release of excess flood flow, significant permanent works including extensive excavation of a channel would be required. The costs of such works were estimated to range upwards of $110 million for a 99 m^3/s (3,500 cfs) capacity, and are considerably more expensive than other options considered later. **A permanent wasteway at Hoop and Holler is not feasible due to economics, increased flood risk downstream, as well as significant opposition from local stakeholders**. However, the option could still be considered as an emergency option for flood relief, but only under dire circumstances; and the effectiveness would be dependent on the situation at the time. Careful planning would be required before execution could be justified.

An alternative solution for mitigating flood damages in the Lower Assiniboine River area consists of protecting the homes and communities individually (similar to what is done within the Red River Basin) rather than increasing the capacity of the Assiniboine River. The cost of this scheme has been estimated to range between approximately $255 and $310 million. Without linear dikes, there would be widespread flooding in the Lower Assiniboine River area, which would spill into the La Salle River system. **Individual and community flood protection in the Lower Assiniboine River area is not a feasible solution due to the potential negative impacts and added costs of damages resulting from widespread flooding**. The transportation network would be severely affected, communities and individuals would need to be evacuated, and agricultural land, which is currently protected by the existing linear diking system, would now be routinely flooded.

C. PORTAGE DIVERSION

The original design of the Portage Diversion (PD channel) included a segment along the downstream end of the west embankment that would allow overtopping and spillage onto the adjacent land to the west if diversion flows exceeded 425 m^3/s (15,000 cfs). This zone was called the “Failsafe”. Since completion of the channel in 1969, the Failsafe has been overtopped on numerous occasions. The damages associated with the overflow are considered unacceptable to local stakeholders.

The study considered options to reduce the damages due to overtopping of the Failsafe:

- Elimination of the Failsafe by raising the East and West dikes in the PD channel along the lower reach. The Failsafe is currently estimated to be overtopped at flows less than 425 m^3/s (15,000 cfs). Costs are estimated to range from $3.6 million to $11 million , for increases of that threshold flow at activation of the Failsafe to 510 m^3/s (18,000 cfs) and to 708 m^3/s (25,000 cfs), respectively.
- The purchase of vulnerable property subject to flooding. Costs are estimated to be relatively high for this option, at about $37 million.
- The constructing of dikes to protect agricultural land from flooding. This was estimated to cost about $12 million.
- Constructing dikes along Cram Creek to control PD overflows through the Failsafe. Two extents of diking were considered, ranging in cost from approximately $7 million to $16 million.

Subsequent sections of this report will reveal that expansion of the Portage Diversion is a leading option for reducing flood potential in the region. Regardless of what form that expansion would involve, it would be logical to incorporate the elimination of the Failsafe within it. Due to

the relatively low cost of eliminating the Failsafe compared to the overall cost of increasing the Portage Diversion capacity, as well as due to the re-occurring economic, environmental and social impacts of flooding that has occurred in recent years from activation of the Failsafe, **it has been concluded that any upgrades to the Portage Diversion should inherently eliminate the Failsafe**.

It is clear that mitigation of floods on the Lower Assiniboine River could be achieved by increasing the amount of flow that can be diverted through the PD into Lake Manitoba. Three concepts were considered in this study:

1. Widening the PD Channel.
2. Construction of an additional channel parallel and to the west of the existing PD.
3. Upgrades to the existing PD, or the "retrofit" option.

In addition to these three alternatives, a "do nothing" option was also considered.

The Portage Diversion Reservoir during flood conditions is a body of water that could cause serious damage if released suddenly through a breach in the retaining structures. The Canadian Dam Association (CDA) has issued widely accepted guidelines for assessing acceptable risks at such structures. It is understood that a Dam Safety Review of the Portage Diversion has not been carried out to ascertain the extent to which this project complies with the CDA Dam Safety Guidelines. Undertaking such a review was not practical within the current study. As a result, it was difficult to speculate on changes to the project that would be acceptable, and changes that could violate the principles of dam safety currently accepted in Canada. The options considered in this study were, as a result, considered conceptual and must be refined after a full dam safety review has been carried out. This review is not anticipated to substantially change the conclusions of the study, however, the results must be considered at the next stage of design.

Widening of the channel was considered for a design discharge increased from 708 m^3/s (25,000 cfs) to 960 m^3/s (34,000 cfs). The conceptual scheme devised included expansion of the inlet control structure and the drop structures, replacement of the outlet structure and river control structure, excavation of approximately 16.7 million cubic metres (21.8 million cubic yards) of earth, with revegetation of disturbed areas, and new bridges. The cost is estimated to be $615 million.

Construction of a parallel channel for a total design discharge of 960 m^3/s (34,000 cfs) would require excavation of approximately 3.5 million cubic metres of earth (46 million cubic yards), revegetation of disturbed areas, a new inlet control structure, along with drop structures and an outlet structure, and new bridges. The cost was estimated to be $265 million. However, as outlined below, the parallel channel option was independent of the existing Portage Diversion and, therefore, would also have to consider the "do nothing" costs.

The "retrofit" option 960 m^3/s (34,000 cfs) would require increased dike heights, dike improvements, modifications and upgrades to the existing control and drop structures, rehabilitation of the river control structure and outlet structure, new bridges and revegetation of disturbed areas. The cost is estimated to be $373 million.

Even if none of the above options were pursued, or if the parallel channel option was selected, there would still be substantial costs required to maintain the existing Portage Diversion in working order for secure future flood protection. This would include refurbishment of the inlet

control structure and drop structures, rehabilitation of the river control structure and outlet structure and future upgrades or replacement of bridges, estimated to amount to a total cost of $136 million

At this level of concept design, the retrofit option was identified as the preferred alternative to upgrade the Portage Diversion capacity, since it had the lowest cost. The cost is estimated to be $237 million more than the "do-nothing" option, but is $28 million less than the combined cost of the parallel channel option and the "do-nothing" option. However, the Retrofit Option requires careful consideration of the dam safety issues and the additional risk associated with this option in the overall decision making process. Additionally, the parallel channel will almost certainly have greater environmental and socio-economic impacts. This is primarily due to the additional land acquisition requirements and expansion of the overall project foot print.

The Province is currently evaluating some of the issues associated with the "retrofit" that have been identified in the report, such as the adequacy of the Portage Diversion dikes that were raised in 2011. Should the Province wish to pursue the retrofit option, this would best be done at a functional design stage where the criteria for the preferred options could be established and a more robust comparison of the alternatives could be undertaken, including consideration of maintenance and operating costs.

D. SOURIS RIVER

There have been many studies of options to control flooding along the Souris River. The most comprehensive report was prepared in the late 1970s for the Souris River Basin Board. The findings from that study were used by KGS Group to guide updates and new studies. The most promising options included reservoirs and diking upgrades at the principal communities along the river at Melita, Souris and Wawanesa.

The reservoirs identified for updated planning studies were the High Souris Dam and the Nesbitt Dam, both located on the Souris River. The High Souris Dam would cost approximately $131 million, and would potentially reduce significant flood peak flows by only 3 to 5% at Portage la Prairie. The Nesbitt Dam would cost approximately $238 million and would potentially reduce significant flood peak flows by only 3 to 6% at Portage la Prairie.

Both the High Souris and Nesbitt Dams would have significant environmental impacts that would include flooding and loss of agricultural land, blocking of fish passage, altering fish habitat associated with the change in flow regime and impeding navigation of the rivers. The loss of agricultural land would receive substantial opposition that would almost certainly delay or possibly ultimately stop the approval process. **Therefore, it has been concluded that large dams on the Souris River should not be considered further due to the significant environmental and operational concerns, as well their relatively high costs compared to other flood mitigation solutions**. Furthermore, it has been concluded from analyses of these sites that similar options elsewhere in the area would also not form viable solutions.

Flood protection projects are currently underway at Souris and Melita to provide flood protection to the highest flood on record (the 2011 Flood) at a cost of approximately $8 to $10 million. **Linear diking has been selected as the most feasible means of flood protection for communities on the Souris River**. The ongoing projects at Melita and Souris will provide a flood protection level approximately equivalent to a return period of 1:180 years and 1:150 years, respectively. The study also evaluated options to increase the flood protection in

these communities to the 1:200 years and the 1:500 years events. The proposed diking now underway would have to be raised by 0.1 m (0.3 ft) to 0.7 m (0.9 ft) in Melita at an estimated cost of approximately $177,000 to $1,300,000. In Souris, the dikes would have to be raised by 0.5 m (1.6 ft) to 2.4 m (7.9 ft) at an estimated cost of $1,645,000 to $3,935,000. In Wawanesa, the dikes constructed in 2011 are higher and the community is protected to the flood of record and the 1:200 year flood. The dikes would have to be raised by 0.2 m to 0.8 m (0.7 ft to 2.5 ft) for 1:500 year protection at an estimated cost of $1,492,000.

E. DAUPHIN LAKE

Dauphin Lake is located about 10 km (6.2 mi) due east of Dauphin, Manitoba. The lake is about 42 km (26 mi) long with a width that varies from 8 km (5 mi) to 18 km (11 mi).The lake has a surface area of approximately 500 km^2 (193 mi^2). Since about the mid-1960s, lake levels have seldom been allowed to fall below El 260.0 m (El 852.8 ft) with water levels below that threshold occurring only for a few days each year – the recorded minimum lake levels are approximately El 259.7 m (El 852 ft). Lake levels have exceeded El 261.5 m (El 857.94 ft) in five separate years since the mid-1960s. The maximum recorded water level on Dauphin Lake of El 262.47 m (El 861.1 ft) occurred during the 2011 Flood and was estimated to have a return period of 1:110 years.

Water levels on Dauphin Lake are regulated by Mossy Dam, which is located on the Mossy River about 150 m (492.0 ft) downstream of the outlet from the lake. During severe inflow events, such as in 2011, all the stoplogs would be removed from this structure to promote as much outflow as possible. During dry years and during the winter period the stoplogs would be installed to limit outflow and maintain water levels as high as would be appropriate.

Currently, the strategy for regulating water levels on Dauphin Lake and the existing structure and channel are unable to achieve the passage of large runoff events quickly enough to prevent large increases in lake levels. Methods to improve the regulation of Dauphin Lake fall into two broad categories, both of which were considered for the study:

1. Reduce and delay the inflow volume into the lake by temporarily storing runoff in the headwaters. This method was analyzed by assessing the existing upstream storage and comparing the typical flood volumes of Dauphin Lake's tributaries to what could realistically be stored in the headwater areas. It clearly demonstrated that this option is impractical and would require an unrealistically large number of new reservoirs. Costs would be very large, and the social and environmental impacts would be unacceptable.

2. Increase the capability to release water from the lake. Two options were assessed:

    a) Construction of a separate diversion channel with a control structure to increase the potential for outflow from Dauphin Lake. A variety of diversion channels were considered, but the costs were found to be high (ranging from $144 million to $323 million), and the potential for reduction of lake levels during floods would be modest.
    b) Increase the conveyance of the Mossy River by excavation in conjunction with an enlarged control structure at the lake outlet. This option has the potential to achieve the desired performance, but the costs are uncertain at this level of study. They are at present believed to be less than $105 million for excavation up to 1 m (3.3 ft) in depth along the river. However, the environmental impacts from excavating in or

beside the Mossy River would be significant, and mitigation of these undesirable effects could eventually cause the cost of the option to expand as studies into the details of the impact evolve.

It is important to recognize that efforts to control rises in levels of Dauphin Lake during flood events would theoretically have an adverse effect on downstream water courses, including Lake Winnipegosis. However, numerical modelling was carried out in this study to show that reductions in peak water levels on Dauphin Lake by 1 m (3.3 ft), for example, would result in a rise of only about 8 to 10 cm (0.25 to 0.3 ft) on the much larger Lake Winnipegosis.

The study also investigated the concern from the public that the lake bottom is infilling with sediment at a rate that would soon reduce the already low capacity to absorb floods. Review of previous measurements of sediment concentrations in the main tributaries showed that the rate of sediment inflow is very small relative to the size of the lake. No significant impact on flood storage is expected for several hundred years.

It was concluded that protection to the highest flood on record through development controls and individual flood proofing initiatives would be the most viable means to improve flood protection at Dauphin Lake.

F. LAKE WINNIPEGOSIS

Outflow from Lake Winnipegosis typically contributes about three quarters of the inflow to Lake Manitoba. The study addressed several possible options to reduce the potential flood outflow from Lake Winnipegosis.

Occasional diversion of water north from Lake Winnipegosis could be achieved over a distance of about 5.9 km (3.7 mi) to Cedar Lake where the excess flow supplied to the Saskatchewan River might be used for additional generation of energy at Manitoba Hydro's Grand Rapids Generating Station. This concept was found to require an expensive channel excavation through the high ground between Lake Winnipegosis and Cedar Lake, and a pump facility would be required to overcome the fact that Cedar Lake is typically at least 1.5 m (4.9 ft) higher than Lake Winnipegosis, even during flood events. The attraction of additional energy generation at the Grand Rapids Plant was investigated. The analyses showed that the power benefits on average were a negligible amount compared to the large annual costs required to pay for the construction of the option. The capital cost of developing a system that could divert up to 283 m^3/s (10,000 cfs) to Cedar Lake was found to be approximately $325 million. If this project had been operational in the 2011 Flood, the peak water levels on Lake Winnipegosis and Lake Manitoba could have been 0.8 m (2.8 ft) and 0.4 m (1.4 ft) lower than what was experienced. On the other hand, there would be disadvantages:

- The environmental impacts of inter-basin transfer of water could be significant, to the extent that ultimate approval to construct would be slow and maybe not even possible.
- The effectiveness of the scheme would be dependent on diligent maintenance of a complex pumping facility that may be operated very infrequently, possibly with several decades between uses.
- The cost of the pump station was approximate only and detailed design of this uncommonly large pump facility could cause cost increases that have not been completely represented by the contingency allowance currently incorporated into the cost estimate.

- The diversion from Lake Winnipegosis would have limited effectiveness for future floods that could emanate from the southern portion of the watershed supplying runoff to Lake Manitoba, including diversions from extraordinary isolated floods in the Assiniboine River.

Occasional diversion of excess water east from Lake Winnipegosis could be achieved over a distance of about 54 km (34 mi) to Lake Winnipeg. Unfortunately, the difference in lake levels between these two water bodies is about 38 m (125 ft) and is not conducive to economical diversion of water. The cost of channel excavation, drop structures to dissipate excess energy, and other facilities amounts to more than $1.3 billion, and was not considered a feasible solution to flooding in Lake Manitoba.

While controversial, the option of controlling outflow from Lake Winnipegosis by constructing a control structure at the outlet of the lake on the Waterhen River would be an effective means of limiting flood inflow to Lake Manitoba. A structure that could hold back adequate water to reduce the peak water level in Lake Manitoba in a 2011 magnitude of flood could be constructed for a capital cost of approximately $58 million. This would include the estimated cost of mitigating environmental effects; but such effects are only approximated in this level of study; and it is possible that other impacts could arise on further more detailed study. As a general approximation, a targeted 0.5 m (1.6 ft) lowering of the peak water level on Lake Manitoba in an extreme flood such as 2011 could cause a 1.0 m (3.2 ft) rise in water level on Lake Winnipegosis. Given the history in Manitoba of aversion to such schemes where downstream benefits are accrued at the expense of upstream damages, this scheme was considered unlikely to be acceptable and was not considered beyond examination of the concept.

The alternatives for diverting water from Lake Winnipegosis to Cedar Lake or to Lake Winnipeg, as well as for controlling outflow from Lake Winnipegosis with a control structure on the Waterhen River are not viable solutions due to the relative costs compared to other alternatives for mitigating flooding on Lake Manitoba, and the environmental impacts associated with these alternatives.

There appears to be a modest potential for control of outflow from Swan Lake (located upstream of Lake Winnipegosis) through the development of a storage reservoir and to reduce flood flows downstream. However, the effect would be small because the watershed upstream of Swan Lake is limited; and its contribution to flood inflows to Lake Winnipegosis is relatively small. In addition, the environmental impact of temporarily storing excess flood water, thereby flooding shoreline on Swan Lake, is expected to be significant.

G. LAKE MANITOBA AND LAKE ST. MARTIN

Increase in discharge capacity from Lakes Manitoba and St. Martin is a viable and direct means to limit rises in lake level in flood conditions on both lakes. The scope of work for the report contained herein includes assessment of this concept. Subsequent to the start of the studies reported herein, KGS Group was retained by MIT to develop a two-stage process to advance the conceptual designs. The first phase of the adjusted study scope (called Stage 1) was similar to what was originally included in the terms of reference for the overall study of the Assiniboine River & Lake Manitoba Basins. However, it included additional effort to expedite this portion of the work and increase the level of detail.

The scope of work and methodology for Stage 1 included:

- Identify Outlet Options for Lake Manitoba & Lake St. Martin.
- Develop Screening Level Designs and Cost Estimates.
- Economic Assessment.
- Stage / Damage Relationships.
- Summary Report.
- Public Open House Event.

The work done in Stage 1 is addressed in this report. In this conceptual level of study, six Lake Manitoba outlet channel options (Option A to Option F) and two Lake St. Martin outlet channel options (Option JB and Option WP) were developed:

- Option A – Twinning the Fairford River.
- Option B – Channel south of Pinaymootang First Nation.
- Option C – Channel slightly less south of Pinaymootang First Nation.
- Option D – Channel following Birch Creek.
- Option E – Bypass channel north of the Fairford River Water Control Structure (FRWCS).
- Option F – Expansion of the Fairford River and FRWCS.
- Option JB – Reach 1, Reach 2 and Reach 3 to Johnson Beach.
- Option WP – Reach 1, Reach 2 and Reach 3 east of Willow Point.

Discharge capacities for the outlet channels from Lake Manitoba to Lake St. Martin ranging from zero (do nothing) to 425 m^3/s (15,000 cfs) have been considered. Discharge capacities for the outlet channels from Lake St. Martin to Lake Winnipeg ranging from 113 m^3/s (4,000 cfs) to 540 m^3/s (19,000 cfs) have been considered. A description of the channel options and a plan showing their conceptual alignments have been included in this report.

Based on recommendations from the Lake Manitoba and Lake St. Martin Regulation Review Committee and the Manitoba 2011 Flood Review Task Force, the governing principle for the design of the outlet channels from Lake Manitoba and Lake St. Martin was that there should be no net increase in flow to Lake St. Martin above the base condition of the existing channel built in 2011, which has a discharge capacity of 113 m^3/s (4,000 cfs).

A statistical analysis was conducted to generate combined flood volumes for the Assiniboine River and local Lake Manitoba inflow, given the limited hydrometric records and limited information on the correlation between floods in the two drainage basins. A numerical flood routing model was also developed to estimate peak water levels on Lake Manitoba and on Lake St. Martin, with and without outlet channels, for a range of combinations of flood volumes. For the 1:200 year flood event, based on the results of the flood routing model and the statistical analysis, the peak still water level on Lake Manitoba without an outlet channel was determined to be approximately El 248.9 m (El 816.6 ft). For the 200-year event, outlet channels with discharge capacities of 142 m^3/s (5,000 cfs) and 283 m^3/s (10,000 cfs) would reduce the peak water level on Lake Manitoba by approximately 0.24 m (0.8 ft) and 0.43 m (1.4 ft), respectively. On Lake St. Martin, the peak still water level without an outlet channel was determined to be approximately El 245.5 m (El 805.4 ft) for a 1:200 year flood event, and the outlet channel would reduce the peak water level by approximately 0.8 m (2.7 ft).

A cost estimate was prepared for each of the alternatives investigated. The Lake Manitoba outlet channel with the lowest estimated cost was Option E ($30 million) for 106 m^3/s (3750 cfs)). However, the hydraulic capacity of this option is limited, and it increases downstream water levels locally. Costs for Lake Manitoba outlet channels that considered greater hydraulic capacities ranged from $101 million (Option A for 142 m^3/s (5,000 cfs)) to $462 million (Option B for 425 m^3/s (15,000 cfs)). Costs for Lake St. Martin outlet channels ranged from $107 million (Option JB for 113 m^3/s (4,000 cfs)) to $330 million (Option WP for 538 m^3/s (19,000 cfs)).

Each outlet channel from Lake Manitoba and Lake St. Martin is viable from a constructability perspective. However, a discharge capacity for an outlet channel that would be sufficient to avoid flooding due to a 2011-sized event would be greater than 425 m^3/s (15,000 cfs). The high cost of construction of such an option makes this unjustifiable.

Each outlet channel option for Lake Manitoba, as well as the inlet to Reach 1 of each of the Lake St. Martin options, should incorporate a control structure to regulate outflows. Also, all outlet channel options for Lake St. Martin should incorporate a permanent access road to facilitate construction, maintenance and operation.

All outlet channel options should consider improvement of fish passage directly at the FRWCS. Improving fish passage at the FRWCS with the construction of a new external fishway would provide an opportunity to decommission the existing fishway, which is located within one of the FRWCS bays. This could improve the hydraulic capacity of the structure at a relatively low cost and could be implemented in a fairly short time frame.

A description of the general environmental concerns has been provided in the report, along with a relative ranking of the channel options. For all options, utilization of the lower portion of Buffalo Creek to pass flow on a permanent basis would have significant environmental impacts and increased potential for flooding of the Dauphin River communities. Opposition from stakeholders was expected to be strong, and this option was not recommended for these reasons.

On Lake Manitoba, the combined biophysical and social environment rankings of the options were the lowest for Option A and the highest for Option C. It has been concluded that Options A and F should not be considered further due to the significant environmental and social concerns. **Furthermore, the Lake Manitoba Outlet Channel Options C and D were preferred due to their cost effectiveness and the positive environmental rankings**.

On Lake St. Martin, the combined biophysical and social environment rankings of the options were equivalent for both Option JB and Option WP; and, therefore, both should proceed to the next stage of study. **However, due to the social environment concerns with having the Reach 3 outlet located at Johnson Beach, the Willow Point option of the Lake St. Martin outlet channel was the preferred alternative**.

Based on the concept designs, the estimated costs and environmental assessment, the following recommendations were developed:

- Lake Manitoba channel discharge capacities in the range of 142 to 283 m^3/s (5,000 to 10,000 cfs) should be considered for further review since lesser capacities will not adequately reduce lake levels and greater capacities are not justifiable due to the high cost.

- The Lake St. Martin Emergency Outlet Channel should be made permanent with a base discharge capacity of 113 m^3/s (4,000 cfs) to control Lake St. Martin levels.
- Additional discharge capacity to the Lake St. Martin Outlet Channel in the range of 142 to 283 m^3/s (5,000 to 10,000 cfs) should be considered for further review to match a Lake Manitoba outlet channel to ensure that flooding will not be aggravated on Lake St. Martin.

Based on the results of the study, later discussed in the report, and available funds, the Province has decided to proceed with preliminary design for the preferred alternatives with outlet channel capacities of 212 m^3/s (7,500 cfs) on Lake Manitoba, and of 326 m^3/s (11,500 cfs) on Lake St. Martin. The estimated costs of the projects are $230 million and $220 million, respectively.

A proposed scope for the next phase of planning (Stage 2) has been included in the report. The focus of Stage 2 will be to complete the preliminary design of the preferred alternatives allowing the next phase of design to proceed to the detailed design stage.

H. EFFECTS FROM SHOAL LAKES

The Shoal Lakes Watershed Study conducted by KGS Group investigated three alternative measures to mitigate flooding issues within the lakes: construction of a diversion / outlet channel from the Shoal Lakes to Lake Manitoba; construction of upland storage within the Shoal Lakes watershed; and, purchase of flood prone lands.

The alternative which had the greatest reduction of water levels on the Shoal Lakes was the system of flood diversion channels. **However, it has been concluded that the most attractive option from an economic perspective was the purchase of flood prone lands, which is currently being pursued by the Province**. The latter would have virtually no effect on water levels on Lake Manitoba, and the former would have only a very small, almost immeasurable effect. The implementation of channel diversions was estimated to cause at most a 3 cm (0.1 ft) rise in water level on Lake Manitoba during severe floods.

I. COMBINATIONS OF SMALL DAMS

A combination of small dams on tributaries of the Assiniboine and Souris rivers in Manitoba was considered as an option for flood mitigation. These dams would be operated strategically to reduce peak flood flows in the Assiniboine and Souris rivers. A combination of 21 small dams was studied to explore the possibilities of this concept.

The impact of operating these 21 small dams showed that the reduction in peak flood flows on the Assiniboine and Souris Rivers ranged between approximately 3% and 13%, depending on the location and size of flood. The reduction in peak flows on the Assiniboine River near Holland was optimistically estimated to be approximately 8%.

Operation of the dams to reduce peak flows on the tributaries was not a preferred flood mitigation strategy, as it would not provide optimal benefits downstream on the mainstem rivers. However, the dams could have relatively small local benefits that could justify further review in the future.

The estimated capital cost of the 21 dam sites is approximately $482 million. With such a high cost, potential environmental impacts, relatively small reductions in flood peak flows, and difficulties in effective operation to achieve the maximum benefits, **it was concluded that flood mitigation with a multitude of small dams was not a feasible solution when compared to other options**.

J. DEVELOPMENT CONTROLS AND LAND USE PLANNING

Manitoba's development controls to minimize the risk of flood damages were reviewed and compared to other jurisdictions. The review identified areas for improvement in the Province's development review and control strategy to minimize the risk of flood damages. In addition, the findings and recommendations presented in the 2011 Manitoba Flood Review Task Force Report and the Lake Manitoba, Lake St. Martin Regulation Review Report have been carefully considered. **The Flood Protection Level should be the 1:200 year flood or Flood of Record, whichever was greater, as is being adopted by the Province of Manitoba for the Province as a whole**. A higher level of protection could be justified on a risk based assessment.

The Lower Assiniboine River, Lake Manitoba and Lake St. Martin, as well as Dauphin Lake were recommended as candidates where designated flood areas should be considered. Following the development of updated flood control works, consideration should also be given to establishment of designated flood areas for the communities along the Souris River, and along the Upper Assiniboine River protected by linear diking systems. **Where there is no designated flood area, flood development controls should be more rigorously enforced by Planning Districts and Municipalities.** Additional support and mechanisms should also be considered for the Planning Districts and Municipalities to help improve enforcement.

K. ADJUSTMENT TO PRACTICES IN LAND USE

Provincial initiatives to reduce flood risk through land use policies and regulations have focused on wetland preservation. Recent practices associated with Forest Management Plans by the Province of Manitoba have been successful to a large extent. However, consideration should be given to policies and regulations to control conversion of grasslands, woodlands and marginal lands to cultivation. Additionally , incentive programs to convert high-runoff land uses to low-runoff uses should be considered.

Integrated Watershed is an approach that helps reduce the land's runoff potential as it relies on the application of multiple strategies. There is a need for better linkages between Integrated Watershed Management Plans (IWMP) and Land Use Planning to ensure the IWMP recommendations related to riparian and shoreline protection, flooding and surface water management are adequately considered and adopted into land use plans.

Manitoba's Surface Water Management Strategy endorses the principles associated with managing land use for the purpose of reducing runoff, and it focuses on regulatory approaches for wetlands. **Other land use changes associated with cultivation of grasslands and deforestation should also be emphasized**.

L. EXPANSION OF WETLANDS FOR FLOOD ATTENUATION

Quantification of the attenuating effects of prairie wetlands on varying magnitudes of floods is complex. This is particularly true for the Assiniboine River. A statistical analysis of long-term

precipitation and stream flow records in the Assiniboine River Basin did not find statistically significant effects of drainage on peak flows and annual flows. Early research indicated that the drainage of prairie wetlands had a relatively minor impact on the peak flows of low frequency flood events such as the 2011 Flood in the Assiniboine River and Lake Manitoba basins. However, the most recent modelling on the role of prairie wetlands on flooding by Pomeroy indicates that wetlands drainage may play a significant role in flooding. This work is unquestionably the best available tool for simulating the hydrology of prairie watersheds including intact and drained wetlands. However, the complexity of the wetland drainage issues demands more thorough basin-wide and geographically-based assessments to verify the model and Pomeroy's findings.

Based on Pomeroy's findings, a basin wide analysis was conducted to determine the viability of wetlands restoration as a flood mitigation solution. It was found that the estimated cost of wetland restoration to obtain a 20% to 30% reduction in flow on the Assiniboine River would range between $1.0 billion to $1.5 billion. **With such high costs, large scale wetland restoration was not identified as a cost effective mitigation option. However, smaller scale projects could be pursued as they arise.** Manitoba's surface Water Management Strategy endorses "no net loss" in wetlands and encourages water retention, which should continue to be promoted. **A multi-year pilot study on wetland restoration should be considered to determine the costs and benefits of wetland restoration on flood attenuation, and the cost would be in the range of $10 to $20 million.**

M. CONTROL OF DRAINAGE IN SASKATCHEWAN

Interprovincial drainage has been a long standing and very contentious issue between Manitoba and Saskatchewan, especially in the Upper Assiniboine River Basin. In the past number of years, Manitoba and Saskatchewan have cooperatively tried to resolve the interprovincial drainage issues. A resolution of the interprovincial drainage issue has been hampered by the lack of clear scientific understanding of the role prairie wetlands play on flood attenuation for varying flood magnitudes. This has contributed to the fact that no acceptable resolution has yet been reached. There is clearly a need for a better forum to resolve interprovincial water management issues such as drainage. Possibilities are to expand the mandate of the Prairie Provinces Water Board or Western Water Stewardship Council, or to establish an interprovincial commission or dispute board.

An analysis of streamflow data identified increasing trends in watershed response in the Upper Assiniboine River Basin. Hydrologic analysis of streamflow records indicated that both peak flows and runoff volumes appeared to be increasing. The difficulty was isolating and quantifying the impact of drainage practices from other land management practices. Changes in drainage practices were likely the predominate issue impacting the increasing runoff response of the watershed. Drainage practices generally resulted in gradual change, and as such, were not easily quantifiable and isolatable through the analysis of streamflow data. Ideally, it could take a paired watersheds assessment with the same land use practices, but with different drainage practices: one in a drained state, and the other in an undrained state, to provide more conclusive data.

Saskatchewan recognizes their growing problems with illegal drainage. However, there appears to be no immediate solution to the problem. Saskatchewan is proposing new legislation to address the illegal drainage activities that also impact Manitoba. Development and implementation of new legislation along with enforcement, however, will take time. **Through the**

Assiniboine River Basin initiative, Manitoba will have opportunities to express their concerns to all jurisdictions regarding the interprovincial drainage issue.

8. BASE ANALYSIS OF INDIVIDUAL OPTIONS AND PREFERRED MEASURES

The study initially addressed all of the options summarized above on an individual basis to determine which would upgrade protection in areas that do not currently meet the provincial standards for flood protection. **Based on the results of the analyses, application of the economic guidelines would not lead to the justification of flood protection for infrastructure in the study area that exceeded the basic requirements of either the 1:200 year flood or the flood of record**. This finding was applicable to all options and combinations of options devised in this study.

This conclusion in turn provided a clear direction for the remainder of the study – **the best measures that could upgrade the level of flood protection for infrastructure, where necessary, should be planned for at least the 1:200 year flood, or the flood of record, whichever is greater, at all locations within the study area**. As discussed in the report, many areas of the Assiniboine River and Lake Manitoba basins already meet that standard. Although some areas meet the previous 1:100 years criteria and not the recent 1:200 years criteria, there have been significant strides towards upgrading flood protection such as the Individual Flood Proofing Initiative and the Community Flood Protection Program.

As a result of these comprehensive and in some cases, on-going, programs, focus was then directed towards a means to improve the level of protection on the Lower Assiniboine River, east of the Portage Diversion and west of the City of Winnipeg. **It was concluded that the protection along the Assiniboine River that would comply with the standard adopted by the Province of Manitoba would require the ability to safely pass the 1:200 year flood. That is a flood of approximately 1,620 m^3/s (57,100 cfs) at Holland, and exceeds the flood of record (2011 Flood) by about 10%.** It became obvious that no single option studied would satisfy the required upgrades, so the study then concentrated on combinations of the options with relatively high economic indicators (primarily benefit / cost ratios) that could provide the desired level of protection. They were as follows:

- Expansion of the Portage Diversion.
- Expansion of the outlet works at Lake Manitoba and Lake St. Martin.
- Enhancement of the dikes along the Lower Assiniboine River.

There are other options that could be interchangeable, such as the Holland Dam, within the above list, but would be far more costly or would have significant environmental and/or socio-economic impacts. The best combination and the extent of each of the above options has been derived through a logical process, as summarized below:

- The Portage Diversion can be readily expanded to nearly double the discharge capacity of its original design. However, as described in the report, the cost of increasing to a discharge capacity that exceeds approximately 960 m^3/s (34,000 cfs) increases sharply.
- The dike protection along the Lower Assiniboine River could be upgraded to reach river flows well in excess of 655 m^3/s (23,100 cfs); but the option was limited by the river flow that could be tolerated within the City of Winnipeg; and the cost was more than the cost of upgrades to the Portage Diversion.

- The outlets at Lake Manitoba and Lake St. Martin should consider an increase in discharge capacity in the range of 142 to 283 m^3/s (5,000 to 10,000 cfs) since lesser capacities will not adequately reduce lake levels and greater capacities were not justifiable due to the high cost.

Consideration of these limits has identified a preferred combination of options that provides the protection against a 1:200 year flood event. It consisted of the following:

- **Expansion of the Portage Diversion to reach a design discharge capacity of 960 m^3/s (34,000 cfs) at an estimated cost of $373 million for the Retrofit Option.**
- **Upgrades to the Lower Assiniboine River capacity to permit passage of a flow of 655 m^3/s (23,100 cfs) at estimated costs of $273 million for upgrades to the existing dikes in their current location and $63 million for a Combination of Individual Flood Proofing, Neighbourhood Diking and Purchase of Vulnerable Properties. This would be tolerable in the City of Winnipeg without any special upgrades to flood protection in that area.**
- **Construction of a new Lake Manitoba outlet with a capacity of 212 m^3/s (7,500 cfs) and making the Lake St. Martin outlet channel permanent with a capacity of 326 m^3/s (11,500 cfs), as committed to by the Province of Manitoba, at an estimated cost of $450 million for the preferred alternatives.**

Economic assessment of this combination has revealed that increasing the level of protection beyond this threshold would not be cost effective.

A hydraulic analysis of these combined measures has shown that water levels on Lake Manitoba could be lowered by approximately 0.7 m (2.4 ft) in a repeat 2011 Flood event, and could maintain water levels below flood stage for a repeat 2014 Flood event. This 0.7m (2.4 ft) reduction is more than the reduction provided by a Lake Manitoba Outlet Channel on its own due to the increased capacity of the Lower Assiniboine River, which can reduce the volume of water that is diverted to Lake Manitoba.

Key environmental and social impacts that could be of concern for each mitigation alternative have been presented in the report. It is anticipated that all three projects of the preferred alternative would require licencing under the *Manitoba Environment Act*, however, only the Lake Manitoba and Lake St. Martin outlet channels are anticipated to require approval by the Canadian Environmental Agency.

Examination of the effects of future trends in more frequent occurrences of major floods revealed that:

- **Use of only the historical record of recent decades of flood peaks would tend to increase the best estimate of the 1:200 year flood in most areas. The increase would vary over the study area.**
- **Estimates of damages averted by any specific improvement in flood protection would increase on an average annual basis and enhance the economic incentive to proceed.**

- **On the other hand, achieving a minimum protection of a 1:200 year flood that is determined based on only the historical record of recent decades of flood peaks rather than the full period of record, would require more extensive protection works and would become more expensive.**

9. RECOMMENDATIONS

Based on the study results and the public open house processes undertaken as a part of the study, the following recommendations are presented:

1. The Lower Assiniboine River has been identified as an area that has infrastructure vulnerable to floods much smaller than the highest flood on record. It is recommended that this area be protected to a target Flood Protection Level of 1:200 years, which is consistent with the 2011 Flood Review Task Force recommendations. This increased protection would consist of upgrading the Portage Diversion to 960 m^3/s (34,000 cfs), upgrading the Assiniboine River capacity to 655 m^3/s (23,100 cfs), constructing a new Lake Manitoba outlet with a capacity of 212 m^3/s (7,500 cfs) and a Lake St Martin outlet channel with a capacity of 326 m^3/s (11,500 cfs). The estimated cost for this recommended flood mitigation system is $1,159 million and is summarized in Table 23.1. Considering the current low standard of protection within the Lower Assiniboine River and the Province's current commitments for Lake Manitoba and Lake St. Martin, these areas should be given the highest priority.

Table 23.1
Summary of Estimated Cost of Preferred Measures
for Upgrading Flood Protection for the Lower Assiniboine River

Flood Protection Infrastructure	Proposed Capacity	Preliminary Estimated Costs for Upgrade
Provincial Assiniboine Dikes	654 m^3/s (23,100 cfs)	$273 Million
Assiniboine River Baie St. Paul to Headingley	654 m^3/s (23,100 cfs)	$63 Million
Portage Diversion	963 m^3/s (34,000 cfs)	$373 Million
Lake Manitoba Outlet	212 m^3/s (7,500 cfs)	$230 Million
Lake St. Martin Outlet	326 m^3/s (11,500 cfs)	$220 Million
Total Estimated Cost		**$1,159 Million**

2. Development controls were considered as a means to restrict future development. Where there are no Designated Flood Areas (DFA) local municipalities and planning districts should more rigorously enforce flood protection levels in new developments and DFAs should be considered for the Lower Assiniboine River, Lake Manitoba and Lake St. Martin, as well as the Dauphin Lake areas.

3. The provincial flood protection policy should be revised to require flood protection to at least the 1:200 year flood, or the flood of record, whichever is greater, at all locations within the study area. This is consistent with recommendations from the Manitoba 2011 Flood Review Task Force Report.

4. The purchase of the flood prone land along the Upper Assiniboine River between Shellmouth Dam and St. Lazare was preferred over linear diking and should be considered further. The estimated cost was approximately $24 million for the flow of 85 m^3/s (3,000 cfs).

5. A multi-year pilot study on wetland restoration should be considered to determine the costs and benefits of wetland restoration on flood attenuation. The cost would be in the range of $10 to $20 million.

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1.0 INTRODUCTION

Since the "Great Flood" of 1950 Manitoba has proactively developed an extensive integrated flood protection system consisting of floodways and diversions, dams, community diking, linear diking adjacent to flood-prone waterways, individual flood protection measures, and controls on development. This integrated flood mitigation strategy has evolved over decades and has worked well as a system since that time in the areas protected by the constructed works. It has avoided billions of dollars in damages throughout the Province.

In spite of this commendable history of flood protection, in the last hundred years there have been at least six major floods that have caused damage in the area around the Assiniboine River, the most recent in 1976, 1995 and 2011. Adjacent to the Souris River, there have been at least six major floods, the most recent in 1976, 1999 and 2011. Each event was unique in peak discharges, total runoff volumes and the timing of the flood hydrographs. Serious damaging floods on Lake Manitoba have been less frequent, with the recent most serious before 2011 occurring in 1954, 1955 and 1956. Except for the flood of 2011, major flooding has not occurred around Lake Manitoba since the Fairford River Water Control Structure (FRWCS) was completed in 1961 (Manitoba, 1961). The previously recorded highest water level occurred in 1979, and was 1.2 m (4 ft) below the 2011 peak water level, and 0.84 m (2.8 ft) below the peak of 1955.

The flood of 2011 in southwestern Manitoba was significant for a number of reasons. However, the most important was that virtually every Assiniboine River and Souris River tributary carried near record peak flows and volumes of runoff. Although not quite as extreme, very high flows were recorded on almost all streams and rivers in the Lake Manitoba watershed as well.

This escalating trend of flood damages in southwestern Manitoba indicated the need for refocussing on the possible expansion of the flood protection system that has served the Province so well in the past. To address the propensity for flooding in the Assiniboine / Lake Manitoba drainage basins, the Province of Manitoba, led by Manitoba Infrastructure and Transportation (MIT), began a study of the floods and the possible means that could be devised to improve protection for the diverse interests in the region. A team led by KGS Group was selected in a competitive process to carry out the conceptual engineering studies critical to

advancing the new protection measures towards construction. KGS Group's team consisted of skilled in-house engineers and scientists, as well as the specialized skills offered by outside consulting firms, InterGroup Consultants (Winnipeg) and Northwest Hydraulic Consultants (Edmonton), and selected external advisors who are well known experts in Manitoba. The principal advisors were Messrs. R. Bowering, B. Harrison, R. Hay, and A. Warkentin all of whom held responsible positions for many years with MIT and its predecessor government departments, and all of whom are well known and respected in the field for their expertise in flood protection in Manitoba.

The process devised by MIT was unique. It involved the deployment of the usual Steering and Technical Committees comprised of staff from MIT, Manitoba Conservation and Water Stewardship (C&WS), University of Manitoba, and Manitoba Agriculture, Food and Rural Development (MAFRD). But it also included input from a Study Liaison Committee (SLC) that had a broader perspective than the engineering focus of the Steering and Technical Committees. In addition to the representation previously identified, the SLC also included representatives from various key towns, cities and municipalities (e.g., Winnipeg and Brandon), as well as from other organisations and local citizen groups (e.g., Emergency Measures Organization (EMO), Manitoba Metis Federation (MMF), Ducks Unlimited, Association of Lake Manitoba Stakeholders (ALMS), and more). A complete list of the SLC members is included in Appendix A. The process included a Workshop held at the initiation of the study, which was attended by almost all the experts in their field related to flood protection, including representatives from municipal, provincial and federal government departments, as well as from the consulting community and the major universities in Manitoba.

The process also included a series of three Public Open Houses held specifically to explain the objectives of the study to the public and to gather input from the stakeholders who have been directly affected by the floods.

1.1 STUDY GOALS AND OBJECTIVES

The goal of Manitoba Infrastructure and Transportation (MIT) was to initiate a conceptual level study of the watersheds of the Assiniboine River and Lake Manitoba. KGS Group was retained in 2012 to carry out this study and provide recommendations for flood protection along the main

stems of the Assiniboine River including Winnipeg, the Souris River, the Qu'Appelle River, and around Lake Manitoba, Lake St. Martin, and Dauphin Lake. It was necessary to examine a wide range of possible measures of both non-structural strategies and structural options. In addition to the conceptual study of these alternatives, the potential benefits, costs, implications and risks associated with the various options have been quantified. This was carried out with the benefit of a wide range of previous studies of the region done over many decades. Inputs from other recent and notable studies were carefully considered:

- The studies by Manitoba 2011 Flood Review Task Force, chaired by David Farlinger.
- The Lake Manitoba and Lake St. Martin Regulation Review Committee, chaired by Harold Westdal.

A number of initiatives for flood mitigation in the region were well underway during the course of the current studies, and they have also been considered in the study assessments and analyses.

A comprehensive analysis of the interconnected flood mitigation system has been completed, including the Shellmouth Reservoir, Assiniboine River, Dauphin Lake, Lake Manitoba, Lake St. Martin, and the City of Winnipeg, as a single unit to identify flood mitigation options to reduce flood risks within that part of the basin. All work has been at a conceptual level of study so as to permit assessment of a wide range of options within the limits of a reasonable budget. While further intense engineering would be required to carry forward the specific recommended schemes identified in this study, the level of study is considered adequate to lead to firm conclusions and directions for detailed design.

1.2 SCOPE OF WORK

The detailed scope of services for this study was based mainly on the Request for Proposal (RFP) issued by Manitoba Infrastructure and Transportation (MIT). There were also several revisions and expansions in scope that evolved during the course of the study. The final scope and the general approaches taken to fulfill that scope are summarized below.

Existing Vulnerabilities – the Engineering Service Provider (ESP) was required to identify all of the major vulnerabilities from floods throughout the study area. KGS Group undertook this

task and prepared a comprehensive description of these vulnerabilities. Emphasis was placed on each of the designated project areas, with respect to infrastructure integrity, operational and public safety vulnerabilities, and potential improvements. An approach was developed similar to that taken in the studies of vulnerability to floods in Winnipeg by KGS Group for the International Joint Commission (IJC) in 1999. Vulnerabilities, consequences of failure or of insufficient capability to protect, and a rating of the urgency for execution were identified and tabulated.

Numerical Modeling – MIT considered that it would be necessary for the ESP to develop one or more computer models for assessing the hydrologic and hydraulic aspects. To this end, a number of commercially available and in-house hydraulic and hydrologic models were used to assist in the evaluation of various flood mitigation options. A number of numerical models were developed to the extent that was deemed appropriate with the limited available data. The application of sophisticated numerical models such as MIKE-11 or FLOW 3D was not deemed justified at this conceptual stage of study and screening of options.

Assessment of Mitigation Options – MIT requested that the ESP consider a wide variety of means to mitigate the effects of floods. All of the options requested by MIT were considered, as well as other options identified during the course of the work. A brief outline of these options is described below.

- **Development Controls –** The study considered ways in which development could be limited, or modified to reduce flood damages, flood fighting and other costs. Vulnerabilities for communities and the City of Winnipeg were identified, including any significant developments within the study area.
- **Restrictions in Land Use –** The study assessed the impact and potential benefits that land use has on the hydrology of an area to preserve, expand or enhance significant areas with lower runoff potential.
- **Expansion and Enhancement of Wetlands –** The study assessed the potential benefits of enhancing and increasing wetlands throughout western Manitoba, in reducing flood damages and how benefits vary for different flood magnitudes.
- **Flood Diversion Channels –** Potential new diversion schemes for flood mitigation were assessed for this study. As well, improvements and enlargements to the Portage Diversion and the Fairford River Control Structure were examined, including costs, benefits, and environmental and social impacts.
- **Community Dikes –** A number of diking initiatives have recently been completed or were in the process of being completed during the course of the study. The study

assessed the adequacy of existing and proposed dikes, as well as reviewed the feasibility of new diking concepts.

- **Flood Storage Reservoirs –** A number of potential small to large dams for flood storage have been assessed in past decades. The ESP was required to address the most promising of those options and update studies of the largest dams. A comprehensive review of the previously studied sites, as well as promising additional sites with suitable locations for storage areas was undertaken for the current study, including additional storage capacity in the Shellmouth Reservoir.

- **Restriction of Flood Inflow from Lake Winnipegosis –** The largest portion of flows into Lake Manitoba comes from Lake Winnipegosis. The ESP was requested to consider the potential to regulate Lake Winnipegosis to enhance flood protection downstream. Regulation was considered in this study, as well as flood diversion channels from Lake Winnipegosis that could reduce flood levels downstream.

- **Enhancements of Dikes along the Lower Assiniboine –** The dikes on the Assiniboine River downstream of Portage la Prairie were tested to their limits in the flood of 2011. The study identified a comprehensive means of enhancing the discharge capacity of the river within the diked area, including the corresponding implications, complete with benefits and costs.

- **Protection at Lake Manitoba –** The Province has initiated a flood mitigation program to provide financial assistance for individual flood protection works. Vulnerabilities that will remain after the current programs are completed have been reviewed and are reported in this study. Recommendations have been made on how these residual flood risks can be mitigated. This included the estimation of benefits, costs and implications of developments and operation. As well, the study assessed options to increase outflow from Lake Manitoba and Lake St. Martin, in alignment with the conclusion of the Lake Manitoba/Lake St. Martin Water Level Regulation Studies (LM&LSMRC, 2013).

- **Effect of Options at Shoal Lakes –** The study was required to incorporate the findings of a recent watershed study of the Shoal Lakes and incorporate them into the assessment of potential flood mitigation options for the region.

- **Temporary Flood Mitigation Options –** The ESP was required to assess in which geographic locations temporary flood mitigation options (sandbags, aquadams, flood tubes, etc.) would be most sensible. The study considered this and identified locations where temporary measures could be used to extend the flood protection level.

Benefit / Cost Analysis and Prioritization – MIT required the ESP to evaluate and quantify the benefits, costs and other implications of all the alternatives studied. This was undertaken at a conceptual level of study with a systematic approach that followed a set of economic guidelines established and agreed in advance by MIT. Benefit / cost analyses were undertaken for all options deemed to be potentially feasible.

Environmental and Social Impacts of the Alternatives – The ESP was requested to identify the environmental and social impacts of the options recommended. The study has complied with this requirement by addressing all resources that would be impacted and by identifying all significant environmental issues.

Public Consultations and Technical Workshop – A technical workshop that invited over 75 technical people with diverse experiences in Manitoba floods from various government and non-government sectors was held to provide a mechanism for input at the beginning of the study. As well, public Open-House events were held across the Province at the beginning and near the end of the study to present the objectives and findings of the study.

In addition to the original scope for the study agreed in 2012, a variety of supplementary tasks were requested. These included (in summary form):

- A concept design development for improvement in the Lake Manitoba and Lake St. Martin Outlets, including bathymetric surveys.
- Miscellaneous assistance to MIT with public presentations.
- More intense studies of options to improve the dikes along the Lower Assiniboine River.
- Review of drainage practices and their effects on Assiniboine River flows.
- Additional effort was directed in certain areas beyond the concept level of study that was intended for much of the study scope. This included, for example, enhancement of selected numerical models and analyses, where appropriate, for legacy tools and a detailed study of upgrades to the Portage Diversion.
- Studies for providing flood protection to individual homes and communities along the Lower Assiniboine River.
- Updates to the study needed to include the statistics of the significant flood that occurred in the spring of 2014.

Given the wide geographic range and the vast number of mitigation options available, there has been a need to limit the studies generally to a conceptual level (except for the more detailed studies included in the extra scope items listed above). Although conceptual and approximate, the study results are considered to be appropriate as the basis of a planning tool for the development of a long-term plan for flood protection in the Assiniboine Valley and Lake Manitoba Drainage Basin.

2.0 STRUCTURE OF REPORT

This report documents the studies conducted and the findings that evolved from over two years of intense engineering studies. It provides a comprehensive record of the process followed by KGS Group and its sub-consultants.

The report is prefaced by an Executive Summary that concisely summarizes the various studies and highlights the findings from each. Conclusions are identified in bold font, with a list of recommendations at the end of the summary.

The report proceeds through the introduction in Section 1.0, which describes the study goals and objectives, as well as scope of work that was prescribed to KGS Group and its sub-consultants.

Section 2.0 describes the structure of the report, which was devised to characterize and document the studies completed and the findings and conclusions.

Section 3.0 summarizes the comprehensive history of documents that were available for the study to launch new assessments and devise means to improve the flood protection system in the region.

Section 4.0 focusses on the process that the study followed to identify existing vulnerabilities in flood protection in the study region. It culminates with a tabular summary of over 100 vulnerabilities and ranks them in terms of importance and the need for resolution. This assisted in forming a basis for the subsequent studies described in this report of needed upgrades to existing facilities.

Section 5.0 addresses the complex hydrology of the study region, including the statistical analyses of flood events, the inter-relationships between flood events in the diverse geographical corners of the study region, as well as speculation on the potential trends in flood potential due to climate change, drainage development and upstream influences beyond the control of the Province of Manitoba.

Section 6.0 introduces a wide array of protection measures that were considered and addressed during the study. Specific measures that were considered for each part of the study area are discussed in separate sections from Section 9.0 to Section 17.0, inclusive.

Section 7.0 describes the basis for the economic analyses that were agreed with MIT and the two levels of steering committees, and subsequently carried out during the study.

Section 8.0 describes a wide array of numerical models that were developed for the study or provided by MIT. It culminates with a list of the numerical tools that will be turned over to MIT as a legacy of this study.

Section 9.0 is the first of nine sections addressing the options for improving flood protection in various zones within the study area.

Section 18.0 discusses development controls and land use planning strategies in Manitoba and identifies areas where designated flood areas should be considered within the study basin.

Section 19.0 focuses on land use changes and their potential impacts on hydrology.

Section 20.0 addresses the controversial issue of wetland restoration and its potential effect on flood mitigation in the study area.

Section 21.0 addresses drainage development in Saskatchewan and the growing view by Manitobans that it is exacerbating flood conditions in Manitoba.

Section 22.0 discusses and provides summary results of studies of the vast array of potential options for improvement in flood protection in the study area. It focusses on the studies of each option on an individual basis, without consideration of the synergy of combinations. The findings of those analyses on individual options formed a strong basis for proceeding to identify effective combinations of options for upgrading flood protection in the Lower Assiniboine River.

Section 23.0 focusses on the primary area where flood protection must be improved to meet provincial standards. This area is the Lower Assiniboine River between Portage la Prairie and the City of Winnipeg limits in Headingley.

Section 24.0 identifies and discusses environmental and social impacts that could be expected with the preferred alternatives for improving flood protection.

Section 25.0 summarizes the process of public open houses and the key input that was provided through those forums.

Finally, Sections 26.0 and 27.0 itemize the various conclusions reached during the studies, and list the recommendations offered by the study to enable the Province to move forward into more definitive levels of design and ultimately construction of effective works.

3.0 BACKGROUND REPORTS

A key component of the study was to become familiar with previous work and studies relevant to the current study area and project scope. Therefore, over 180 reports were reviewed as part of this task. About 160 of these reports were reviewed immediately after the initiation of the project. These reports were categorized in a detailed tracking system to summarize the following information regarding the documents:

- MIT's Document Number (if applicable).
- Date of Report.
- Title.
- Author(s).
- Number of Pages.
- Brief Synopsis.
- Reviewer Comments (considering current study scope).

The entire list of reports with categorized details is provided in Appendix B.

Reports reviewed varied widely in topics, authors and dates. The review was generally limited to reports prepared after approximately 1950. Examples of the diversity of the authors included various departments of Manitoba Governments, Engineering Consultants, Manitoba Water Commission, Saskatchewan Nelson Basin Boards, University Professors and other areas of research, governance and consulting. The wealth of information gained through the review of the reports was quite large, providing a broad spectrum of potential projects. Included were a number of potential reservoir sites not only on the Assiniboine River but the Souris River, Qu'Appelle River, Shellmouth River and others. It was found that, of the water retention studies, many were linked to water supply and did not address flood protection benefits in the benefit / cost analysis. Display of a regional approach was not evident. Some reports attempted to provide the estimated extent of flood risks. However, most mapping used appears now to be out of date.

All of the reports reviewed formed background information for the current study. Specific pertinent details identified through the review process have been referenced in this report where applicable.

In addition to these 160 reports, a literature review was conducted of an additional 26 reports specific to the role of prairie wetlands in flood attenuation. A summary and synopsis of each of these reports is provided in Appendix B. The findings of this literature review formed the basis of the analyses of wetlands later discussed in Section 20.0 of the report contained herein.

During the review process, the study also identified background information on flood protection models in other jurisdictions outside Manitoba. One area of particular interest was California, where the state has begun a long process of improving flood management systems by investing heavily to complete emergency repairs near high risk urban areas, evaluating dikes that protect densely populated urban areas, informing the public about flood risks, enacting new laws, and providing funds to lead a sustained effort to improve flood management state-wide. The California Department of Water Resources has launched “FloodSAFE California” – a multi-faceted program to improve public safety through integrated flood management. The program builds on recent progress in flood protection initiatives and is fuelled by almost $5 billion provided through approved bond measures. Such intense investment will clearly result in the establishment of programs and standards that will be of great interest to Manitoba.

3.1 POST 2011 FLOOD REPORTS

Several initiatives were launched by the Province of Manitoba in response to the 2011 Flood event including the 2011 Flood Review Task Force, chaired by David Farlinger, and the Lake Manitoba Lake St. Martin Regulation Review Committee, chaired by Harold Westdale.

The 2011 Flood Review Task Force applied a method of review for the 2011 Flood that investigated many different social, political and technical issues that arose from the many components of the flood. Operations of the Province were reviewed, such as flood forecasting, operation of water control structures and adequacy of existing flood control infrastructure. In conjunction, flood response procedures such as mitigation measures, preparedness and fighting capacity were analyzed. Other policy and governing issues such as communications to the public, land use policies and zoning, environmental impacts, and flooding on First Nations land were also reviewed.

The 2011 Flood Review Task Force released their report, which included 126 recommendations in April 2013. The results and recommendations of the 2011 Flood Review Task Force (MFRTF, 2013) formed the basis of the current study. Key recommendations that guided some of the assumptions and flood mitigation measures investigated in the current study are listed below, and include in bold the sections of the current report that were influenced by these recommendations.

7. *A study of the diking system on the Lower Assiniboine River be conducted to assess the potential implications of its failure, and strong consideration be given to acting on the recommendations arising out of that assessment.* ***(Section 10.1)***

74. *Examine and adopt means to accommodate any impact on hydrology and global warming and climate change when planning, designing and implementing future water use projects.* ***(Section 5.2.3)***

75. *Investigate the feasibility of developing storage on the major tributaries that contribute runoff to Dauphin Lake including Valley River, Vermillion River, Ochre River and Turtle River.* ***(Section 13.5)***

76. *Investigate the feasibility of developing a second controlled outlet from Dauphin Lake into Lake Winnipegosis. Local residents suggest that a possible location for this additional outlet channel would be east of, and parallel to, the Mossy River.* ***(Section 13.5)***

77. *Investigate alternative means to prevent or reduce flood damages on the Assiniboine River below Shellmouth Dam:*

    - *Continue to pursue the installation of spillway gates on the Shellmouth Dam, but examine and quantify the flood control benefits and assess any impact on other uses served by the reservoir.* ***(Section 9.2)***

    - *Investigate the merits of a buy-out of flood-prone valley bottom lands downstream of Shellmouth Dam, which would increase operational flexibility of Shellmouth Dam and eliminate future downstream flood damages and liability.* ***(Section 9.4)***

    - *Investigate the feasibility of constructing dikes along the margins of the Assiniboine River below Shellmouth Dam to reduce flood damages on flood prone valley bottom lands and increase operational flexibility.* ***(Section 9.3)***

78. *Continue to work with the Province of Saskatchewan to better understand the impact of agricultural drainage on the hydrology of the Assiniboine River, and to identify and control drainage activities in both Saskatchewan and Manitoba.* ***(Sections 20.0 & 21.0)***

79. *Retain and maintain the upgraded dikes along the Portage Diversion Channel, as a contingency for future large flood events. Upgrade drop structures and bridge crossings where necessary.* ***(Section 11.3)***

80. *Upgrade the outlet structure on the Portage Diversion channel where it enters Lake Manitoba, to improve the hydraulic performance during passage of the design flow.* ***(Section 11.3)***

81. *Investigate means to reduce damages, which result from spills through the wasteway (fail safe) on the Portage Diversion:* ***(Section 11.2)***

    - *Raise both the east embankment and the crest elevation of the wasteway.*
    - *Acquire any additional land control that may be required to accommodate spills through the wasteway.*

82. *Upgrade and modernize the Portage Diversion headgate structure.* ***(Section 11.3)***

83. *Examine the feasibility of developing additional storage on the Assiniboine River (such as the Holland Dam) upstream of Portage la Prairie, which would reduce the frequency of operation and the magnitude of diversions through the Portage Diversion, provide water supply benefits in the region and reduce the reliance on the Shellmouth Dam for low flow augmentation in the Lower Assiniboine River.* ***(Sections 9.5 and 12.3)***

84. *Examine the feasibility and effectiveness of developing storage on tributaries to the Assiniboine River.* ***(Section 17.0)***

85. *Develop and implement a program to upgrade the Assiniboine River dikes between Portage la Prairie and Headingley. The optimal design capacity should be determined from detailed engineering studies now underway.* ***(Section 10.1)***

86. *As one component of a properly engineered diking system, determine the optimum location and capacity of a permanent controlled wasteway (either north or south of the Assiniboine River), and construct such a structure to pass Assiniboine River flows in excess of the combined capacity of the Portage Diversion and Assiniboine River channel dikes. The facility should be sized with due consideration of the potential flood risk/benefits and the capacity of the landscape to accommodate the design flow through the breach.* ***(Section 10.2)***

87. *Adopt the recommendations of the Lake Manitoba and Lake St. Martin Regulation Review Committee regarding additional outlet capacity requirements for Lake Manitoba and Lake St. Martin, with due consideration for the engineering studies that are being conducted.* ***(Section 15.0)***

88. *Examine the feasibility of alternative schemes suggested to the Task Force by local residents to reduce or control inflow to Lake Manitoba and thereby reduce the need for additional outlet capacity from Lake Manitoba and Lake St. Martin:*

    - *A diversion from Lake Manitoba to Lake Winnipeg* ***(Section 15.0)***
    - *A diversion from Lake Winnipegosis to Lake Winnipeg* ***(Section 14.3)***
    - *An outlet control structure on the Waterhen River to control outflow from Lake Winnipegosis* ***(Section 14.4)***

104. Implement clear policy measures to ensure future development does not knowingly occur on land subject to flooding without appropriate mitigation. Possibilities include: ***(Section 18.0)***

- *The development of mechanisms through district planning boards to prevent future developments below the Flood Protection Level (FPL).*
- *Expansion of Designated Flood Areas into regions that experience chronic flooding of developed properties.*

105. Develop and adopt a uniform Flood Protection Level (FPL) throughout the Province. Such a standard should strike a balance between long-term risk tolerance, the economic damages resulting from flooding, the economic costs of building to higher FPLs and the uncertainty in the estimated FPLs due to limited hydrologic data. ***(Sections 4.2, 22.0 & 23.0)***

106. Permit local developers and institutions to adopt a risk-based approach for determining a higher standard of flood protection where appropriate, to be implemented locally where justified from a benefit-cost analysis or where facilities are of a critical nature. The minimum standard would be enacted through regulation and the higher risk-based approach would be at the discretion of the authority or proponent investing in the new capital works. ***(Sections 4.2, 22.0 & 23.0)***

The Lake Manitoba and Lake St. Martin Regulation Review was developed by a committee of 12 members. Their report was released in April 2013 and made recommendations on the range of regulations for each of the lakes, the need for a new outlet from Lake Manitoba to Lake St. Martin and recommendations with respect to land use planning and zoning. The results and recommendations of the Lake Manitoba and Lake St. Martin Regulation Review Committee formed the basis of the current study. Key recommendations that guided some of the assumptions and flood mitigation measures investigated in the current study are listed below and include in bold the sections of the current report that were influenced by these recommendations.

9.1.1 The Committee recommends that the Emergency Channel be made permanent: **(Section 15.0)**

- *The capacity should be sufficient such that Lake St. Martin can be maintained within the desirable range of 797 to 800 feet at least 90 percent of the time.*
- *Consideration should be given to the inclusion of a control structure at the mouth of the permanent outlet channel to limit outflow at times of low lake levels.*
- *The current Reach 3 should be redirected so that the channel outlets south of Willow Point on Lake Winnipeg.*

9.1.2 The Committee recommends that a second channel be constructed between Lake Manitoba and Lake St. Martin that would provide the total outlet capacity to meet the original design criteria for the Fairford Water Control Structure. **(Section 15.0)**

9.2.2 The Committee recommends that in designing the permanent outlet from Lake St. Martin as recommended above, consideration must be given to both the desirable lake level range for Lake St. Martin (797 to 800 ft ASL) and to the effects of an additional outlet from Lake Manitoba to Lake St. Martin. **(Section 15.0)**

9.3.2 The Committee recommends that the Province:

- *Increase the standard for the Flood Protection Level to one based on a 1:200 year flood event.* ***(Section 4.2)***

- *Assist local developers and institutions in the application of a risk-based approach for developing a higher standard of flood protection, which would be implemented locally on an economic basis or where facilities are of a critical nature involving emergency response, important infrastructure or large public investment. The minimum standard would be enacted via regulation and the augmented risk assessment would be at the discretion of the authority investing in the new capital works. (****Sections 4.2, 22.0 & 23.0****)*

4.0 EXISTING VULNERABILITIES

The study terms of reference required that all major flood vulnerabilities throughout the study area be identified. These vulnerabilities formed the basis for planning the assessment of the current state of flood control works

4.1 TECHNICAL WORKSHOP

The process to confirm / refine vulnerabilities identified in this study included a technical workshop developed jointly by the Manitoba Infrastructure & Transportation (MIT) Steering Committee and KGS Group. Approximately 75 technical people with diverse experience in Manitoba floods were invited from various government and non-government sectors. The purpose of the workshop was to provide a mechanism for input from the attendees prior to KGS Group completing the comprehensive conceptual-level study that would identify and assess potential options to reduce flood risk for communities and major infrastructure in the Assiniboine River, Souris River, Qu'Appelle River, Dauphin Lake, Lake Manitoba, Fairford River, Lake St. Martin, Dauphin River and Shoal Lakes.

The workshop was held on February 6, 2013 and was an interactive process facilitated by KGS Group's experts. Prior to the workshop, attendees reviewed a preliminary List of Vulnerabilities and List of Mitigation Measures prepared for the study to gain familiarity and increase the effectiveness of the event. The workshop was organized in four sessions. In the first session, KGS Group outlined the terms of reference and the project objectives outlined in the original Request for Proposal (RFP). KGS Group had summarized key findings from an extensive list of background reference reports that had been reviewed, as previously discussed in Section 3.0.

In the second and third sessions, an overview of the preliminary List of Vulnerabilities and List of Mitigation Measures was summarized and an opportunity was provided for questions and clarifications. The attendees were then divided into six to eight breakout groups to provide a diverse representation of the experience within each group. Discussions were then focused on whether or not the groups generally agreed with the listed items, whether new items should be added to the lists, and on providing a preliminary ranking / rating of the most significant

vulnerabilities and mitigation measures. Results from each breakout group were then presented to the entire workshop group and further discussed.

In the fourth session, an overview of other key issues that could impact the outcome of the study was presented by KGS Group. The key issues discussed included:

- The potential for floods to increase in magnitude due to climate evolution / variability and changes in use / development.
- What is an appropriate period of record for historical analysis (e.g., 40 years, 100 years)?
- What is an appropriate flood protection level (solely a function of cost / benefit analysis)?

Following a brief question period, the key issues were discussed by the breakout groups and feedback was provided in the same manner as described for Sessions 2 and 3.

Results and input received at the workshop were incorporated into the study where applicable. Input received during Session 2 on the vulnerabilities was directly incorporated into the list of vulnerabilities that are discussed in Section 4.3. Input received on the List of Mitigation Measures during Session 3 was considered during the selection process of the final list of flood mitigation measures investigated during the study. This is further discussed in Section 6.0.

Input received during Session 4 on the period of record for historical analysis and on the potential for increasing flood magnitudes formed the basis of the hydrologic analyses for the study. It is discussed in Section 5.3. Input received on the appropriate flood protection level is discussed in the next section (4.2) of this report.

4.2 MANITOBA CURRENT POLICY FOR FLOOD PROTECTION

The target Flood Protection Level (FPL) varies by location across the Province of Manitoba and is based on the following criteria:

- The 1:200 year event, as recommended by the Lake Manitoba and Lake St. Martin Regulation Review Committee and the 2011 Flood Review Task Force, which is greater than the previous 1:100 year standard as discussed further below.
- The highest flood on record if greater than the 1:200 year event.

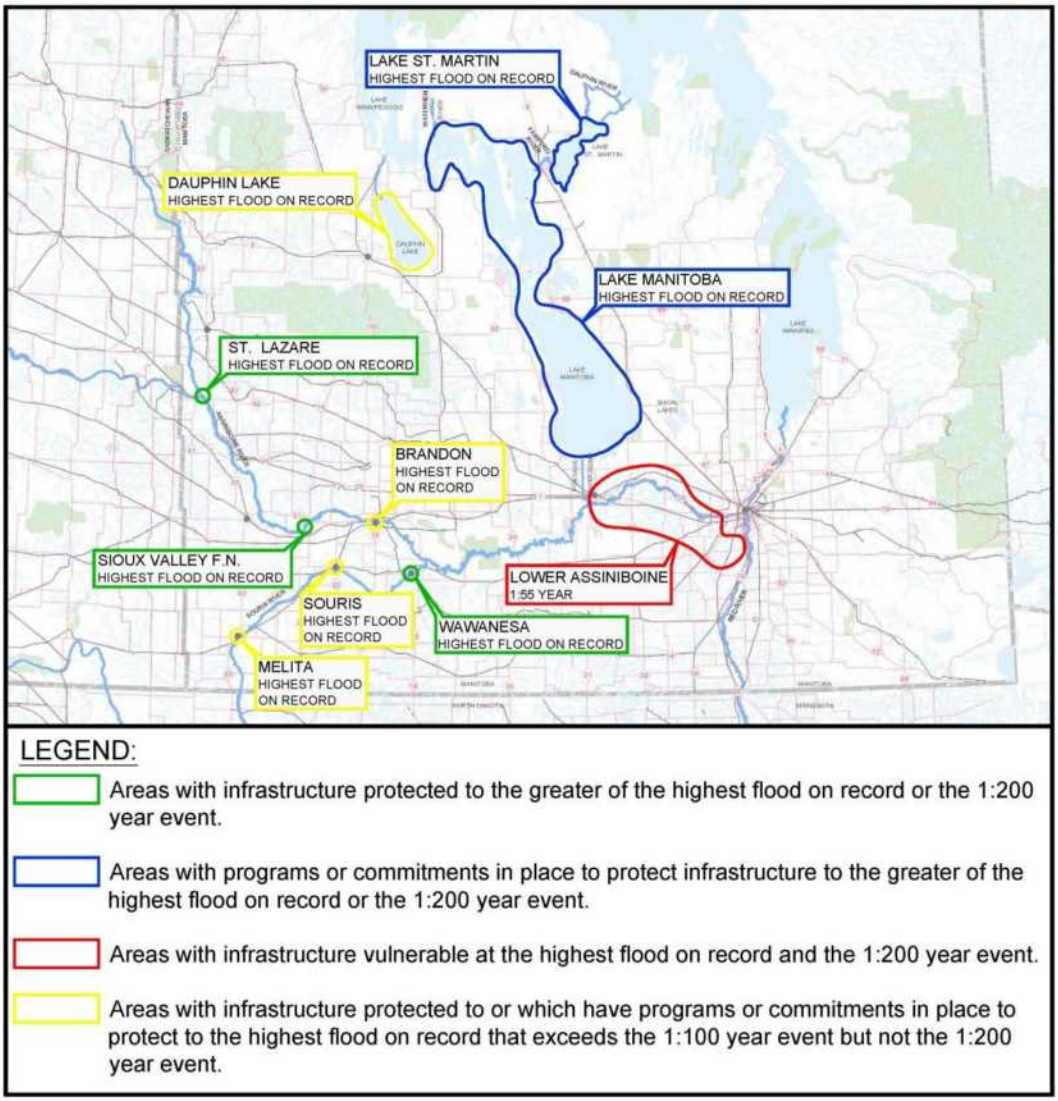
- A higher flood event that can be justified in an economic analysis that considers costs and benefits.

The current policy for flood protection was guided by the results of the 2011 Flood Review Task Force. In their report, the 2011 Flood Review Task Force concluded the following:

> *Even though provincial flood protection policy requires flood protection to the higher of the 100-year flood level or the flood of record, flood damages continue to occur in Manitoba. The standard response of water managers has been to re-compute the 100-year flood level based on the latest flood or to raise the FPL to the flood of record, as was done for Lake Manitoba after the 2011 Flood. These new levels determine the Province's flood protection requirements that municipalities must include in their planning documents, but this site-specific practice has resulted in uneven flood risk geographically and has created considerable confusion.*
>
> *It is the Task Force's opinion that the 100-year flood standard is too low when one considers risk over the longer term. With a 100-year flood, there is a one percent chance that it will occur in any one year, but in any one location the probability that the 100-year flood will be exceeded at least once in a 10-year period is 10 percent and the probability over a 50-year period is 40 percent. This longer term risk is considered too high and is generally not understood by Manitobans. It is suggested the 200-year flood may provide an appropriate level of long-term risk tolerance, given the economic damages resulting from floods, the economic costs of building to higher FPLs and the uncertainty with estimation of FPLs from limited hydrologic data.*

Feedback received at the technical workshop was generally in agreement with the current provincial policy, and concurred with the findings of the 2011 Flood Review Task Force. On this basis, the existing flood protection levels (or soon to be improved levels due to programs and commitments that are currently underway) for communities and individuals at key locations of the Assiniboine River & Lake Manitoba Basins were estimated. This identified areas that do not meet the target FPL defined by the provincial policy. These areas are shown on Figure 4.1.

Figure 4.1
Level of Protection at Key Locations of the Assiniboine River & Lake Manitoba Basins



As shown on Figure 4.1, the Lower Assiniboine River was identified as the highest priority area. It has infrastructure that would be vulnerable at flood magnitudes much less than both the largest flood on record and the 1:200 year event. Various options for flood protection, including standalone and combinations of individual projects have, therefore, been considered to increase the flood protection level in the Lower Assiniboine River area and are presented in this report.

Figure 4.1 also shows that there are several other areas, which do not meet the current provincial policy. Those areas have infrastructure protected to, or have programs or

commitments in place to protect for, the largest flood on record, which exceeds the 1:100 year event but not the 1:200 year event. Those areas include Dauphin Lake, Melita, Souris and Brandon. Various options have been considered to increase the flood protection level in those areas and are presented in this report.

In addition to the vulnerabilities shown on the map, many others were identified through this study. These are discussed in greater detail in the section that follows.

4.3 LIST OF VULNERABILITIES

A comprehensive list of all vulnerabilities and potential inadequacies of the existing flood protection system in the Assiniboine River & Lake Manitoba Basins was developed for this study. This was done in cooperation with the engineering community of southern Manitoba and various other technical people attending the Workshop, as described in Section 4.1. Table 4.1 contains the list, with vulnerabilities separated by geographic areas, along with the approximate flood magnitude indicating the threshold above which that vulnerability could lead to damage. Precise determination of the threshold conditions is difficult and fraught with uncertainties and issues that defy rigorous quantification. Consequently, a more qualitative approach was taken that separated the threshold conditions into three categories:

1. **H** – only a factor for large infrequent floods.
2. **M** – a factor for moderate floods or larger.
3. **L** – a factor for all floods, low to high.

The last column in the table provides a numerical rating developed largely by a consensus from the senior study team members at KGS Group, with input from contributions offered by the Workshop attendees. The rating ranges from a value of 1 (high level of concern) to 3 (low level of concern).

Table 4.1A
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
1. Overall Protection System (not necessarily associated with any one component of system)				
a. Emergency Preparedness and Response Plans	Well defined plans and general procedures to follow under extreme flood conditions, or during breach conditions, may not be available to the extent currently adopted for major water retaining structures (inadequacy may vary by jurisdiction). Plans for evacuation on a large scale should also be considered.	Possibility of unnecessary loss of lives and property because of inappropriate reaction resulting from inadequate opportunity to plan.	M	1.2
b. Unstable or degrading river banks, shoreline & dikes / seepage through the dikes	Risk of failure at stages below design level or levels below what was envisaged. Inadequate inspection and maintenance (including ownership issues - provincial vs private) can exacerbate the degradation.	Flooding of protected areas, as well as potential for loss of life.	M	1.5
c. Assessment with respect to current Dam Safety Guidelines	Safety of some water retaining structures (dikes, control structures, embankments) with respect to current design standards is unknown.	Possibly more risk of failure of water retaining structures than currently considered acceptable by experts in the field.	L	2.3
d. As-built drawings	As-built drawings of some components not available, or available ones contain incorrect information.	Uncertainty in assessing risks of failure or need to upgrade to current standards.	L	2.2
e. Flood monitoring and information gathering / interpretation / dissemination	Less than desirable level of flood monitoring & data handling is possible with current staff level and flood control plans.	Could miss early signs of problems that could lead to serious failures and could otherwise have been prevented.	M	1.5
f. Attrition of Experienced Staff Resources	Lack of comprehensive succession plan could jeopardize future responses to emergencies.	Less than optimum response to abnormally large floods.	M	1.5
Notes: 1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.				

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.1B
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
1. Overall Protection System (not necessarily associated with any one component of system)				
g. Uncertainties in flood forecasting	Response to floods requires accurate predictions of flood conditions that may develop. Inadequate meteorological monitoring & information on precipitation can hamper the ability to generate an accurate flood forecast.	Inappropriate actions taken: either too little effort to fight flood, or too much, causing unnecessary expenditures.	M	1.3
h. Flood Risk Maps and other GIS data outdated / unavailable	Flood risk maps, hi-resolution maps, topographic & bathymetric data... are out of date and are not in hands of stakeholders.	Insufficient knowledge for effective development / effective flood fighting.	M	1.3
i. Ability to plan and implement long term flood mitigation strategies in a timely manner	The planning and implementation of new flood mitigation works takes time and money.	Flood damages may occur before flood mitigation strategies can be implemented.	M	1.9
j. Development of transportation infrastructure can affect flood protection works	Roads / bridges can reduce hydraulic capacity.	Reduction in flood protection level, inaccuracies in flood forecasting.	M	2
k. Inadequate changes in land use & development controls	Building / farming / cottagers are getting closer to water within the flood plain along rivers and lakes.	Increased risk of flooding.	M	1.8
l. Optimum operation of flood control works	Many control works are operated based on rough guidelines and/or for local interest only. Actual operation during floods requires special consideration based on the unique circumstances at the time.	Greatest flood reduction benefits for entire system may not be achieved.	H	1.7
m. Ability to implement Emergency Preparedness and Response Plans	Operation of flood control works and decision process during an emergency depend on the availability of resources and the ability to assemble and deploy them quickly.	Slow response or shortage of resources can lead to less than optimum flood protection.	M	1.5

Note:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.1C
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
1. Overall Protection System (not necessarily associated with any one component of system)				
n. Access to communities, dikes and other flood control works limited	Access may be difficult due to poor maintenance, inadequate infrastructure, or flooding / damages to transportation network.	Reduced ability to react quickly or with machinery/equipment can increase risk of failure.	H	1.9
o. Increasing potential for floods	Wide fluctuations in extreme events such as floods may be exacerbated due to climate change / variability and land use / development.	Increasing potential for failure of existing works and the need to consider changes in design standards as time progresses.	H	2
p. Inadequate changes in drainage practices	Current agricultural practices and infrastructure expansions are constantly expanding the current drainage network.	Increased risk of flooding due to higher runoff locally and from upstream jurisdictions. Increased potential for siltation in rivers and reservoirs.	M	1.5
q. Failure of flood protection infrastructure	Failure of Flood Protection Infrastructure (Dams, Dikes, Control Structures, etc...) including structures in upstream jurisdiction, can lead to sudden surge in discharges.	Increased risk of flooding as well as potential for loss of life.	H	1.3
r. Inadequate interjursdictional coordination & communication	Lack of data sharing, coordination and communication between jurisdictions (municipal, provincial and international) can lead to less than optimum flood protection.	Greatest flood reduction benefits may not be achieved.	L	2
s. Imbalance in representation of Social Groups	Some social groups may not be as prepared for flooding as others (both permanent and temporary protection).	Increased risk of flooding due to potential inconsistencies in flood protection level.	L	1.9

Notes:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.
2. Communities such as Brandon and Portage la Prairie may have vulnerabilities similar to those defined for the City of Winnipeg. Further discussions with authorities in those jurisdictions would be required to be specific.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.2
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
2. Flood Protection Along Upper Assiniboine River - Shellmouth to Portage la Prairie, Manitoba				
a. Current land use in agricultural areas encroach on flood plain	Bankfull capacity frequently exceeded from Shellmouth to Grand valley.	Extensive crop losses due to inability to seed or due to drowning of crops.	M	1.8
b. Height of dikes in Brandon and at other upstream locations	Risk of overtopping - existing dikes and river banks have limited capacity to protect infrastructure.	Flooding of homes, businesses and other infrastructure in low-lying zones of Brandon.	H	1.5
c. Height of dikes in St. Lazare	Risk of overtopping - existing dikes and river banks have limited capacity to protect infrastructure.	Flooding of homes, businesses and other infrastructure.	H	1.8
d. Height of dikes in Sioux Valley First Nations	Risk of overtopping - existing dikes and river banks have limited capacity to protect infrastructure.	Flooding of homes, businesses and other infrastructure.	H	1.7
e. Height of dikes in Spruce Woods Provincial Park	Risk of overtopping - existing dikes and river banks have limited capacity to protect infrastructure.	Flooding of infrastructure.	H	2.0
f. Bridge restrictions & failures / damages	Capacity restriction at bridges can increase water levels upstream. Risk of pier failure or dislocation of deck or combination due to high water levels and velocities.	Reduced access due to road closures, infrastructure damages or loss of bridge. Failure of bridge can reduce capacity of crossing and cause an increase in water levels upstream.	M	1.9
g. Low lying roads in Assiniboine River Valley	Risk of overtopping - road closures / damages.	Limited access across the Assiniboine River Valley and to communities for flood fighting activities. Damage to infrastructure.	M	2.2
h. Ice effects in river	Backwater from ice jams may raise water levels above flood protection level.	Flooding of agricultural lands, communities, infrastructure...	L	1.9
i. Water management decisions in upstream jurisdictions	Inflow to Manitoba dependant on water management decisions and policies in upstream jurisdictions (land use / operation strategies for structures).	Greatest flood reduction benefit for Manitoba may not be achieved.	M	1.4

Note:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.3
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
3. Flood Protection Along Lower Assiniboine River - Portage la Prairie to Winnipeg				
a. Insufficient bankfull / dike capacity along river	Water levels may exceed flood protection level when river flows rise due to limits of discharge capacity in the Portage Diversion Channel.	Flooding of homes, infrastructure and agricultural interests in floodplain adjacent to Assiniboine River and along La Salle River.	M	1.2
b. Dikes are degrading, riverbed may be aggrading	Risks of dike failures are increasing either during floods or afterwards due to lowered protection as years advance. River rating curve is shifting and hampers accurate forecasting of flood levels.	Flooding of homes, infrastructure and agricultural interests in floodplain adjacent to Assiniboine River and along La Salle River, possibly due to sudden releases as dikes fail.	M	1.1
c. Ice effects in river	Backwater from ice jams may raise water levels above dike height.	Flooding of agricultural lands and dwellings.	L	1.6
d. Bridge restrictions & failures / damages	Capacity restriction at bridges can increase water levels upstream. Risk of pier failure or dislocation of deck or combination due to high water levels and velocities.	Reduced access due to road closures, infrastructure damages or loss of bridge. Failure of bridge can reduce capacity of crossing and cause an increase in water levels upstream.	M	1.9
e. Controlled release of flows into LaSalle River	Controlled release of flows into the LaSalle watershed (e.g. Hoop n' Holler) may be required when Portage Reservoir inflows greatly exceed flood protection works.	Flooding of homes, infrastructure and agricultural interests downstream of breach and along La Salle River.	H	1.6
f. Limited access to dikes	Access may be difficult due to poor maintenance & inadequate infrastructure.	Reduced ability to react quickly or with machinery/equipment can increase risk of failure.	H	1.3
g. Backwater effects on Long Lake Drain	High Assiniboine River water levels cause a backwater effect on Long Lake Drain.	Flooding of agricultural interest in floodplain near Assiniboine River.	H	2.8

Note:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.4
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
4. Flood Protection Along Souris River in Manitoba				
a. Current land use in agricultural areas encroach on flood plain	Water levels build up due to excessive runoff.	Productivity losses on agricultural lands in the floodplain. A few homes affected during large floods.	M	2.2
b. Dike height in Melita	Risk of overtopping.	Flooding of homes, businesses and infrastructure.	H	1.4
c. Dike height in Souris	Risk of overtopping.	Flooding of homes, businesses and infrastructure.	H	1.4
d. Dike height in Wawanesa	Risk of overtopping.	Flooding of homes, businesses and infrastructure.	H	1.4
e. Integrity at Wawanesa Dam and abutments	Erosion / undermining of slopes adjacent to dam could lead to failure	Dam breach and sudden release of water. Loss of water supply infrastructure.	H	1.4
f. Shifts in river water levels due to channel degradation	River rating curve is shifting and hampers accurate forecasting of flood levels.	Inaccurate flood forecasts, overstating predictions of flooding.	L	2.2
g. Bridge restrictions & failures / damages	Capacity restriction at bridges can increase water levels upstream. Risk of pier failure or dislocation of deck or combination due to high water levels and velocities.	Reduced access due to road closures, infrastructure damages or loss of bridge. Failure of bridge can reduce capacity of crossing and cause an increase in water levels upstream.	M	1.6
h. Low lying roads in river valley	Risk of overtopping - road closures / damages.	Limited access across the Souris River Valley and to communities for flood fighting activities. Damage to infrastructure.	H	1.8
i. Forecasting dependent on flood predictions from upstream jurisdictions	Actual Souris River inflows to Manitoba may differ from predictions in upstream jurisdictions.	Inaccurate flood forecasts can lead to inappropriate actions taken.	M	1.3
j. Upstream jurisdictions water management	Inflow to Manitoba dependant on upstream jurisdictions water management decisions and policies including land use and operation of water control infrastructure.	Greatest flood reduction benefit for Manitoba may not be achieved.	M	1.6

Note:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.5
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
5. Flood Protection around Lake Manitoba				
a. Natural Inflow can exceed outflow capacity to the extent that flood levels beyond normal are possible	Significant rises in water levels above the normal range.	Flooding of homes and seasonal residences, agriculture, related infrastructure.	H	1.7
b. High inflows due to operation of Portage Diversion	Inflows to Lake Manitoba can be exacerbated by diversion of flood flows through the Portage Diversion.	Higher flood levels on the lake than would have occurred without the Portage Diversion.	H	1.3
c. Lake Manitoba communities susceptible to high water levels	Risk of flooding / dike overtopping to Lake Manitoba communities, including cottagers and First Nations.	Flooding of homes, seasonal residences, businesses and related infrastructure.	H	1.4
d. Lake Manitoba rural shorelines susceptible to high water levels	Risk of flooding / dike overtopping.	Flooding of homes, businesses and agricultural interests around lake.	H	1.6
e. Lake susceptible to wind effects	Risk shoreline & dike erosion / overtopping, causing breach formation and flooding. Loss of vegetation due to extended periods of high water levels can increase the risk of erosion.	Flooding of homes, seasonal residences, businesses and related infrastructure.	H	1.7
f. Ice effects on Lake can be exacerbated during periods of high water levels	Risk of ice movement onto the shoreline.	Infrastructure damages due to ice. Shoreline & dike erosion can cause breach formation and flooding.	M	1.9
g. Limited winter outflows due to backwater and downstream flooding concerns	Lake cannot be drawn down after autumn freeze-up due to reduced channel capacities downstream. Fairford outflows often reduced to prevent serious ice jams on Fairford R. and Dauphin R.	Higher lake level at end of winter, aggravates flooding on Lake Manitoba shoreline during spring and summer.	M	1.3
h. Uncontrolled inflow from Waterhen River	Uncontrolled inflow from the Waterhen River can exacerbate flooding on Lake Manitoba.	Flooding of homes and seasonal residences, agriculture, related infrastructure.	H	2.3

Note:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.6
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
6. Flood Protection at Lake Pineimuta, Lake St.Martin, and along Fairford and Dauphin Rivers				
a. Lake St. Martin / Pineimuta communities susceptible to high lake levels	Flooding of residences, businesses and some agricultural interests; this has been exacerbated by the improvement in outflow capacity at Fairford Dam and the Fairford River.	Flood damages to residential buildings, infrastructure and agriculture, including prolonged evacuation of local residents.	M	1.1
b. High ground water in the Lake Pineimuta and St. Martin areas	High groundwater levels may be exacerbated by high surrounding lake and river levels and cause flooding before direct inundation from lakes/rivers.	Flooding of residences, businesses, and agricultural interests.	M	2.0
c. River bank heights in Dauphin River First Nations	Risk of overtopping.	Flooding of residential homes, businesses and related infrastructure, prolonged evacuation of residents may be necessary.	H	1.8
d. Low lying roads susceptible to high water levels	Risk of overtopping - road closures / damages.	Limited access to communities and flood protection infrastructure, including prolonged evacuation of local residents.	H	1.5
e. Ice effects in the Dauphin River increase water levels	Increased water levels in Dauphin River and Lake St. Martin in the winter season due to ice jamming.	Impacts on agricultural lands near river and on transportation due to closure of PR 513.	H	1.9
f. Uncertainties in future operation of Lake St. Martin Emergency Outlet Channel	Lake St. Martin Emergency Outlet Channel was constructed under emergency conditions and the impact of potential future operation of the channel is currently unclear.	Uncertainties in current flood protection level. Greatest flood reduction benefits may not be achieved. Environmental concerns, particularly fisheries.	M	1.1
g. Lake susceptible to wind effects	Risk shoreline & dike erosion / overtopping, causing breach formation and flooding. Loss of vegetation due to extended periods of high water levels can increase the risk of erosion.	Flooding of homes, seasonal residences, businesses and related infrastructure.	M	2.0
h. Inadequate land drainage around dikes	Inadequate land drainage can lead to flooding from excessive runoff.	Flooding of residences, businesses, and agricultural interests.	M	1.5

Note:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.7
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
7. Flood Protection at Dauphin Lake & Mossy River				
a. Inflow can exceed outflow to the extent that lake levels rise above flood threshold levels	Excessive inflows that cannot be released readily from Dauphin Lake cause high water levels and lakeshore flooding.	Flooding of homes and seasonal residences, businesses and agricultural interests around lake.	H	2.3
b. Lake susceptible to wind effects	Risk of shoreline & dike erosion / overtopping, causing breach formation and flooding. Loss of vegetation due to extended periods of high water levels can increase the risk of erosion.	Flooding of homes and seasonal residences, businesses and agricultural interests around lake.	M	2.3
c. Limited channel capacity of Mossy River	Overbank flows occur during high lake outflows. Outflow capacity at dam is limited by backwater from small channel.	Flooding of agricultural lands near the Mossy River. Higher lake levels due to reduced outflows.	H	2.3
Note: 1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.				

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.8
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
8. Flood Protection at Lake Winnipegosis & Waterhen River				
a. Inflow can exceed limited outflow capacity to the extent that flooding of lakeshores can occur	Excessive inflows that cannot be released quickly from Lake Winnipegosis cause high water levels and lakeshore flooding.	Lake Winnipegosis susceptible to large water level fluctuations. Flooding of homes and seasonal residences, businesses and agricultural interests around lake and communities.	H	2.4
b. Ice effects in Waterhen River increases water levels along the river	Increased water levels in Waterhen River in the winter season due to ice jamming.	Potential for flooding of homes, businesses increased along Waterhen River.	H	2.1
c. Ice effects in Waterhen River reduces outflow capacity from Lake Winnipegosis	Lake Winnipegosis outflow decreases in winter due to ice effects.	Prolonged high water levels in Lake Winnipegosis with increased risk of flooding; exacerbates spring floods into Lake Manitoba when ice clears.	H	2.6
d. Bridge restrictions & failures / damages	Capacity restriction at bridges can increase water levels upstream. Risk of pier failure or dislocation of deck or combination due to high water levels, velocities, ice effects in Waterhen River.	Reduced access due to road closures, infrastructure damages or loss of bridge. Failure of bridge can reduce capacity of crossing and cause an increase in water levels upstream.	H	2.8
Note: 1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.				

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.9
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
9. Flood Protection at Shoal Lakes				
a. Inflow and rainfall (minus evaporation) can produce high lake levels	Lake has no outlet except at record high levels, so levels gradually rise and lakes become much larger during extended wet spells.	Extensive inundation of agricultural lands and roads near the lake.	H	2.0
b. Low lying roads near lakes	Risk of overtopping.	Road closures & damages to infrastructure affects access to communities and flood protection infrastructure.	H	2.0
Note: 1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.				

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.10A
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
10. Flood Protection Infrastructure Within City of Winnipeg				
a. Internal drainage system - capacity	Water level backup due to high river levels combined with high local runoff.	Flooding of basements/surface flooding.	H	1.9
b. Sewer / drainage system - manhole sealing	High river levels trigger the need to seal manholes across the City.	Flooding of basements/surface flooding.	H	2.3
c. Sewer system control	High river levels trigger sewer system control procedures to protect infrastructure (i.e. gate closures, temporary pumping).	Flooding of basements/surface flooding.	H	2.3
d. External agencies	Multiple property owners such as apartment complexes are required to protect their buildings on their own.	Flooding of property owner's infrastructure. Potential vulnerability to City if adequate flood protection not implemented by property owner.	H	2.4
e. Residential Isolation	High river levels trigger the requirement to isolate property owners from rest of sewer system to protect City of Winnipeg in case of secondary dike failure.	Flooding in Winnipeg.	H	2.1
f. Flood pump station failure	Water level backup due to high river levels combined with high local runoff.	Flooding of basements/surface flooding.	L	1.3
g. Ice effects	Ice jamming at bridges.	Backwater effects have potential to cause overtopping of dikes.	M	1.6
h. Power supply / backup systems	Outage of critical pumps that maintain low water levels within Winnipeg.	1.Flooding of basements. 2.Eventual surface flooding.	M	2.0
i. Primary dikes - perceived ability to top up	1. Topping up primary & secondary dikes may be impractical in time frame available. Prioritization may be required. 2. Topping up dikes may cause significant traffic disruption.	Not as much protection against severe floods as currently contemplated.	H	1.6
j. Primary Dikes - internal failure	Variety of possible failure modes all leading to breach	Flooding in Winnipeg.	H	1.7

Note:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.10B
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
10. Flood Protection Infrastructure Within City of Winnipeg (Continued)				
k. Primary Dikes - overtopping	Dike breach formation.	Flooding in Winnipeg; this can be accentuated by simultaneous occurrence of flood peak inflows from the Red River and the Assiniboine River, but only when the capacity of the Portage Diversion is exceeded and more flow must be directed towards Winnipeg. Otherwise, simultaneous peaks lower the river level in Winnipeg.	H	1.4
l. Secondary Dikes - internal failure	Variety of possible failure modes, all leading to breach.	1. Flooding of areas protected by secondary dikes. 2. Limited capability to prevent backup of sewer system into areas outside of secondary dikes.	H	1.7
m. Secondary Dikes - overtopping	Breach formation.	1. Flooding of areas protected by secondary dikes. 2. Limited capability to prevent backup of sewer system into areas outside of secondary dikes.	H	1.7
n. Limited capacity to evacuate	Evacuation on a large scale has not been planned and execution would be fraught with difficulties without prior planning.	Ineffective evacuation, possibly risk to lives.	H	1.6
Note: 1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.				

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.11A
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
11. Portage Diversion, Portage Reservoir and Control Structures				
a. Ice effects - in reservoir	1. Dike overtopping/breach formation. 2. Release of uncontrolled excessive flow down Lower Assiniboine River. Ice control can be reduced as sediment deposits increase in the reservoir.	Increased flood flows/levels in Lower Assiniboine and in Winnipeg.	M	1.4
b. Ice effects - in river downstream	Release of uncontrolled excessive flow into Winnipeg.	Increased flood flows/levels in Lower Assiniboine and in Winnipeg.	M	1.4
c. Dike overtopping/breach formation	Release of reservoir storage into Assiniboine River/Winnipeg.	Increased flood flows/levels in Lower Assiniboine and in Winnipeg.	H	1.3
d. Downstream erosion at River Control Structure	Could lead to failure of structure/breach formation; potential erosion at Southport Bridge piers.	Increased flood flows/levels in Lower Assiniboine and in Winnipeg; potential failure of Southport Bridge.	H	1.9
e. Reduced tailwater levels	Could lead to failure of structure/breach formation.	Increased flood flows/levels in Lower Assiniboine and in Winnipeg.	H	2.1
f. Gate failure (either or both structures) : -hoist failure -control mechanism failure -gate structure failure	Possibly: 1. Overtopping of dykes, or 2. Release of excessive flows into Winnipeg.	Increased flood flows/levels in Lower Assiniboine and in Winnipeg.	H	1.3
g. Design Capacity exceeded in emergency conditions	Insufficient discharge capacity leads to exceeding design capability under emergency conditions.	Increased Lake Manitoba inflow. Increased risk of dike failure along channel.	H	1.0
h. Low discharge capacity at "failsafe" section	Original design of west dike along channel includes a "failsafe" section, which when overtopped, causes flooding of lands to the west.	Flooding after failure of failsafe impacts local agricultural interests and environment (wetlands).	M	1.5
Note: 1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.				

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.11B
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
11. Portage Diversion, Portage Reservoir and Control Structures				
i. Bridge failures / damages	Risk of pier failure or dislocation of deck or combination due to high water levels and velocities.	Reduced access due to road closures, infrastructure damages or loss of bridge. Failure of bridge can reduce capacity of Portage Diversion Channel.	H	1.9
j. Failure / damages to drop structures	Erosion due to high discharges and velocities could cause damages or lead to failure of the structure	Infrastructure damages. Failure could cause sudden release of excess water downstream.	H	1.4
k. FSL of Reservoir above control structure mechanical room.	Risk of flooding the mechanical room	Potential inability to operate control structure.	M	1.0
l. Dike stability & seepage	Unstable dikes or seepage can lead to the failure of the flood protection works at levels lower than had been envisaged.	Flooding of homes, agricultural interests, and other infrastructure downstream of breach. Failure to the east could cause uncontrolled overland flows along Lower Assiniboine River.	H	1.0
m. Mechanical / electrical failure	Inability to operate control structure due to critical failure of mechanical / electrical system.	Increased flood flows/levels in Lower Assiniboine and in Winnipeg.	L	1.1
Note: 1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.				

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

Table 4.12
Summary of Vulnerabilities and Potential Inadequacies under Flood Conditions

Vulnerability / Potential Inadequacy	Description	Consequences of Failure or of Inadequacy	"Concern Range" of River Flows	Rating
Column 1	*Column 2*	*Column 3*	*Column 4*	*Column 5*
12. Shellmouth Dam				
a. Gate failure	Variety of possible modes.	Uncontrolled release from low level outlet.	M	1.6
b. Dam breach	Piping, overtopping (inflow exceeds Inflow Design Flood) leading to breach formation.	Uncontrolled release of Lake of the Prairies into Assiniboine River and Winnipeg.	H	1.6
c. Downstream erosion	Structural failure of spillway/breach formation	Uncontrolled release of Lake of the Prairies into Assiniboine River and Winnipeg.	H	1.9
d. Insufficient reservoir capacity to permit control of flood inflows	Attenuation of flood inflows is insufficient to prevent frequent flooding of valley bottom, particularly in the first 15 km reach below the dam.	Flood damages to agricultural interests can be caused or accentuated by delayed and extended high flows after a flood event.	H	1.4
e. Sediment deposition in reservoir has not been monitored	Reduction in flood storage capacity is unknown.	Uncertainties in flood forecasting and water level control; potential for degradation of flood protection offered by the dam.	M	2.1
f. Mechanical / electrical failure	Inability to operate conduits due to critical failure of mechanical / electrical system.	Increased flows/levels in Assiniboine River downstream of dam.	M	1.6

Note:
1. Listed in Column 1 are vulnerabilities / potential inadequacies to the flood protection system and not of infrastructure vulnerable to flood damages nor consequences due to failure of flood protection system.

Legend	
Concern Ranges – Column 4	***Rating Structure – Column 5***
H – Only a factor for large floods	Rating (1. High, 2. Medium, 3. Low)
M – A factor for moderate or larger floods	Indicates level of concern and urgency
L – A factor for all floods	of investigation and resolution

4.4 DISCUSSION

The purpose of developing the comprehensive list of vulnerabilities in Table 4.1 is primarily to recognize the variety of problems that exist. Many of the issues raised in Table 4.1 must be addressed by the Province separately from the engineering studies that are the focus of the study reported herein. Those needing separate treatment by the Province are related, for example to:

- Development of documents that will facilitate emergency measures being carried out during flood events.
- Regulatory issues such as zoning and development controls.
- Optimization of operation of existing flood protection works.
- The reality of, and potential effects of, future climate change.
- Inter-jurisdictional issues.
- Coordination of water management decisions with upstream authorities.

Most vulnerabilities / inadequacies are self-evident and can be understood from the descriptions given in Table 4.1. Of all the vulnerabilities / inadequacies identified in Table 4.1, the three issues outlined below have, as viewed by the study team, the highest priority of concern and urgency of resolution.

1. Design Capacity of Portage Diversion and Lower Assiniboine River Exceeded in Emergency Conditions

This issue received the highest priority of all the vulnerabilities described in Table 4.1. The reviewers were unanimous in their view that this had the highest possible rating.

The Portage Diversion was designed to pass 700 m^3/s (25,000 cfs), with an equal amount released simultaneously into the Lower Assiniboine River. The diversion in 2011 exceeded this amount by 30%, and the release into the Lower Assiniboine River was limited to much less than 700 m^3/s (25,000 cfs). It is clear that improvements in both of these capabilities are urgently needed. This vulnerability has been addressed in considerable detail in the report that follows.

In addition, there are vulnerabilities of the existing facilities that threaten even the safe passage of the existing design magnitudes of flood. These include the stabilities of dikes at the Portage Diversion and along the Assiniboine River, inundation of the mechanical room floor at the River Control Structure of the Portage Diversion, and the potential for gate failure in either the River Control Structure or the Intake Control Structure of the Portage Diversion. These require urgent attention and upgrading. Detailed engineering evaluations are required to define the best means to carry out these upgrades.

2. Flood Potential at Lakes Pineimuta and St. Martin and Downstream Waterways

The concern for flooding in this zone is very high. This has been a chronic problem for decades and requires urgent attention. The means to alleviate this issue have been addressed in the study and are summarized in Section 15.0.

3. Emergency Preparedness and Response Plans

The response to the 2011 Flood was outstanding. A comprehensive review by the 2011 Flood Review Task Force (MFRTF, 2013) showed that major problems were effectively addressed and decisions made in a timely and purposeful fashion. However, the majority of individuals involved in directing the flood fight would almost certainly admit that many decisions were made under duress and were based on less than full knowledge of optimum reactions. There is no doubt that many of the plans and responses to emergencies must be made “on the fly” due to the unpredictable nature of the situation. There are others, however, that would benefit greatly from orderly engineering analysis, resolution, and preparation of clear plans beforehand.

This theme has been recognized in the Dam Safety field, and has led in recent decades to the requirement in many jurisdictions (although not yet in Manitoba) of detailed Emergency Preparedness and Response Plans (EPRP). These are routinely prepared for dams that could cause damage to life and/or property if failures were to occur and inundation would result. These EPRPs lay out details of the best responses to all foreseeable failure modes. They include notification charts, inundation maps to guide emergency evacuations, and other pertinent information that could improve the response to the emergencies that could develop.

Although the classic EPRPs for dams are not ideal templates for the reaction to the flood emergencies that could face the Assiniboine River or Lake Manitoba, they do demonstrate the underlying value of pre-planning emergency actions.

This value of pre-planning would be particularly true for the unprecedented flood magnitudes that are capable of occurring. They may demand many more serious and cost-incurring responses than were required in 2011, such as:

- Mass evacuations of potentially affected areas, including Winnipeg.
- Emergency response to breaches in or overtopping of flood retaining structures.
- Operation of the flood control works in unprecedented conditions which would require engineering decisions that would be difficult to make without methodical pre-planning.

The Province's vulnerability to the present lack of documentation and plans to manage extreme floods is viewed to be significant and requires attention. Similarly, documentation of well thought out plans to manage in the moderate category (i.e., up to 2011 Flood magnitude) is also important. It may be that such an extreme flood might not occur in southern Manitoba for many years. By that time, many of the engineers/planners/officials who currently have the first-hand knowledge provided by direct experience will no longer be available. Clearly, documented plans prepared by today's experienced staff would be a real asset for future staff unfamiliar and inexperienced with the magnitude of the risks that could arise.

Key components of such documentation should include, but not necessarily be limited to:

- Lines of authority within the provincial government.
- The primary authority responsible for the overall execution of the flood fight.
- Identification of the coordinating group(s) that would serve under the direction of the primary authority.
- Plans for upgrading, maintaining, preparing, and operating / controlling the flood fighting system under all foreseeable conditions, including those that significantly exceed the severe flood of 2011.
- Schedules and steps for raising dike crest elevations if required.
- Plans for patrol services to monitor key water retaining structures and nearby utilities such as transmission lines and bridges.

- Plans for the sequence of the closure of bridges that could be threatened by high water levels.
- Notification charts for key organizations such as Federal, Provincial, and Municipal Governments, Manitoba Hydro, CPR, CNR, MTS and qualified consulting engineering firms and individual consultants, with plans in place for liaison.
- Plans for widespread dissemination of flood fighting information to the public.
- Consideration should be given to the establishment of a Review Board who would be responsible for an audit of the preparedness for fighting floods up to an extreme magnitude. Inspections of facilities and review / updating of plans would be required as part of this audit on a regular basis, possibly every five to seven years. The auditors should ideally be known and respected members of the communities. Some, but not all, should be qualified professional engineers with extensive experience in flood preparedness and fighting.
- Detailed plans for evacuation should be in place.
- Means to combat typical problems such as breach formations in dikes, failure of flood pump stations, electrical outages at critical locations where flood protection works depend on continuous power supplies, and excessive rainfall that would overtax the land drainage system.
- Monitoring plans should be well developed, including the needs, and ways and means to obtain critical water levels, river flows, precipitation amounts, and other critical meteorological data and forecasts.
- Execution of tests of various parts of the plans should be scheduled, particularly if several years have passed since a major flood event.

5.0 HYDROLOGY OF THE ASSINIBOINE RIVER & LAKE MANITOBA BASINS

5.1 GENERAL

Together, the drainage basin of the Assiniboine River and Lake Manitoba cover an area of approximately 246,000 km^2 (95,000 mi^2), with over half of the area located outside of Manitoba, either in Saskatchewan or North Dakota. The drainage basin of Lake Manitoba is approximately 82,800 km^2 (32,000 mi^2) in size and includes Lake Winnipegosis, Dauphin Lake, Lake Manitoba and Lake St. Martin. Lake Manitoba releases outflow into Lake Winnipeg through the Fairford and Dauphin Rivers.

The drainage basin of the Assiniboine River is approximately 163,200 km^2 (63,000 mi^2) in size and includes the Assiniboine River, Souris River and Qu'Appelle River watersheds. The Assiniboine River meets the Red River in the City of Winnipeg. However, during large flood events a portion of the Assiniboine River flows is diverted into Lake Manitoba through the Portage Diversion. A map of the Assiniboine River & Lake Manitoba Basins is provided on Drawing 5.1 (located at the end of Section 5.0).

For the purpose of this study, the drainage basins of the Assiniboine River and Lake Manitoba were subdivided into the following six areas, and are shown on Drawing 5.2 (located at the end of Section 5.0):

1. Upper Assiniboine River.
2. Souris River.
3. Lower Assiniboine River.
4. Dauphin Lake.
5. Lake Winnipegosis.
6. Lake Manitoba and Lake St. Martin.

The hydrologic analyses conducted for the current study varied by area depending on the data available and the flood mitigation alternatives investigated. Sections 5.2 to 5.4 provide the details of the major hydrologic tasks conducted for the study. Further analyses were conducted

on an area by area basis throughout the study and have been summarized where applicable in various sections of this report.

5.2 FLOOD FREQUENCIES

Statistical analyses of flood frequencies at key locations along the Assiniboine and Souris Rivers, as well as on other tributaries of the Assiniboine and Lake Manitoba Basins, were provided by MIT. The results are attached in Appendix C. The probability distributions of flood peaks were generated based on data that was available for the entire period of record at each site and included the 2011 Flood peak. Where available, historic flood events such as the 1882 Flood on the Assiniboine River were also included in the analyses. Discussion of the effects of selecting the entire period of record for analysis, as compared to a shorter period, is provided in Section 5.2.1.

Since the flood frequency analyses were conducted by MIT at the beginning of the project reported herein, the probability distributions of flood events, also known as “flood frequency curves” do not include the 2014 Flood peak. The exception to this is for the Assiniboine River at Brandon. At this location, the flood statistics include the 2014 Flood so as to determine its impact on the analyses of the alternatives for flood protection at the City of Brandon. Further discussion on the impacts of the 2014 Flood event in Brandon and on the overall flood frequencies used in this study is provided in Section 5.2.2.

A summary of flood frequencies and peak flood flows at key locations of the study area is provided in Table 5.1. Computed flood peaks that have an annual probability of being equalled or exceeded of 1:100 and 1:200, as well as information on the largest flood peak on record at those locations, are also provided on the table. Return periods are the inverse of the annual probability (i.e., 1 / (annual probability)) and represent, on average, the number of years that would occur between such flood events. Return periods are of course abstract measures that are only presented as a convenient basis to compare flood probabilities.

Table 5.1
Summary of Flood Frequencies at Key Locations in the Assiniboine River & Lake Manitoba Basins

River	Location	1:100 cfs (m^3/s)	1:200 cfs (m^3/s)	Peak 2011 Flood Discharge	
				Return Period	Discharge cfs (m^3/s)
Assiniboine River	near Russell	15,400 (436)	21,700 (614)	92	14,670 (415)
	at St. Lazar	18,900 (535)	23,200 (657)	206	23,300 (660)
	at Brandon	34,200 (968)	42,300 (1,198)	140	38,000 (1,076)
	near Holland	47,500 (1,345)	57,100 (1,617)	140	52,000 (1,473)
Qu'Appelle River	near Welby	11,800 (335)	15,000 (425)	140	13,600 (385)
Souris River	at Melita	20,900 (593)	27,900 (790)	180	27,200 (770)
	near Souris	24,500 (695)	32,200 (913)	150	28,300 (801)
	near Wawanesa	25,200 (714)	32,200 (911)	140	28,300 (801)
Mossy River	below Dauphin Lake	3,400 (95)	3,600 (103)	100	3,400 (95)
Waterhen River	near Waterhen	15,400 (437)	17,300 (489)	120	15,900 (451)

Note: Frequency curves are based on the entire period of record and exclude the 2014 Flood, except for the Assiniboine River at Brandon.

In addition to the flood frequency analyses provided by MIT, frequency analyses of flood volumes were conducted for the Souris River at Wawanesa and the Assiniboine River near Brandon. The data for the frequency analysis was obtained from Water Survey of Canada (WSC) Gauge No. 05NG001 for the Souris River and Gauges No. 05MH001 and 05MH013 for the Assiniboine River. The following steps were taken to conduct the frequency analysis:

1. Identified the hydrograph associated with the peak discharge event on a yearly basis for the entire period of record at the three locations.

2. Isolated each hydrograph by identifying the start and end date of the runoff event. The start date was defined by the moment when there was a sudden rise in recorded discharges. The end date was defined by the moment when recorded discharges returned to antecedent conditions (i.e., conditions prior to the start of the runoff event).

3. Calculated the total runoff volume for each hydrograph. In some instances such as for the 2011 event, the calculated volumes include runoff from multiple hydrological events since runoff from the first event was not entirely completed before the start of the next event(s).

4. Conducted the frequency analysis for the period of record at the three locations using the US Army Corp of Engineers HEC-FFA software.

The results of the two frequency analyses are provided in Appendix C. Table 5.2 summarises the computed volumes for the 100- and 200-year flood events at the two locations. The table also shows volumes for the Assiniboine River at Holland, taken from the 2011 Flood Review Task Force Report (MFRTF, 2013). The frequency curve at Holland is also included in Appendix C.

Table 5.2
Summary of Flood Volume Frequencies at Key Locations in the Assiniboine River & Lake Manitoba Basins

River	1:100 (dam^3)	1:200 (dam^3)
Assiniboine River at Brandon	4,280,000	5,200,000
Assiniboine River near Holland	8,900,000	10,500,000
Souris River near Wawanesa	3,500,000	4,400,000

Lake Manitoba, Lake St. Martin and Dauphin Lake are all regulated, which makes it difficult to compute accurately the frequencies of water levels. Furthermore, when combining the effects of wind on lake levels, frequencies of water levels can vary by location around the lakes for the same return period event.

For Lake Manitoba, a numerical model using the principles of Monte Carlo Analysis was developed to address these issues and to determine frequencies of water levels. This numerical model is discussed in detail in Section 8.5.1 of this report. Information on the largest flood on record and the 1:200 year flood for Lake Manitoba is summarized in Table 5.3. On Dauphin Lake, a combination of frequency analyses were conducted by MIT; their results for the largest flood on record and the 1:200 year flood are also summarized in Table 5.3.

Table 5.3
Summary of Frequencies for Wind Eliminated Water Levels on the Lakes

Location	1:200 Water Level	Highest Flood on Record		
		Year	Return Period	Peak Wind Eliminated Water Level
Dauphin Lake	262.6 m (861.6 ft)	2011	100	262.4 m (860.8 ft)
Lake Manitoba	248.9 m (816.6ft)	2011	310	249.1 m (817.1 ft)

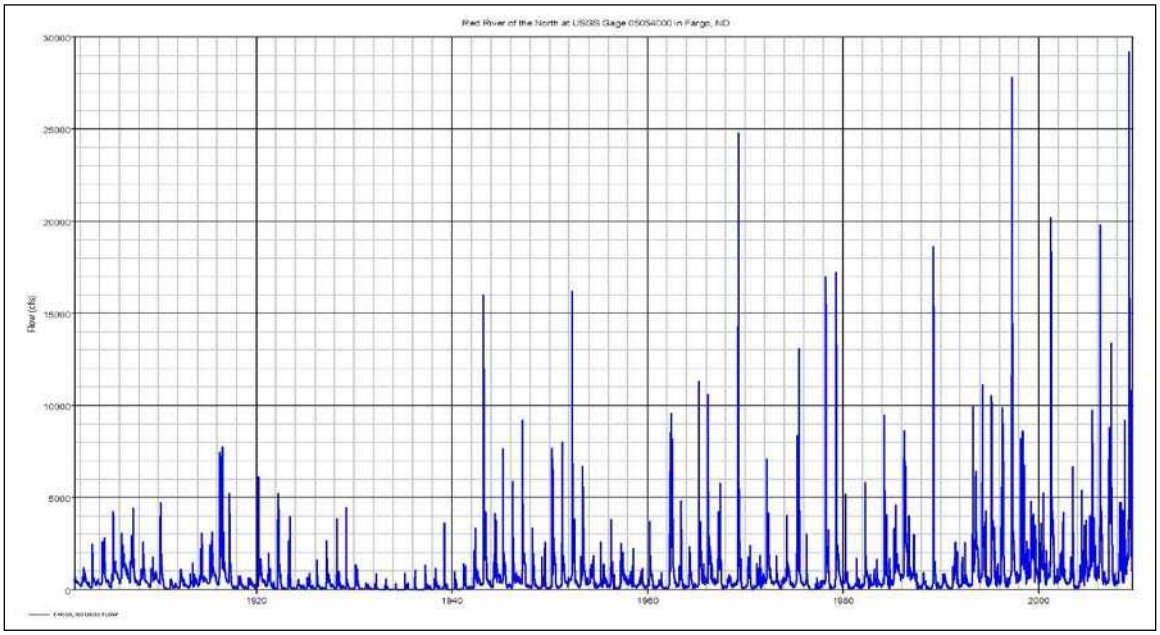
5.2.1 Period of Record

Based on feedback received during Session 4 of the Technical Workshop, as discussed in Section 4.1, it was decided that the entire period of record, including historic events where data was available, would form the basis of the frequency analyses conducted for the study.

It is well known that some studies and planning of flood protection works in other jurisdictions outside of Manitoba have made conscious decisions to focus the flood damage statistics on recent decades of records rather than the entire period of record. A good example of a major project that is currently being justified on the basis of primarily the most recent portion of the flood flow records is the Fargo Moorhead Flood Diversion Project (USACE, 2011) located in the upper catchment of the Red River. Figure 5.1 shows the trend in annual flood peak flows at Fargo since the start of formal records.

The clear increase in flows after about 1945 has been interpreted by the US Army Corps of Engineers (USACE) to be a trend that will continue. The USACE does not speculate on the specific causes; however, it is clear the causes include combinations of a variety of processes such as climate change, climate variability, land use changes and construction of extensive drainage works. In the face of this apparent obvious upswing in flood potential, the statistics used by the USACE for the studies of the Fargo Project have been weighted towards the flood records since 1942.

Figure 5.1
Flood Peaks in Red River at Fargo since 1990
Showing Noticeable Upward Trend in Flood Peaks



In spite of the actions taken in other jurisdictions, it is clear that the local engineering community in Manitoba is not in full acceptance of the principle of using foreshortened flood records. Many credible authorities maintain that the current trend in high flood potential is predominantly part of the normal cyclical pattern of flood events. This diversity of opinion was attested by the attendees at the Technical Workshop. Therefore, and as directed by MIT, the primary basis for the estimation of the Expected Annual Damages (EAD) was the full extent of flood records available for a site or region where a flood protection option was being considered. These were variable in length depending on the data available in each area of the study.

The effects of using shorter, more recent records of floods reflecting the recent few decades of wet conditions as compared to the entire period of record has been computed at two locations. The results of the comparison are shown on Figures 5.2 and 5.3, and have been incorporated into the sensitivity analyses of three flood mitigation measures, as discussed in Section 22.2.

Figure 5.2
Comparison of Frequency Curves for the Assiniboine River at Holland

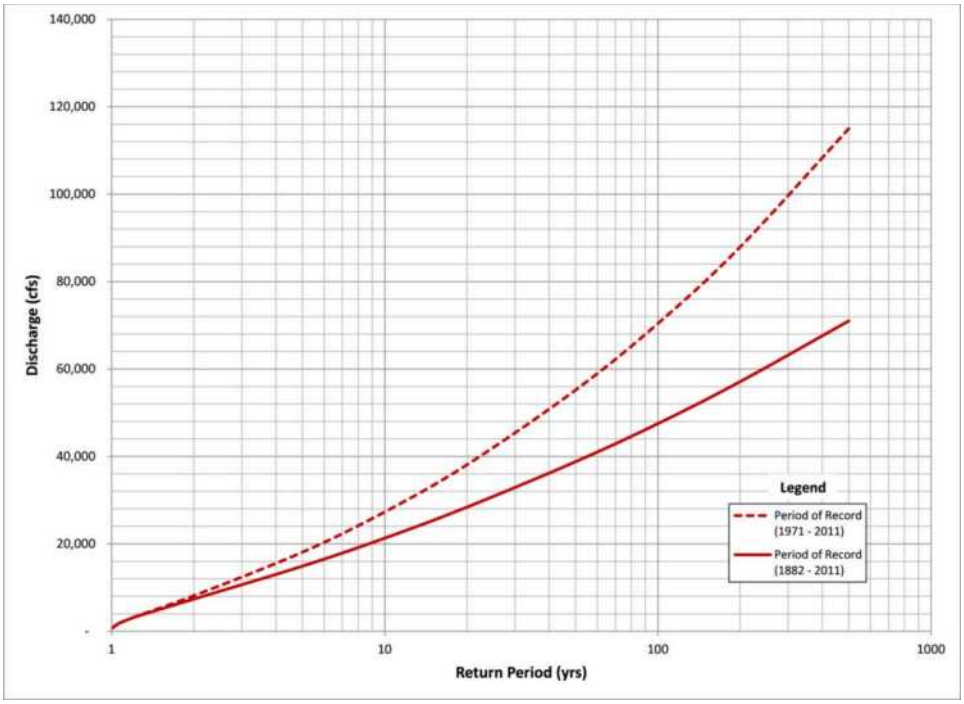
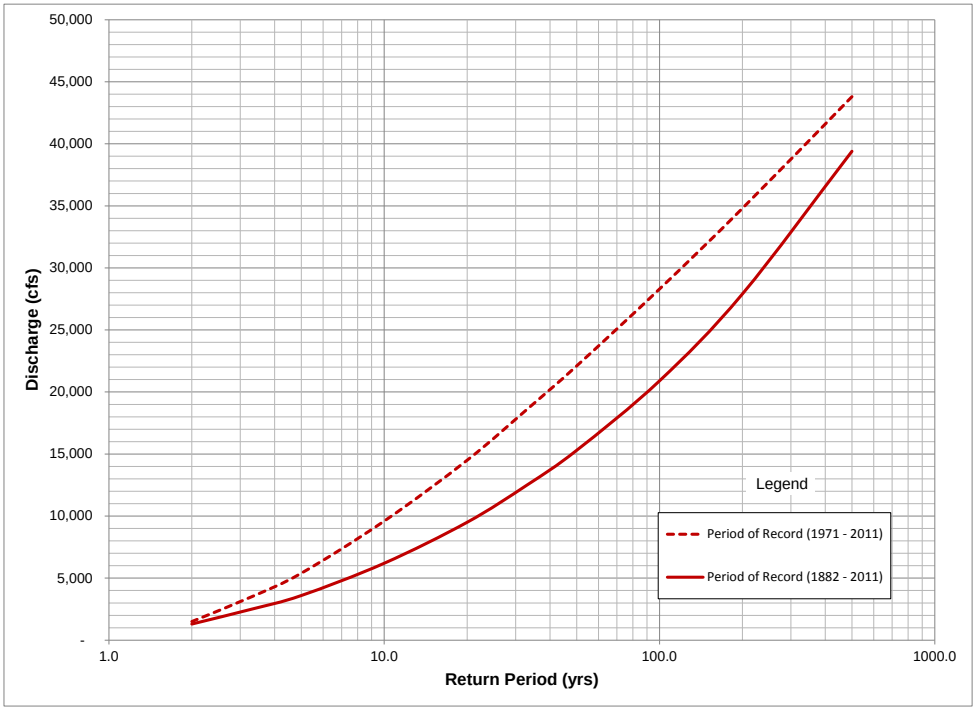


Figure 5.3
Comparison of Frequency Curves for the Souris River at Melita



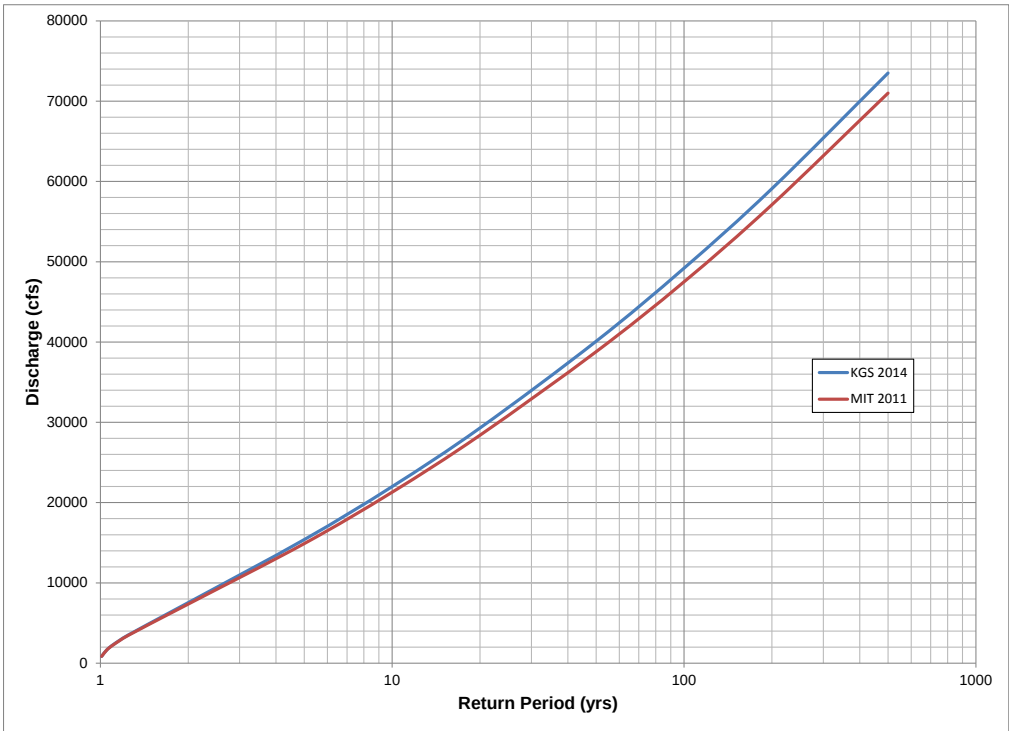
5.2.2 Effects of the 2014 Flood

Heavy rains in the summer of 2014 resulted in yet another significant flood event within the Assiniboine River & Lake Manitoba Basins, with record high flows on many tributaries. On the Assiniboine River, peak flows were near, or exceeded, those of the 2011 Flood event.

Since hydrologic analyses were conducted at the beginning of the study, frequency analyses provided by MIT did not include the 2014 Flood event. Most of the analyses of flood mitigation alternatives were also already completed at the time of the 2014 Flood event and, therefore, did not include data from the 2014 Flood.

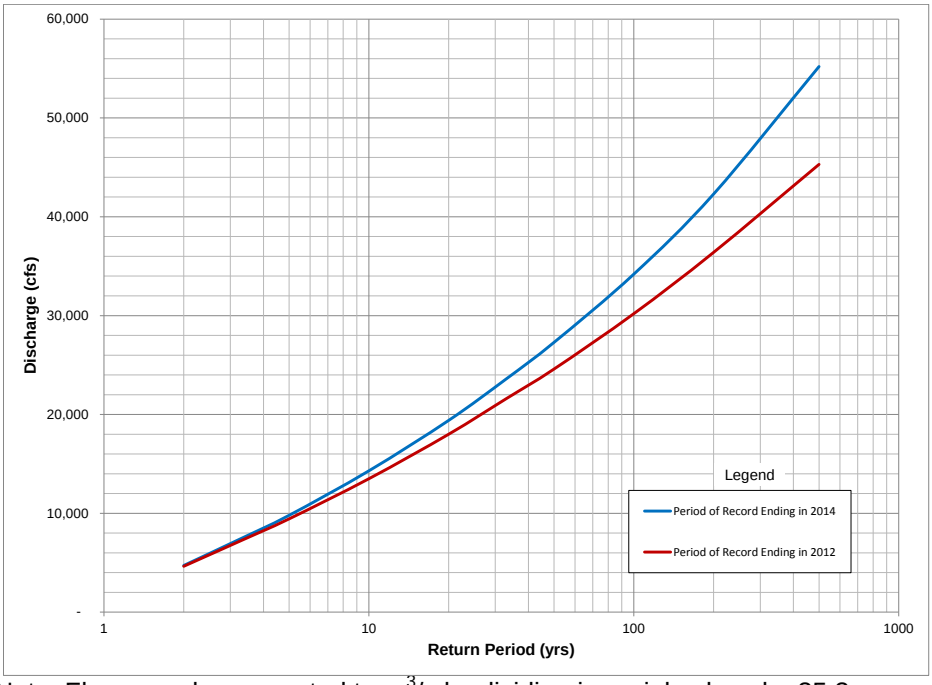
The effects of the 2014 Flood on the frequency curves adopted for the study vary by location. Comparisons at three locations are shown on Figures 5.4, 5.5 and 5.6.

Figure 5.4
Comparison of Frequency Curves for the Assiniboine River at Holland



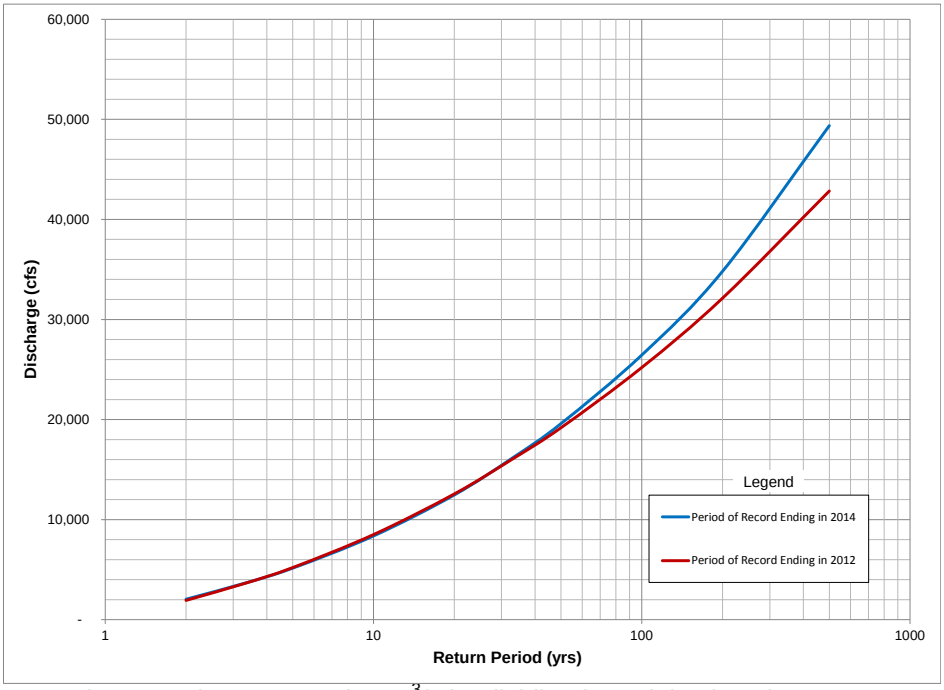
Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

Figure 5.5
Comparison of Frequency Curves for the Assiniboine River at Brandon



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

Figure 5.6
Comparison of Frequency Curves for the Souris River near Wawanesa



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

As shown on Figure 5.5, the 2014 Flood event caused a notable increase in the flows for a given flood frequency event on the Assiniboine River at Brandon. On this basis, the analyses for the City of Brandon were updated to include the 2014 Flood event. This is further discussed in Section 9.6 of this report.

In other locations, the effects of including the 2014 Flood did not appear to be as significant. Nevertheless, a qualitative review of the effects of the 2014 Flood on the overall conclusions for each flood mitigation alternative has been included in this report where applicable.

5.2.3 Effects of Climate Change

The 2011 Flood Review Task Force Report (MFRTF, 2013) concluded that the 2011 Flood was a very extreme and rare event and prevention of flood damages would not have been possible in all locations. In addition, scientific evidence is increasingly suggesting that global warming and climate change could lead to greater variability in the weather and possibly more extreme drought and flood events in the future.

Furthermore, they recommended the Province examine and adopt the means to accommodate any impact on hydrology of global warming and climate change when planning, designing and implementing future water use projects.

As noted in the 2011 Flood Review Task Force Report, the Hydrologic Forecasting Branch within the Hydrologic Forecasting and Water Management Division, Manitoba Infrastructure and Transportation has the following primary objective:

> *"... the provision of flood condition reports, forecasts and warnings to enable effective coordination of flood response planning and flood fighting activities at all levels of government and the private sector. The Centre thus promotes public safety and flood damage reduction".*

Within this context, the Hydrologic Forecasting and Water Management Division is responsible for the operation of dams to help prevent or reduce flooding and to ensure adequate water supplies for irrigation, industrial and personal use, and for providing data and forecasts for the operation of floodways and diversions.

In addition, the Hydrologic Forecasting and Water Management Division has responsibility for, (among other tasks):

- Conducting hydrologic and hydraulic analysis and modelling to determine the impacts of developments, land use changes and climate change on surface water regimes.
- Continuing to develop and/or improve flood forecasting techniques.

It is clear that the Hydrologic Forecasting and Water Management Division is responsible for a wide range of activities, including assessment of climate change within the water sector in Manitoba that go beyond the monitoring and forecasting of flood events.

Over the past five years, the understanding of climate change impacts and adaptation in Canada has increased both as a result of new research and through practical experience. Key conclusions arising from this update to the 2008 assessment report, "*From Impacts to Adaptation: Canada in a Changing Climate*" (NRC, 2014) include:

- Canada's climate is changing, with observed changes in air temperature, precipitation, snow and ice cover and other indicators. Further changes in climate are inevitable.
- Changes in climate are increasingly affecting Canada's natural environment, economic sectors and the health of Canadians.
- Extreme weather events are a key concern for Canada, and there is growing confidence that some types of extreme events will increase in frequency and/or intensity as the climate continues to warm.
- Adaptation is accepted as a necessary response to climate change, complementing global measures to reduce greenhouse gas emissions. Adaptation enhances the social and economic resilience of Canadians to climate change impacts.
- Adaptation is occurring with increasing frequency and enhanced engagement. Continued action will help to build capacity, address information needs and overcome challenges.
- Adaptation can sometimes turn risks into opportunities, and opportunities into benefits.
- Collaboration and adaptive management are approaches that governments and industry are increasingly pursuing to advance adaptation.

5.2.4 Flood Frequencies for the Assiniboine River Downstream of Portage la Prairie

Flows on the Assiniboine River downstream of Portage la Prairie have been regulated by the Portage Diversion since 1970 by diverting excess floodwaters to Lake Manitoba in 33 of the

past 45 years. This creates many difficulties in conducting frequency analyses of flood flows on the Lower Assiniboine River. For the purposes of this study, it was determined that flood frequencies on the Assiniboine River upstream of Portage la Prairie would form the basis for the flood frequencies downstream of Portage la Prairie. It was assumed that a portion of the Assiniboine River flow would be diverted into the Portage Diversion. The flood frequency curve for the Assiniboine River at Holland (Appendix C) was used to define Assiniboine River conditions upstream of Portage la Prairie.

The assumed portion of flow diverted into the Portage Diversion was based on the current operating guidelines and experience with operation of the Portage Diversion in the past. The key assumptions are listed below:

Open Water Conditions:

- The Portage Diversion is not operated when flows in the Assiniboine River upstream of Portage la Prairie are less than 283 m^3/s (10,000 cfs).
- A minimum of 75% of flows that exceed 283 m^3/s (10,000 cfs) upstream of Portage la Prairie are diverted into the Portage Diversion.
- The Portage Diversion is operated up to its maximum capacity of 708 m^3/s (25,000 cfs) to maintain flows below 510 m^3/s (18,000 cfs) in the Assiniboine River downstream of Portage la Prairie.
- Flows in the Portage Diversion cannot exceed its design capacity of 708 m^3/s (25,000 cfs).

Ice Conditions:

- The Portage Diversion is operated up to its maximum capacity of 708 m^3/s (25,000 cfs) to maintain flows below 142 m^3/s (5,000 cfs) in the Assiniboine River downstream of Portage la Prairie, when ice is present in the river.
- Flows in the Portage Diversion cannot exceed its design capacity of 708 m^3/s (25,000 cfs).

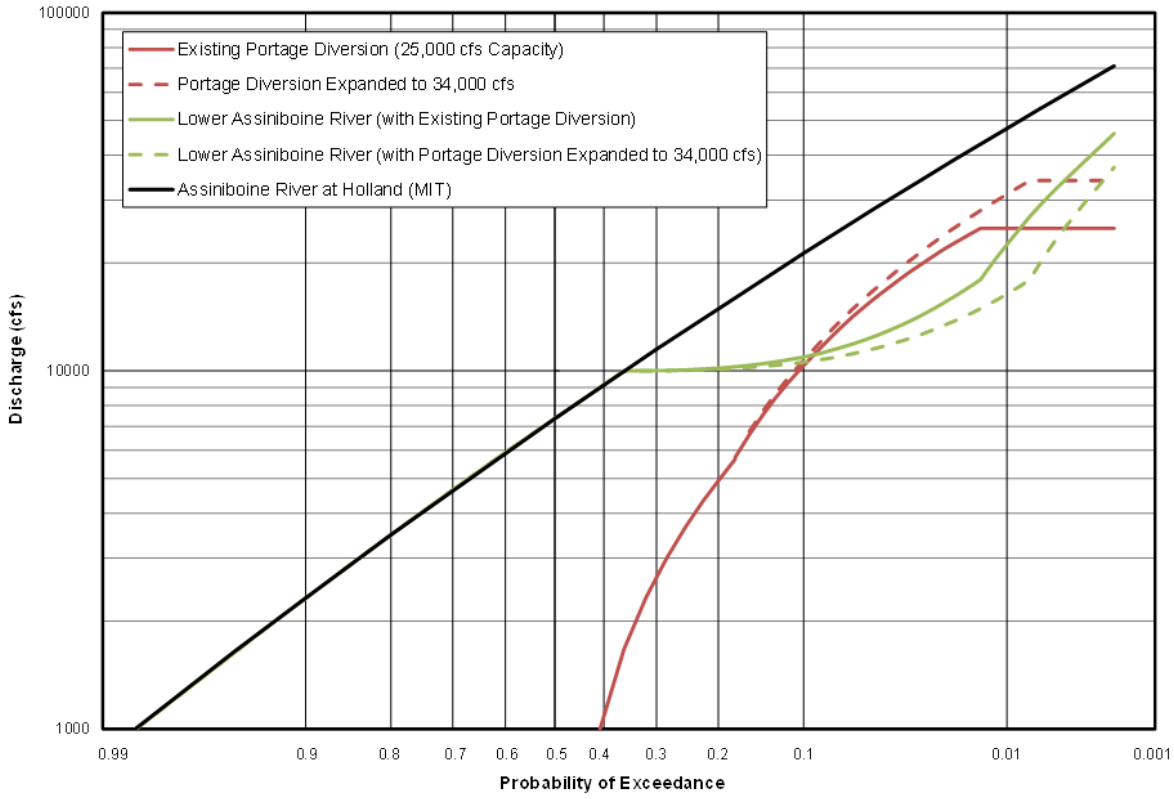
A review of the Provincial Flood Control Infrastructure Operating Guidelines was initiated by the Province of Manitoba in 2014. The outcome of the review may recommend changes to the Portage Diversion operating guidelines. However, without prior knowledge of what those changes may be, only the current operating guidelines and past operation conditions were considered valid for the hydrologic analyses of this study.

Based on the assumptions listed previously, flood frequency curves were developed for both the Portage Diversion and the Assiniboine River downstream of Portage la Prairie. The results are summarized in Table 5.4, and plotted on Figure 5.7.

Table 5.4
Frequency of Flows at Portage Diversion
Based on Maximum Capacity of 25,000 cfs

Return Period (years)	Portage Diversion		Assiniboine River downstream of Portage Diversion		Assiniboine River at Holland	
	m^3/s	cfs	m^3/s	cfs	m^3/s	cfs
500	708	25,000	1,303	46,000	2,010	71,000
200	708	25,000	909	32,100	1,617	57,100
100	708	25,000	793	22,500	1,501	47,500
70	708	25,000	510	18,000	1,218	43,000

Figure 5.7
Flood Frequency Curves of the Lower Assiniboine River and Portage Diversion



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

In 2011 and 2014, flows in the Portage Diversion were allowed to exceed its design capacity. Conceptual options to expand the capacity of the Portage Diversion have also been presented in this study. Based on similar operating assumptions for a 963 m³/s (34,000 cfs) Portage Diversion as those outlined above for the 708 m³/s (25,000 cfs) Portage Diversion, a flood frequency curve for an option wherein the Portage Diversion capacity has been expanded to 963 m³/s (34,000 cfs) is also provided on Figure 5.7. The results are summarized in Table 5.5.

Table 5.5
Frequency of Flows at Portage Diversion
Based on Maximum Capacity of 34,000 cfs

Return Period	Portage Diversion		Assiniboine River Downstream of Portage Diversion		Assiniboine River at Holland	
	m^3/s	cfs	m^3/s	cfs	m^3/s	cfs
500	963	34,000	1,048	37,000	2,010	71,000
200	963	34,000	654	23,100	1,617	57,100
100	878	31,000	467	16,500	1,345	47,500
70	793	28,000	425	15,000	1,218	43,000

5.3 STATISTICAL DEPENDENCIES / INDEPENDENCIES BETWEEN FLOOD EVENTS

5.3.1 Assiniboine River and Lake Manitoba

Lake Manitoba has a drainage area of approximately 79,800 km^2 (30,800 mi^2), which includes the surface area of the lake at approximately 4,500 km^2 (1,700 mi^2). Under normal conditions, the dominant source of inflow to Lake Manitoba comes from local runoff from the surrounding watershed. The largest tributary to Lake Manitoba is the Waterhen River, which has a drainage area of approximately 55,100 km^2 (21,300 mi^2), including the surface area of Lake Winnipegosis (over 5,300 km^2 (2,000 mi^2)). However, during periods of floods in the adjacent Assiniboine River watershed, the Portage Diversion Channel (PDC) may transfer water from the Assiniboine River and thereby contribute inflow to Lake Manitoba. The Assiniboine River has a drainage area of approximately 161,000 km^2 (62,100 mi^2) at Portage la Prairie. When necessary, the PDC diverts excess flow at Portage la Prairie away from the communities located downstream on the Assiniboine River, including Winnipeg.

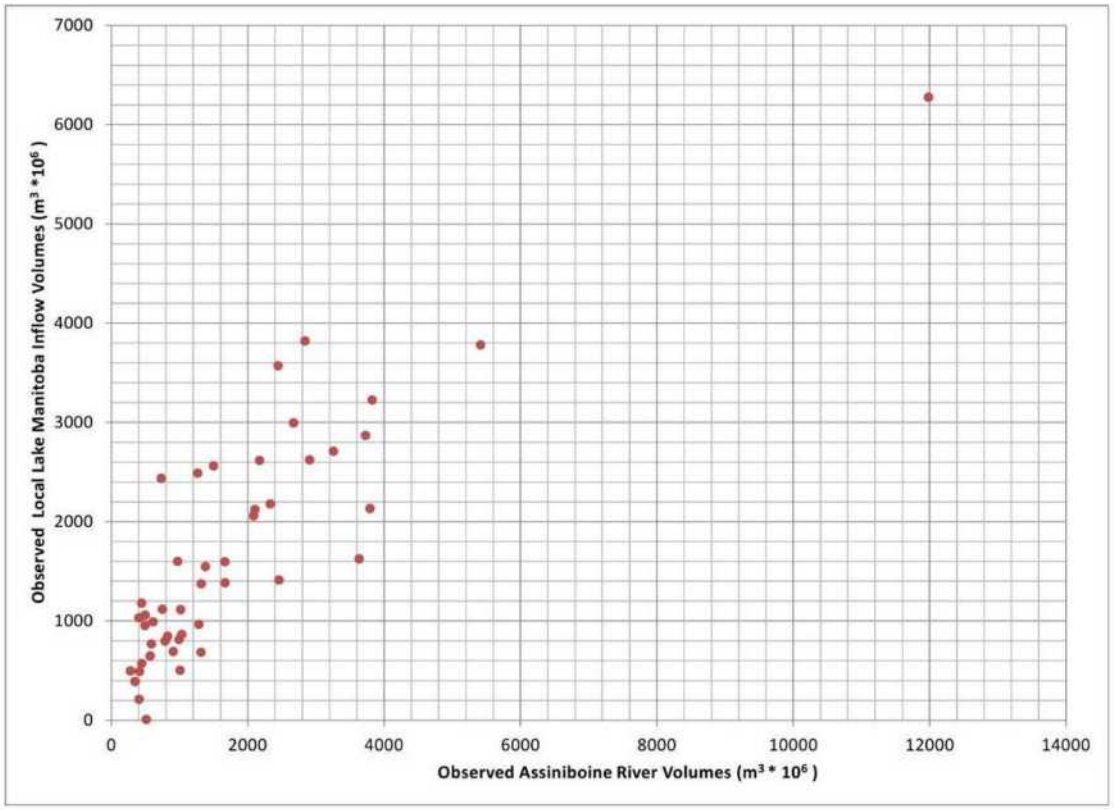
For the purposes of this study, inflow to Lake Manitoba was separated between the two sources:

1. Local inflow from the watershed surrounding Lake Manitoba.
2. Inflow from the Portage Diversion.

The volumes of the Assiniboine River flood were computed in this study by summing the daily reported river flows at the Water Survey of Canada Station Assiniboine River near Holland (Hydrometric Station 05MH005), for the period from April 1st to October 31st each year. This is a typical period when the PDC may be in operation. Calculation of the volumes of inflows to Lake Manitoba was based on the traditional method of computing "Inflow Available Outflow" (IAO) and implicitly included the components of precipitation and evaporation. After July, inflows to Lake Manitoba diminish sharply with significant increases in evaporation. The time period of April 1st to July 31st was, therefore, chosen to calculate the total volumes of those inflows for purposes of developing annual inflow hydrographs.

Floods on the Lake Manitoba and Assiniboine River watersheds are often, although not always, coincident. Figure 5.8 shows the recorded data of flood volumes from 1961 to 2011, inclusive. In most wet years, a relatively high runoff occurs in both catchments. Nevertheless, there are some years where the coincidence is less apparent. When all years are compared, the correlation coefficient for these datasets is approximately 0.86, which suggests a reasonably strong statistical correlation.

Figure 5.8
Observed Assiniboine River and Local Lake Manitoba Inflow Volumes



Results of this analysis on statistical dependencies / independencies of floods on the Lake Manitoba and Assiniboine River Basins formed the basis of the Monte Carlo Model of the Upper Assiniboine River and Lake Manitoba discussed in Section 8.5.1 of this report. Those results were used in the estimation of water levels and evaluation of benefits of the various flood mitigation options on Lake Manitoba.

5.3.2 Red River and Assiniboine River

An issue which has been of particular concern in this study is that pertaining to the likelihood of coincident peaks on the Assiniboine River and the Red River in Winnipeg. Coincident peaks would raise water levels in both rivers in the Winnipeg area with the potential to exacerbate flooding. This issue is of the greatest interest for large floods, which could stress the capability of flood control works. The study evaluated the magnitude and timing of the largest flood peaks on the two rivers during the period from 1826 to 1913. The effectiveness of Assiniboine crests in

contributing to Red River crests is computed for 36 flood events, and the frequency of large controlled Assiniboine contributions at Headingley is noted for several Portage Diversion capacities.

5.3.2.1 Hydrometric Database

Flows on the Assiniboine River in Winnipeg are highly controlled by the Portage Diversion, while flows on the Red River in Winnipeg are highly controlled by the Red River Floodway. A comparison of controlled flows in the Winnipeg area would be of little use for the planning of flood control works. An analysis of the contributions of the two rivers upstream of these control works is needed to assess the coincidence of flow crests and their effects on controlled river flows and levels in Winnipeg. Accordingly, recorded flows for the Assiniboine River at Holland (PTH 34) and the Red River near Ste. Agathe were used. The time of travel from Holland to Winnipeg is about two days greater than that from Ste. Agathe to Winnipeg and, therefore, a two-day shift was applied for the comparison of peak dates. Flow records at Holland are not available prior to 1961. Therefore, flow hydrographs for earlier years were estimated by adding daily flows for the Assiniboine River at Brandon and the Souris River at Wawanesa and further adding 70 m^3/s (2,500 cfs) for local flows downstream of these gauges. Peak flows for Holland prior to the Shellmouth Reservoir operation (before 1969) were adjusted to present use based on a comparison of natural and controlled flows at Brandon from a 2006 report by Manitoba Water Stewardship on the flood risk at Brandon (MWS, 2006).

Flows for the Red River at Ste. Agathe are not available prior to 1958. There is usually a significant inflow between Emerson and Ste. Agathe, and thus, flood routings would be required to generate flow estimates for Ste. Agathe prior to 1958. Since such routings were not readily available, recorded peaks for the Red River at Winnipeg were used prior to 1958 when major flood control works were absent. A six-day time of travel for the (Brandon+Wawanesa) flows to Winnipeg was used for this period.

Systematic hydrometric records do not exist on the Assiniboine River or Red River prior to 1912. Peak flows and dates for flood years from 1826 to 1904 were obtained from reports by W.F. Rannie, University of Winnipeg (Rannie,1999), Canada Water Resources Division Report (CWRD, 1953) , Manitoba Water Stewardship regarding the flood risk at Brandon (MWS, 2006),

Royal Commission on Flood Cost Benefit (RC, 1958). Information on historical flood peaks at Grand Forks, N.D. was obtained from a U.S. National Weather Service web site (US NWS, 2014).

5.3.2.2 Coincidence of Major Flood Peaks

Annual peak discharges and dates for 36 flood events are shown on Table 5.6. While flood peaks on the Red and Assiniboine (with appropriate lag times to Winnipeg) often occur within a few days of each other for smaller events, they are usually different by at least one or two weeks for larger floods. If the Red River experiences an extreme flood while the Assiniboine has only a moderate or small flood, the Red River crest is almost always later. When the Assiniboine has a major crest and the Red River has only a moderate or small crest, the Assiniboine crest is almost always later. One notable exception is the 1882 Flood, when a record high Assiniboine crest appears to have occurred about a month earlier than a large Red River crest. However, it is possible that the anecdotal data for this flood is in error with respect to the dates.

Table 5.6
Red River and Assiniboine River Flood Crests for 36 Flood Events from 1826 to 2013

Year	Red River Ste. Agathe Peak (m^3/s)	Date	Controlled Assiniboine at Holland Peak (m^3/s)	Date	[1] Timing Factor (% of Holland Crest Effective at Wpg)	[2] Baie St. Paul (flow in excess of bank full 708 m^3/s PD capacity)	[2] Baie St. Paul (flow in excess of bank full 963 m^3/s PD capacity)
2013	1165	4-May	530	1-May	91		
2011	2140	30-Apr	1460	13-May	67	292	37
2010 a	1730	6-Apr	117	9-Apr	91		
2010 b	1190	2-Jun	100	30-May	36		
2009	2570	21-Apr	620	13-Apr	69		
2005 a	1380	9-Apr	618	6-Apr	68		
2005 b	1540	5-Jul	530	11-Jul	85		
2001	1640	26-Apr	482	9-May	84		
1999	1660	16-Apr	542	25-May	56		
1997	3230	1-May	552	18-Apr	66		
1996	2120	2-May	581	18-Apr	81		

Year	Red River Ste. Agathe Peak (m^3/s)	Date	Controlled Assiniboine at Holland Peak (m^3/s)	Date	[1] Timing Factor (% of Holland Crest Effective at Wpg)	[2] Baie St. Paul (flow in excess of bank full 708 m^3/s PD capacity)	[2] Baie St. Paul (flow in excess of bank full 963 m^3/s PD capacity)
1995	1430	1-Apr	780	29-Apr	17		
1987	1390	8-Apr	333	7-Apr	98		
1979	2320	5-May	546	24-Apr	78		
1976	1180	8-Apr	1460	20-Apr	20	292	37
1974a	1420	26-Apr	909	22-Apr	90		
1974b	1160	24-May	909	22-Apr	47		
1970	1450	1-May	501	28-Apr	100		
1969	1460	3-May	694	24-Apr	61		
1966	1870	16-Apr	203	9-May	71		
1960	1060	18-Apr	*393	16-Apr	98		
1956	^1974	27-Apr	*445	16-May	61		
1955	^1615	12-Apr	*604	20-May	34		
1950	^3030	19-May	*279	10-May	85		
1948	^2123	30-Apr	*512	7-May	46		
1927	^1630	16-May	*456	5-May	96		
1923	^1855	28-Apr	*528	7-May	51		
1922	^1076	20-May	*538	6-May	82		
1916	^2426	22-Apr	*491	18-Apr	52		
1913	^1218	15-Apr	*364	1-May	27		
1904	^2208	24-Apr	*1122	9-May	50e		
1902	^1500	15-Apr	*917	4-Jul	50e		
1897	^2600	24-Apr	*1072	?	80e		
1882	^2420	5-May	*1542	5-Apr	50e	374	119
1861	^3540	6-May	1000e	?	?		
1852	^4670	19-May	1500e	27-May	85e	332	77
1826	^6370	22-May	1400e	30-May	80e	232	

^ Peak discharge for Red River at James Avenue in Winnipeg.
* Computed controlled peak flow for Holland based on Brandon, plus Wawanesa, plus 70 m^3/s local flow.
? Historic value unknown.
1. Holland controlled crest two days before Ste. Agathe crest, divided by Holland crest that season as a percentage.

2. Controlled by Shellmouth and Portage Diversion and with 50 m^3/s net local inflow d/s Holland.
3. Bank full capacity of 510 m^3/s assumed from Portage to Baie St. Paul.
4. Flows can be converted to cfs by multiplying metric values by 35.3.

The difference in timing of the Assiniboine and Red River crests may not be the best indicator of the severity that it implies for flood levels in the City of Winnipeg. Hydrographs for major floods tend to have a relatively long duration of high flows due to the extensive overbank storage. Thus, if the flood hydrographs have a relatively flat top, it is possible that a ten day difference in dates means very little in terms of flow magnitude. To assess the significance of the differences in crest dates, the Assiniboine River crest on the day of the Red River crest was expressed as a percentage of the seasonal Assiniboine River crest. The percentages are shown on Table 5.6 in the column noted as the 'Timing Factor'. For example, the 2009 Flood has a percentage of 69. The Assiniboine crest at Holland was 620 m^3/s (21,900 cfs) on April 13. However, the flow had decreased to 427 m^3/s (15,100 cfs) by the time the crest occurred on the Red River at Ste. Agathe on April 21, thus 427/620 = 0.69 or 69%. The percentages are mostly in the 60% to 80% range for the largest Red River floods at Winnipeg, although those for some large Assiniboine floods tend to be much lower. The lack of discharge or stage hydrographs for the floods prior to 1905 makes it very difficult to estimate the percentages for these floods.

The relatively low timing factor percentages for most larger floods shows that coincident large peaks on the Red River and Assiniboine River are quite rare. The occurrence of coincident peaks may actually be beneficial for the City of Winnipeg for most floods, as described in the next section of this report.

5.3.2.3 Effect of Flood Control Works on Coincident Peaks

Coincident peaks on the Red River at St. Norbert (downstream of the Red River Floodway) and on the Assiniboine River at Headingley will result in higher levels in the City of Winnipeg than non-coincident peaks of the same magnitude. This has led to the belief by some that coincident peaks at Holland and St. Adolphe also pose a greater threat to the City of Winnipeg. This is not the case, unless the floods exceed the capacity of the Portage Diversion and the Red River Floodway. Except for such extreme floods, coincidence is a benefit for Winnipeg since it enables more effective use of flood control works. Coincidence means that Assiniboine River flows upstream of the Portage Diversion will be greater than for non-coincidence. This produces

a higher natural level on the Red River at the Floodway Inlet, thus enabling more water to be diverted down the Floodway. Meanwhile flows on the Assiniboine River entering Winnipeg will not be higher due to the coincidence, since the excess from Holland is simply diverted via the Portage Diversion. The net result is lower river flows and levels in Winnipeg. This is one of the reasons why river levels in Winnipeg remained below El 18 ft (JAPSD) during the period of extreme flows on the Assiniboine River at Holland in May 2011. Levels in Winnipeg declined during the period May 6-16 when the Assiniboine at Portage climbed to record highs, despite minimal declines in Red River flows at Ste. Agathe.

5.3.2.4 Coincidence of Early Flood Crests from Manitoba Runoff

The Assiniboine River often produces more than one spring crest due to the vastness of the watershed and variable topography. The time of travel from headwater areas such as the Saskatchewan portions of the Souris River, Qu'Appelle River and Upper Assiniboine River to Winnipeg is in the order of two to four weeks. The most extreme Assiniboine River floods are produced by heavy runoff from two or three of these headwater areas, compounded by significant runoff from the Manitoba portions of the watershed. An early crest from the Manitoba portion is quite common, followed by a crest from headwater areas some weeks later. The first crest is produced by runoff from the area downstream of Brandon including the portion of the Souris River downstream of Souris, Manitoba. This portion often produces the largest crest of the year at Holland and at Portage la Prairie (upstream of the Portage Diversion). The largest such crest on record occurred on April 22, 1974, at 909 m^3/s (32,100 cfs mean daily) and far surpassed the second crest from headwater areas. The Red River seldom produces multiple spring crests. An exception was 1950 when heavy rainstorms in early May resulted in a much larger second crest. However, the annual crest on the Red River at the Floodway Inlet can be an early one produced mainly by runoff from the Manitoba portion. The largest Red River crests are invariably the result of heavy runoff from the larger U.S. portion of the watershed.

The first crest on the Assiniboine River has a high probability of coinciding with an early crest on the Red River. Since both result from Manitoba runoff, the time of travel to Winnipeg is similar. Also, the odds are quite high that both could be of similar relative magnitude in a given year since the causal factors (soil moisture, snow cover, melt conditions, rain) are for a relatively small geographic area. Thus, if one has a 90^{th} percentile runoff, the other likely will have a

similar event. Conditions for 1974 are a good example, where an early crest of 1,420 m^3/s (50,100 cfs) at Ste. Agathe on April 25-26 virtually coincided with the first crest on the Assiniboine River producing a natural crest of 28.0 ft at James Avenue. However, these early flood peaks are still relatively small compared to flood peaks from headwater areas in Saskatchewan and the United States. Due to the relatively small drainage areas, it is unlikely that early flood peaks from Manitoba runoff would exceed the capacity of flood control works. For example, the early flood peak in the Assiniboine River in 1974 was reduced to 419 m^3/s (14,800 cfs) at Headingley; and the added effect of Red River Floodway operation resulted in a controlled crest of only 17.4 ft at James Avenue on May 2, 1974. This example demonstrates that coincident crests from Manitoba runoff on the Red and Assiniboine rivers, while quite frequent in occurrence, typically occur when the flood magnitudes are low or moderate. They are typically not a serious threat to the City of Winnipeg.

5.3.2.5 Effect of Local Rainstorms

The spring flood crest of 1974 appeared to have passed, and the Red River Floodway operation had ceased by mid-May. Then a series of thunderstorms produced up to 100 mm (4 in) of rain on the May long weekend, mainly on the western tributaries of the Red River in Manitoba. This resulted in a flood crest of 19.2 ft at James Avenue on May 22, 1974 despite renewed operation of the Floodway. These rains had very little effect on Assiniboine River flows upstream of Headingley. The crest at Ste. Agathe following this storm was 1,160 m^3/s (41,000 cfs), considerably lower than the April crest; and the flow at Holland (lagged to Winnipeg) was less than half of the April crest. Part of the reason for the high crest at Winnipeg was the low flows at Portage la Prairie, which reduced natural flows and levels in the Winnipeg area and resulted in less water being diverted via the Red River Floodway. The other reason was very high flows on local streams such as the La Salle River, Sturgeon Creek and other streams in the Winnipeg area, which are uncontrolled. The 1974 May long weekend downpours are an example of how heavy thunderstorms usually affect only small portions of the Assiniboine and Red River watersheds, thus minimizing the possibility of coincident high crests on both rivers due to torrential rains.

5.3.2.6 Coincident Major Flood Peaks from Headwater Areas

Floods which pose the greatest threat to the City of Winnipeg are those generated by very high runoff from most portions of the Red River and Assiniboine River watersheds, including large areas lying outside of Manitoba. Such floods have the potential to overwhelm the flood control system protecting Winnipeg. Since the combined watersheds are so large (from the Duck Mountains in the north, Moose Jaw to the west, northern South Dakota to the south and central Minnesota to the east), it would be expected to be rare that causal parameters such as autumn soil moisture, snow water content and spring rain would be extreme simultaneously over the entire 285,000 km^2 (110,000 mi^2) watershed. Also, it is rare that all the causal parameters would be very high in any given year. This is confirmed by an examination of soil moisture and snow cover maps for the period since 1948, but this is not to say that it can never happen. Conditions leading up to the 2011 Flood were unusually high over most of the Red and Assiniboine watersheds. Spring and/or summer rains are typically smaller scale and are seldom heavy over the entire watershed. For example, the major May rainstorm, which produced the record Souris River flood in 2011, affected mainly the Saskatchewan portion of the Souris River watershed. Nevertheless, a series of rainstorms over a period of three to four weeks could produce heavy rain over large portions of the Souris, Assiniboine and Red River watersheds. The 2011 Flood in the Assiniboine River is another example. It is possible that even larger floods will develop in the future, although such floods would be rare. The 2013 Flood in the South Saskatchewan River watershed is a reminder that floods can exceed even anecdotal record floods of the past.

The issue at hand is the probability of coincident major floods on the Red River and Assiniboine River of a magnitude that would overwhelm flood control works. This already occurred on the Assiniboine in 2011 when the discharge capacity of the Portage Diversion needed to be increased on an emergency basis to prevent serious flooding on the Lower Assiniboine River. Fortunately, the Red River did not experience a major flood during 2011; and, moreover, the Red River crested well ahead of the Assiniboine, resulting in a timing factor of only 67% (Table 5.6).

While meteorological factors suggest that major floods in both the Assiniboine and Red watersheds in any one season have a low probability of occurrence due to the enormous size of the watershed, accounts of actual widespread severe floods in large watersheds would be the

best indicator, if a long period of flood records were available. The record summer flood of 1993 throughout the large Missouri River and upper Mississippi watersheds is an example, which shows that successive rainstorms over a period of four to six weeks can affect large areas and produce severe flooding. Historical floods in the Red and Assiniboine watersheds, including anecdotal floods, could shed much light on the issue, if the information was reliable. Unfortunately, records for the largest floods were not reliable, especially for the Assiniboine River.

While there were references to high stages and ice flows on the Assiniboine River in 1826 and 1852, these were based on observations near the Forks, where levels were severely affected by the backwater effects from the Red River. Rannie (1999) cites other evidence of high Assiniboine River and Souris River flows at points further upstream, such as at Portage la Prairie, Minot and the Qu'Appelle River, which suggests that Assiniboine flows near Portage la Prairie were likely similar to those of 1976. Peak flows appear to have been at least a week later than on the Red River (both at Winnipeg); but with high volumes on both rivers, hydrographs would likely have flat tops, so the difference in dates may be of little significance. With Red River flows (upstream of the Forks) in 1852 at least as great as those of 1997 (4,672 m^3/s - 1,213 m^3/s in Table 5.6, or 165,000 cfs-42,800 cfs) and 1826 Red River flows at least 1,600 m^3/s (56,500 cfs) greater than in 1852, the combination of Red and Assiniboine crests would have strained the flood control system protecting Winnipeg.

Severe floods are known to have occurred at Brandon in 1882, 1897, 1902 and 1904; but little is known about flood crests on the Souris River in Manitoba and the Assiniboine River at Portage la Prairie during these years. Rough estimates of flows at Holland were made based on estimates of flood peaks in the Souris River at Minot as reported by Rannie (1999). Crests on the Red River upstream of the Forks during these floods were much lower than in 1997. While crest dates for these floods are somewhat uncertain, it appears that there was a large difference in crest dates between the Red and Assiniboine, resulting in low percentages for the Timing Factor. If these Assiniboine floods were to reoccur, they would stress flood control works on the Assiniboine River; but the threat to the City of Winnipeg would depend largely on the magnitude of the associated Red River flood at the Floodway Inlet. This is discussed further in the following section.

5.3.2.7 Effect of Portage Diversion on Assiniboine River Floods

The implication of Assiniboine River floods at Holland on flows at Baie St. Paul is shown on Table 5.6 for PD capacities of 708 m^3/s (25,000 cfs) and 963 m^3/s (34,000 cfs). A net local inflow of 50 m^3/s (1,760 cfs –in excess of losses to channel storage) is assumed. The present channel capacity of 510 m^3/s (18,000 cfs) downstream of Portage la Prairie is exceeded in five years for the 708 m^3/s (25,000 cfs) PD capacity and in four years for the 963 m^3/s (34,000 cfs) capacity. The estimated Baie St. Paul flow of 557 m^3/s (19,600 cfs) for 2011, for the 963 m^3/s (34,000 cfs) capacity of the PD, somewhat exceeds the observed flow of 510 m^3/s (18,000 cfs). This is due to minimal local flow in 2011 due to lack of rain downstream of Holland near the time of the crest. The 1976 crest at Holland had a sharp crest, which attenuated as the flood travelled downstream; so the recorded crests downstream of Portage la Prairie are somewhat less than shown in Table 5.6.

The estimated peak flows at Baie St. Paul for major historical Assiniboine River floods exceed the bank full discharge capacity by less than 400 m^3/s (14,100 cfs) for the original discharge capacity of the PDC, and by 37 m^3/s (1,300 cfs) to 119 m^3/s (4,200 cfs) for the 963 m^3/s (34,000 cfs) discharge peak through the PDC in 2011. Controlled flood peaks at Baie St. Paul range from 802 m^3/s (28,300 cfs) to 884 m^3/s (31,200 cfs). For most floods, these flood crests would normally attenuate before reaching the Red River due to overbank storage and flows into the La Salle and Sturgeon Creek watersheds.

5.3.2.8 Effects of Coincident Floods on Controlled Flows at James Avenue in Winnipeg

Additional information on the coincidence of floods may be obtained from a comparison of natural and controlled flows on the Red and Assiniboine Rivers in the Winnipeg area for the largest floods on each river (Table 5.7). Flows on the following table are in m^3/s and stages at James Avenue are in feet above datum.

Table 5.7
Summary of the Largest Recorded Floods on the Red and Assiniboine Rivers

Year	James Avenue Natural Peak (m^3/s)	Assiniboine River		Floodway Inlet Peak Flow (m^3/s)	James Avenue Peak Stage		
		Portage Inflow (controlled) (m^3/s)	*Headingley (coincident) (m^3/s)		Natural (ft)	Controlled (ft)	Date
1826	6370	1400	1120e	4970e	36.5	---	May 22
1852	4670	1500	1275e	3170e	34.5	---	May 19
1997	4620	365	55	3908	34.4	24.5	May 3
2009	3568	700	337	2633	32.5	22.3	Apr 16
1861	3540	1000	?	2400e	32.5	---	May 8
2011	3276	1470	521	2124	31.2	19.6	May 6
1979	3058	410	241	1900	30.4	19.2	May 9
1950	3030	200	232e	2700	30.3	21.5	May 19
1974	2050	380	410	300	22.8	19.2	May 21
1916	2430	215	440e	1850	23.5	---	Apr 22
1882	2420	1540	770e	1600e	25.7	---	May 8
1897	2250	1070	860e	1850e	22.4	---	Apr 13
1904	2208	1120	560e	1500e	24.2	---	Apr 24
1976	1810	1400	100	1300	20.8	15.8	Apr 9
1902	1500	920	460e	940e	18.1	---	?

Notes:
e Estimated from limited historical information. PDC operation is not included in Assiniboine flows at Headingley.
* Controlled flow two days prior to the Red River crest at James Avenue in Winnipeg.
? Historic value unknown.
1. Flows can be converted to cfs by multiplying metric values by 35.3.

The summary of large floods shows that, for most large Red River floods, such as those of 2009, 1997, 1979, 1950, 1897 and 1861, controlled inflows from the Assiniboine River to the Portage Reservoir were not particularly high. Operation of the Portage Diversion would have enabled all the Lower Assiniboine crests for these floods to be reduced to below the bank full discharge capacity at the time of the crest in downtown Winnipeg. All pre-regulation flood crests at Portage la Prairie, except those of 1826, 1852 and 1882 could have been reduced to minimal values by PDC operation, had it been available. Only the 1852 and 1882 crests would still have exceeded the bank full discharge capacity downstream of Portage la Prairie based on a

discharge capacity of the PDC of 708 m^3/s (25,000 cfs). Major Assiniboine floods such as those of 2011, 1976, 1904 and 1882 occurred in conjunction with less than extreme Red River floods and did not, or would not have posed a threat to the City of Winnipeg with present flood control works. The 1882 Assiniboine River crest could have produced difficulties along the Lower Assiniboine River. High Assiniboine River flood crests typically occurred weeks after the Red River flood crest, and, therefore, the effective Assiniboine flows at Headingley (effective in raising Red River levels in Winnipeg) are much lower than the Assiniboine River flood crest at Southport in Portage la Prairie. The 1852 and 1826 floods appear to be possible exceptions to this.

Computed regulated peak stages for the 1826 and 1852 floods were not readily available. However, the available data suggests that these floods would still pose some difficulties for the City of Winnipeg.

5.3.2.9 Probability of Greater Floods and Their Coincidence

A previous study on the probability of Red River floods greater than any known or anecdotal floods was performed in a 1999 study by A.A. Warkentin (Warkentin, 1999). Synthetic flood peaks for the Red River at Winnipeg were generated via random generation of five flood causal parameters. The results suggested that Winnipeg might experience a flood crest of 5,663 m^3/s (200,000 cfs) once every 200 years, 6,938 m^3/s (245,000 cfs) every 500 years and 8,353 m^3/s (295,000 cfs) every 1,000 years. While the methodology used is speculative, there is little doubt that floods larger than those known to date have the potential to occur in the future. The record flood of 2011 on the Assiniboine River, that of 1993 on the Missouri and Mississippi and the record 2013 Flood on the South Saskatchewan River suggest that floods are becoming more extreme, perhaps due to changes in climate and changes in land use patterns.

While larger floods are likely to occur in the Red and Assiniboine watersheds in the future, this may not significantly increase the flood risk for the City of Winnipeg unless major floods occur simultaneously in both watersheds. The flood ‘record’ from 1826 onward suggests that the probability of simultaneous major floods is quite remote, having occurred only once or twice during the 188 year period from 1826 to 2013. The flow record for the Assiniboine River is very poor prior to 1912, so there is considerable uncertainty based on the flood ‘record’. The more

reliable record since 1912 based on daily recorded river flows strongly suggests that the probability of coincident major floods is low, since there is not a single case during this 102-year period where it has occurred.

Another approach to assess the likelihood of coincident major floods is that based on meteorology and climate. As indicated previously in this report, causal parameters such as soil moisture and snow cover can be extreme over both watersheds since they develop over many months and can result from numerous storms, each affecting only portions of the Red River watershed upstream of James Avenue, Winnipeg. While these parameters can produce large floods, it is unlikely that these floods will be extreme. Extreme floods still require plenty of rainfall on top of saturated soils and major snow packs. Such heavy rains are unlikely to fall over both watersheds since the time period is usually only weeks rather than months for soil moisture and snow cover. Heavy rains, other than localized thunderstorms, tend to be associated with the atmospheric jet stream persisting in a given pattern, thus resulting in repeated storms in a given region. Examples are the 1826 and 1950 rains over the Red River watershed (upstream of Winnipeg) and the 1999, 2011 and 1955 rains over the Upper Assiniboine and Souris watersheds. Another factor is the timing and rapidity of the snowmelt, which for example, can vary greatly from the U.S. portion of the Red River to the Assiniboine Basin upstream of Brandon. This promotes poor timing of the crests, with Assiniboine River crests typically following Red River crests by several weeks. The meteorology, therefore, suggests that extreme floods, such as 1826 on the Red River and 2011 on the Assiniboine River are very unlikely, while not impossible.

5.3.2.10 Conclusions

1. The first Assiniboine River crest from the area downstream of Brandon often coincides with a crest from the Manitoba portion of the Red River. However, these portions of the watersheds are too small to create a serious flood risk for the City of Winnipeg. Crests from larger portions of the watersheds including out of Province portions are the main concern.

2. Coincident crests on the Assiniboine River at Portage la Prairie and the Red River at Ste. Agathe tend to reduce river levels in Winnipeg, as long as the discharge capacity of the Portage Diversion is not exceeded. This is due to the fact that higher natural flows on the Assiniboine River enable greater diversions via the Red River Floodway, while Assiniboine flows toward Winnipeg can be held steady or be reduced by operation of the Portage Diversion.

3. Torrential rains, such as those of the Victoria Day long weekend in 1974, can produce severe localized flooding in Winnipeg and elsewhere. However, such intense thunderstorm rains typically affect relatively small areas and thus are not a threat for producing extreme river levels in themselves.

4. The historical flood record for the period of continuous hydrometric record since 1912 strongly suggests that the probability of extreme floods occurring simultaneously on both the Assiniboine and Red River watersheds is very low. There is no such occurrence during the entire 102 year period of record.

5. The historical anecdotal floods of the 1800's suggest that major floods occurred in both the Red and Assiniboine watersheds in 1826 and 1852. However, the estimated discharges for the Assiniboine River are unreliable since there are no stage or flow records outside of downtown Winnipeg. Nevertheless, the possibility of extreme flows like those of 2011 on the Souris and Assiniboine rivers cannot be discounted. If the flows were in the extreme range, then there would be at least two cases where extreme floods occurred in both of those watersheds during the past 188 years.

6. Extreme floods occurred on the Assiniboine River in 1882, 1904, 1976 and 2011; however, in each case, Red River flows were only in the moderate range. Large Red River floods such as those of 1950, 1979 and 1997 all had moderate to minor Assiniboine River flows.

7. Most large Assiniboine River crests upstream of Portage la Prairie occur after the crest of the Red River at St. Norbert. As a result, the Assiniboine River contribution to Red River crests is much less than the Assiniboine crest itself. This difference in timing of crests greatly reduces Winnipeg flood peaks from what they could have been if the timing was coincident. The floods of 1826 and 1852 may have been exceptions to this, but there is much uncertainty due to the lack of sufficient information.

8. Meteorological considerations suggest that the occurrence of simultaneous extreme floods in the Red River and Assiniboine River is low, mainly due to the fact that heavy rainfall seldom covers very large areas. Heavy rain is needed to transform serious snowmelt floods into extreme events or to produce extreme summer floods.

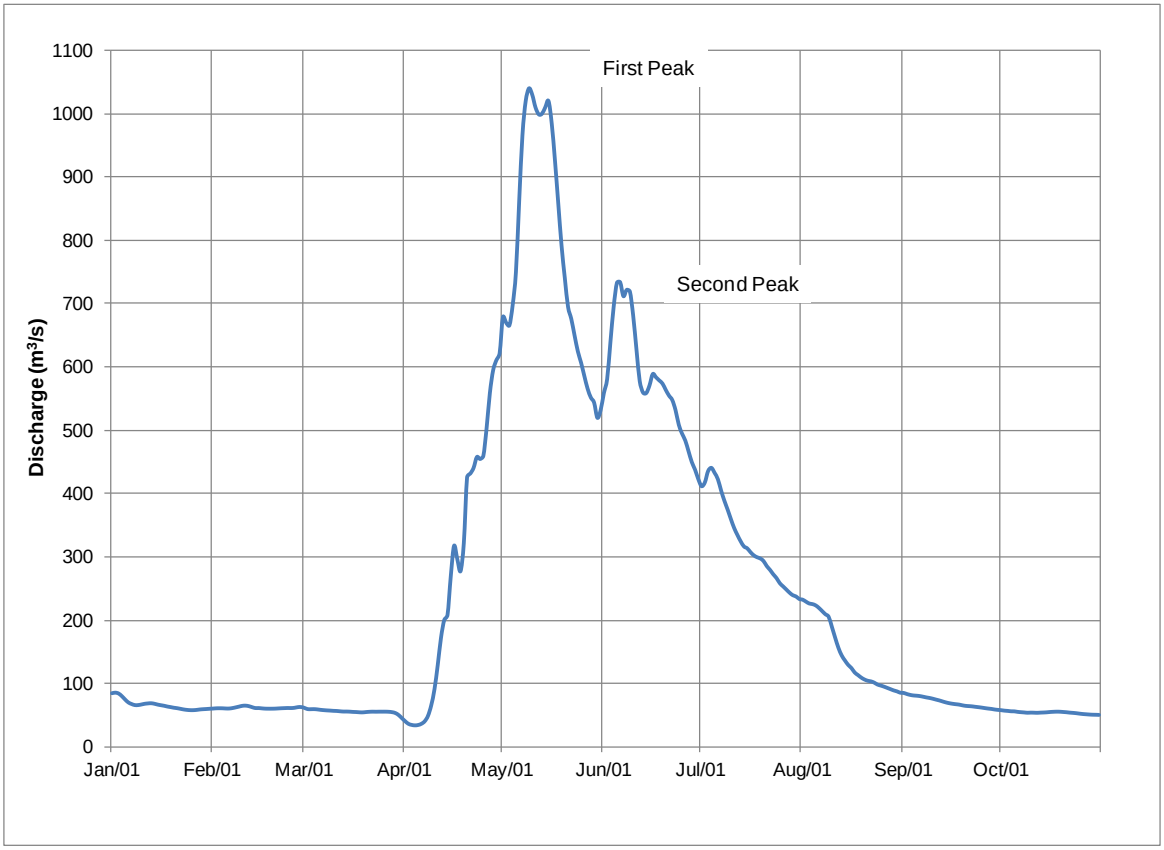
9. Based on the review described above, the probability of simultaneous major floods in the Red River and Assiniboine River watersheds is believed to be very low.

5.4 DEVELOPMENT OF DESIGN HYDROGRAPHS

Extensive hydrologic data exists on the Assiniboine River at Brandon and the Souris River for a period of record spanning over 100 years. Daily flows have been affected by regulation. On the Assiniboine River upstream of Brandon, regulation of flows occurs from the Shellmouth Dam (Lake of the Prairies) and from natural lakes at Good Spirit Lake and Fishing Lake. The Qu'Appelle River reservoirs include Eyebrow Lake, Buffalo Pound Lake and Last Mountain Lake (Long Lake).

The effect of the reservoirs is to extend the runoff period from snowmelt and rainfall events, which often mean that the runoff from the first event is not entirely complete before a secondary rainstorm event occurs, resulting in a secondary peak. This was evident on the Assiniboine River during the recent flood of 2011. Figure 5.9 illustrates the 2011 Flood discharges on the Assiniboine River near Brandon.

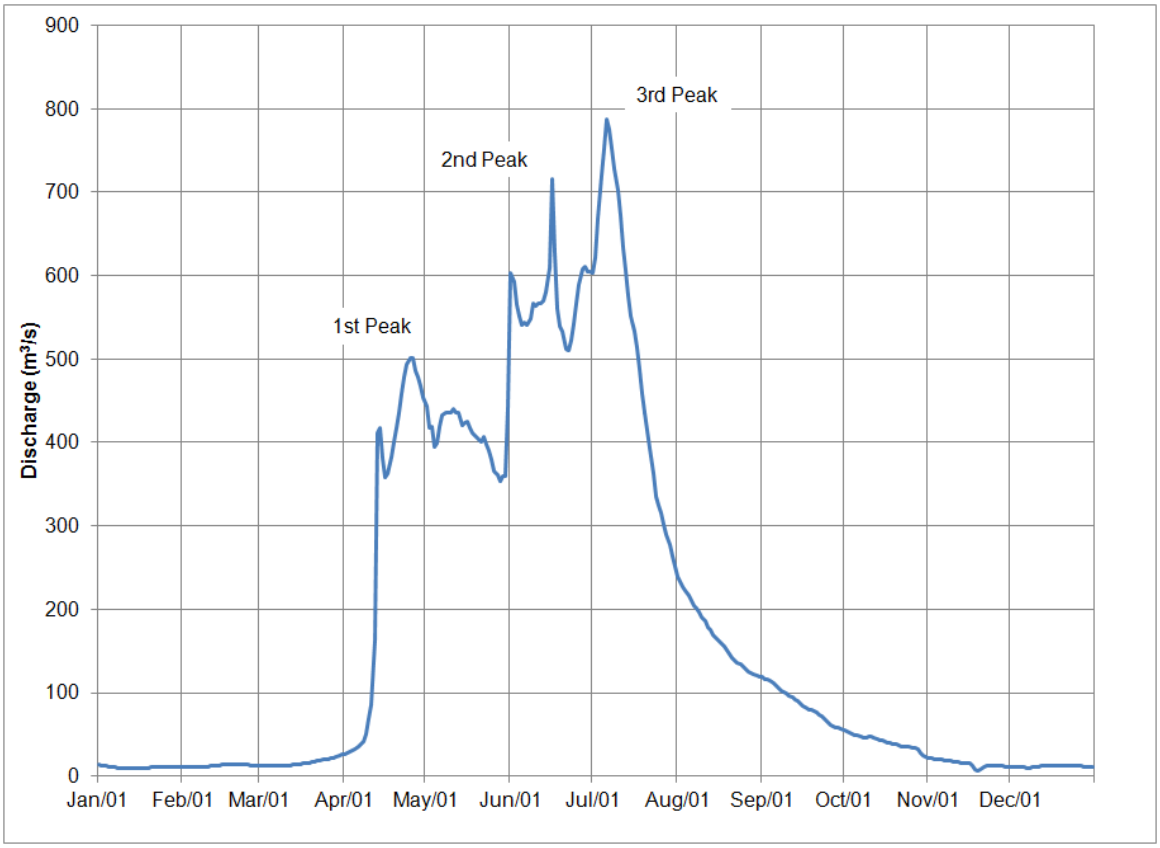
Figure 5.9
Assiniboine River near Brandon 2011 Hydrograph



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Similar conditions were observed on the Souris River during the 2011 Flood. Figure 5.10 illustrates the 2011 Flood discharges on the Souris River at Wawanesa. Snowmelt runoff combined with rainfall in April resulted in the initial peak on April 23. Additional rain storms during the summer produced two subsequent peaks on June 15 and July 5. The three peaks on the Souris River occurred from three independent events.

Figure 5.10
Souris River at Wawanesa 2011 Hydrograph



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

While daily discharges are usually reduced by regulation, which affects the hydrograph shape, the end of storm total runoff volume is not changed appreciably compared to unregulated conditions since water that is stored is eventually released by the end of the flood season. This must be taken into consideration when defining design hydrographs.

The methodology for the determination of design flood hydrographs for the Assiniboine River near Brandon and the Assiniboine River near Holland is summarized in this section. The hydrographs formed the basis for evaluating the discharge capacities of options for flood protection in the Assiniboine River and Lake Manitoba. Hydrographs have been developed for flood return periods of 2 years to 500 years. These have been derived with similar methods to those outlined in Section 7 of Hydrology for Engineers (Linsley et al., 1975). The final Hydrographs are in agreement with the peak discharges and runoff volumes obtained from the frequency analyses discussed in Section 5.2.

5.4.1 Design Flood Hydrograph Derivation Method

Careful consideration was given to selecting the typical hydrographs for defining the design hydrographs. For this study, typical hydrographs derived from the Souris River at Wawanesa and the Assiniboine River near Brandon during the 2011 Flood event were selected for the following reasons:

- The 2011 event occurs after 1995 once the construction of the Alameda Dam and Rafferty Dam was completed.
- Extensive hydrometric data is available at both locations for this event.
- Familiarity of the engineers preparing the analysis of the hydrologic conditions that occurred in 2011.
- The 2011 hydrograph included runoff from a combination of a snowmelt event followed by distinctive runoff events, which could be segregated and analysed separately.

Hydrographs on the Assiniboine River near Brandon and the Souris River at Wawanesa are provided on Figures 5.9 and 5.10. As discussed previously, the typical compound of the 2011 Flood hydrographs occurred as a result of independent meteorological conditions. The scaling of the total hydrograph is, therefore, done by scaling only one of the multiple events to the required peak discharge based on the frequency analysis of annual peak discharges. Scaling of more than one of the independent events would result in an event having a combined probability that is much greater than the return period of the peak frequency discharge.

The methodology used for this study in the derivation of the frequency flood hydrograph is summarized below:

- Typical hydrographs were selected (Figures 5.9 and 5.10).
- The combined hydrographs were segregated into separate independent hydrographs. Each segregated hydrograph represented either runoff due to snowmelt or due to a rainfall event. The shape of the segregated hydrographs was determined based on a review of historic hydrographs that occurred from single runoff events, and by comparing the rising and recession limbs of the hydrographs. A representative hydrograph shape was determined for each site and formed the basis of the segregated hydrographs for the Souris River, provided on Figures 5.11 and 5.12, and for the Assiniboine River, provided on Figures 5.13 and 5.14.

Figure 5.11
Souris River Segregated Hydrographs 1 and 2

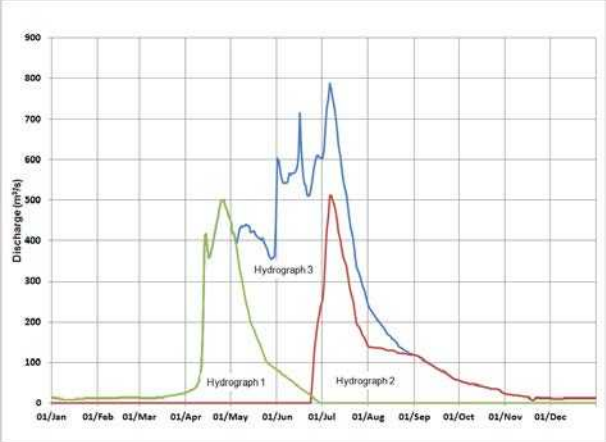


Figure 5.12
Souris River Segregated Hydrograph 3

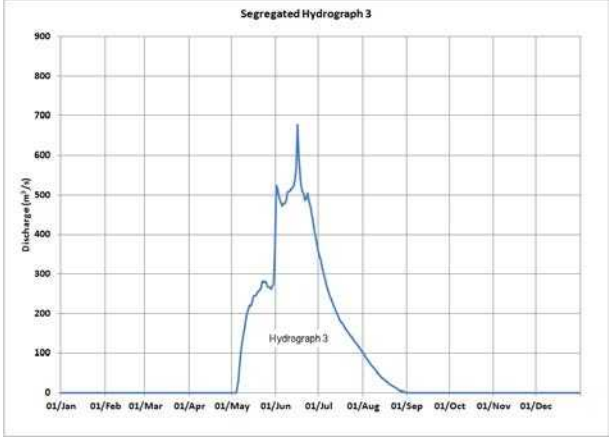


Figure 5.13
Assiniboine River Segregated Hydrograph 1

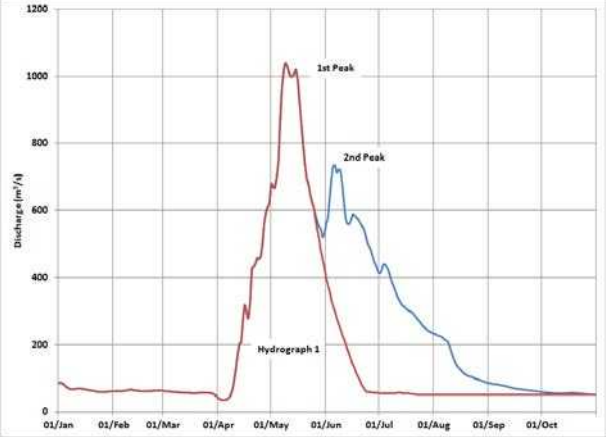
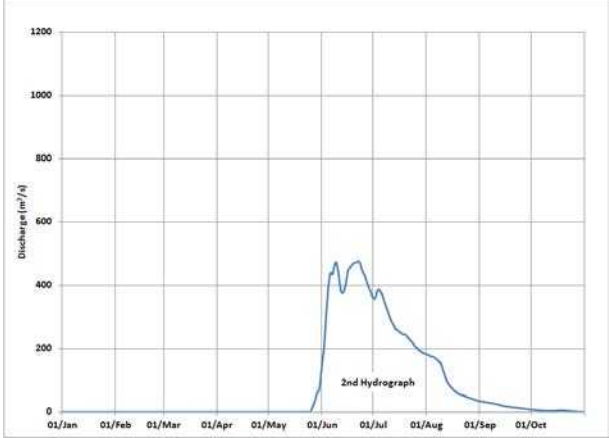


Figure 5.14
Assiniboine River Segregated Hydrograph 2

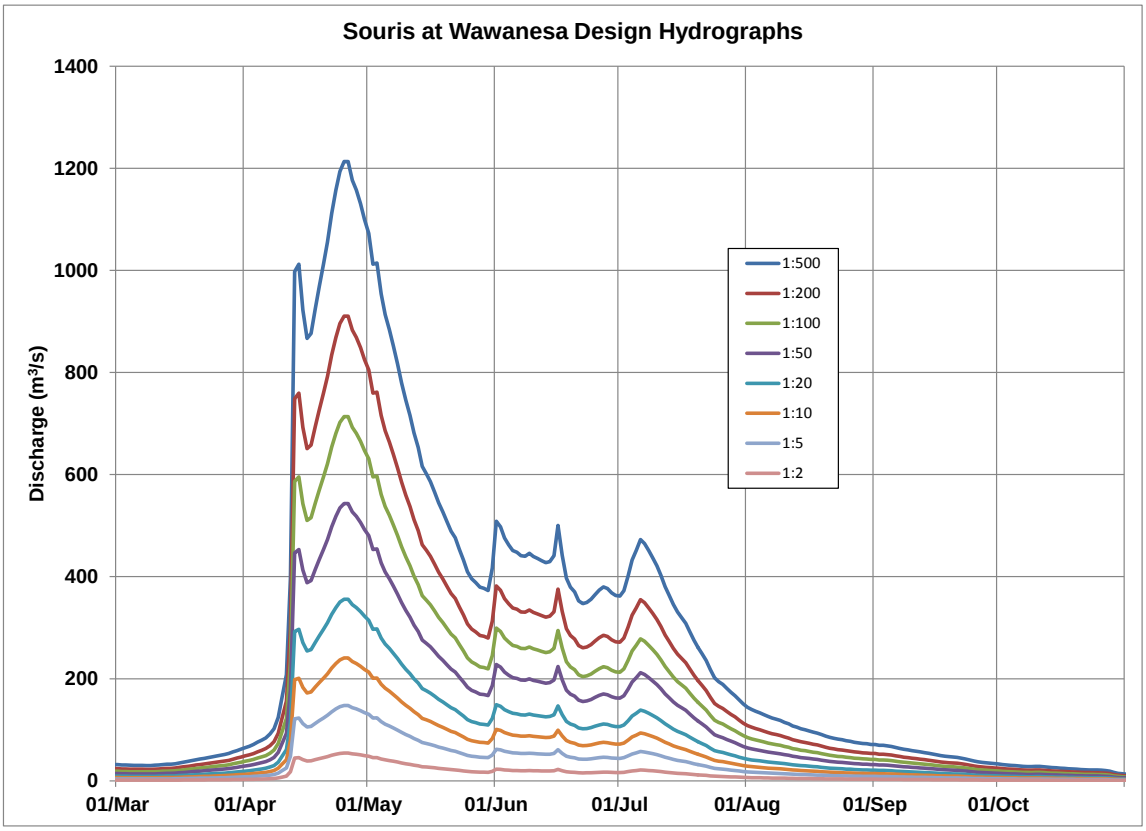


Note: Flows can be converted to cfs by multiplying metric values by 35.3.

- The segregated hydrographs that represented runoff due to snowmelt (Hydrograph 1 in Figures 5.11 and 5.13) were scaled to match the 1:500 year discharge event (provided in Section 5.2) to produce the 1:500 year peak for the Assiniboine River at Brandon and the Souris River at Wawanesa.
- The segregated hydrographs representing rainfall (Hydrographs 2 and 3 in Figure 5.11 and 5.12 and Hydrograph 2 in Figure 5.14) were added to the scaled hydrographs from Step 3 to produce the combined flood event hydrographs.
- The combined hydrographs were then scaled to match the peak discharges to the different flood frequencies provided in Section 5.2.

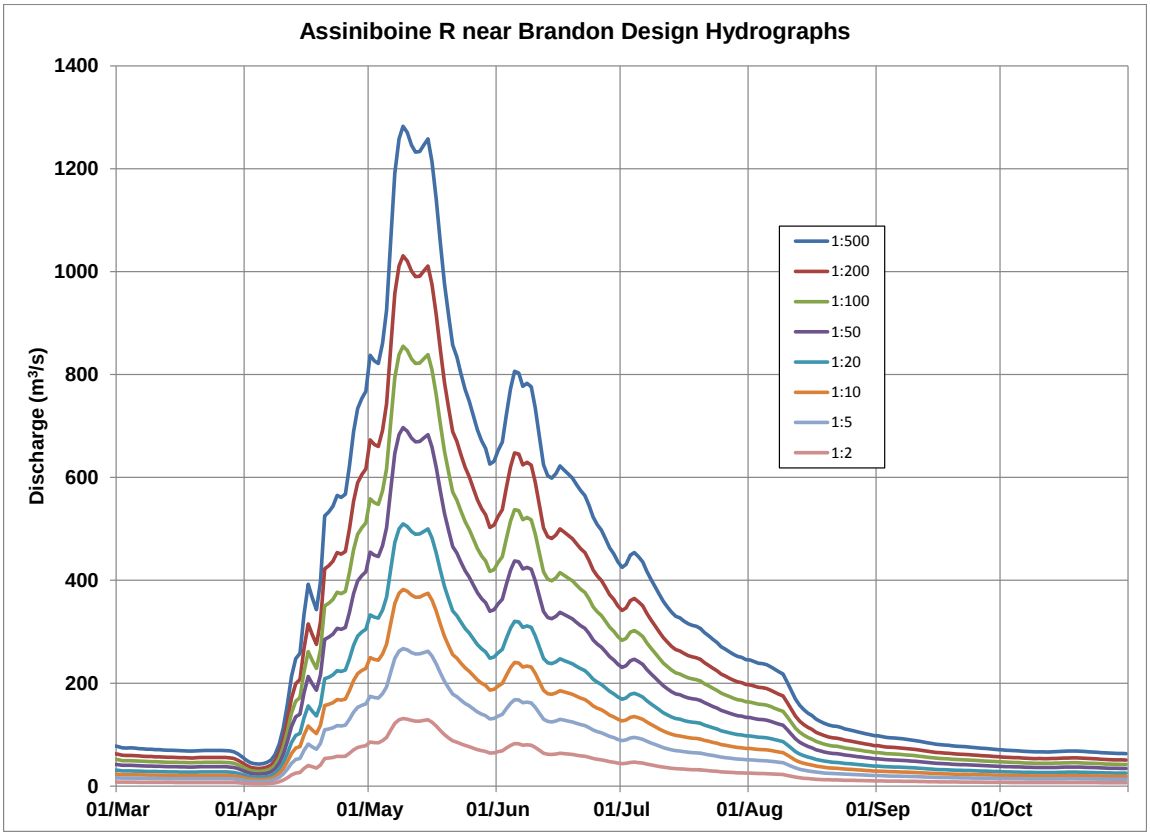
- The volumes for each of the hydrographs were compared to the different runoff volume frequencies provided in Section 5.2. The segregated hydrographs representing rainfall were then scaled so that the volume from the combined hydrographs corresponded to the different runoff volume frequencies. The final hydrographs for the Souris River at Wawanesa and the Assiniboine River near Brandon are provided on Figures 5.15 and 5.16. The volume of the design hydrographs compared to the runoff volume frequencies are provided on Figures 5.17 and 5.18.

Figure 5.15
Souris River at Wawanesa – Design Flood Hydrographs



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 5.16
Assiniboine River near Brandon – Design Flood Hydrographs



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 5.17
Souris River at Wawanesa – Flood Volume Frequency Curve

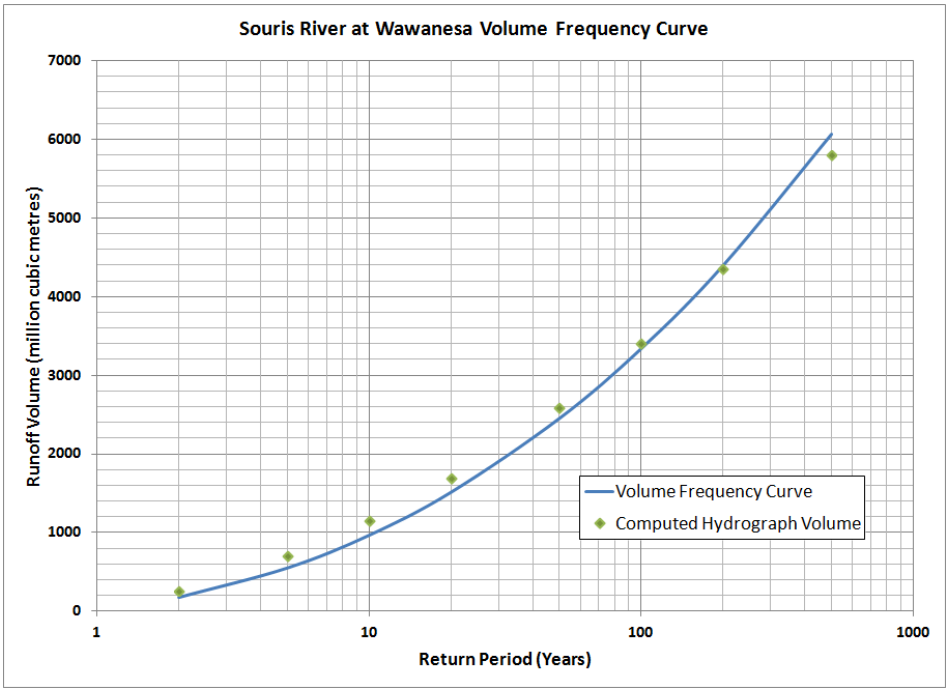
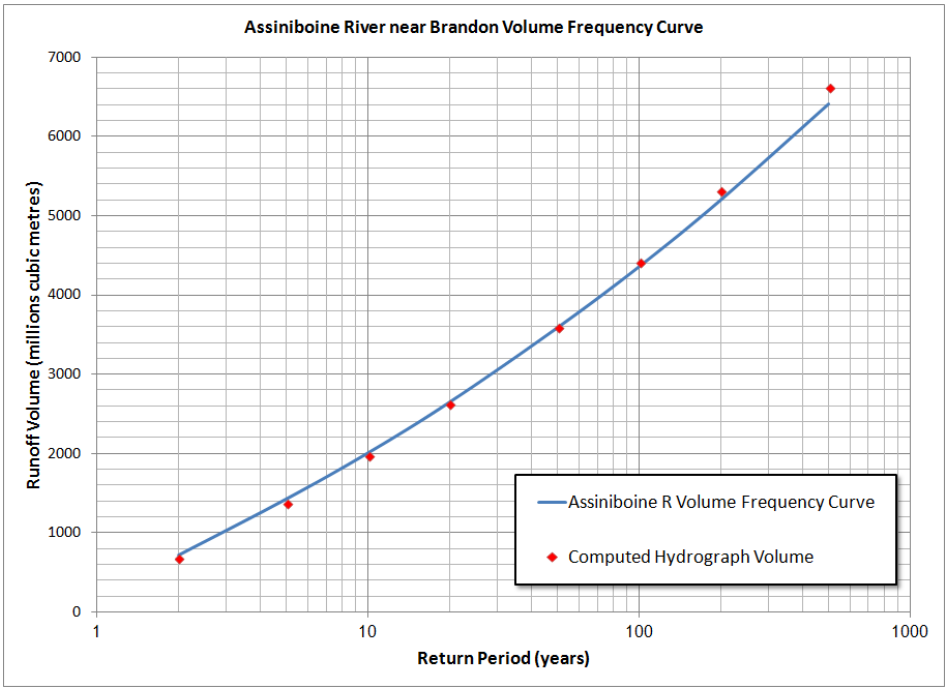


Figure 5.18
Assiniboine River near Brandon – Flood Volume Frequency Curve



- The flood hydrographs at Holland were then generated by adding the flood hydrographs at Brandon and Wawanesa, and were then scaled such that the computed hydrograph would agree with the value of the peak discharge from the frequency relationships at Holland.

- The volume of the hydrographs at Holland was then compared to the various runoff volume frequencies provided in Section 5.2. This is shown on Figure 5.19. The volumes were in agreement and, therefore, no further adjustments were necessary. The final hydrographs for the Assiniboine River near Holland are provided on Figure 5.20.

Figure 5.19
Assiniboine River near Holland – Flood Volume Frequency Curve

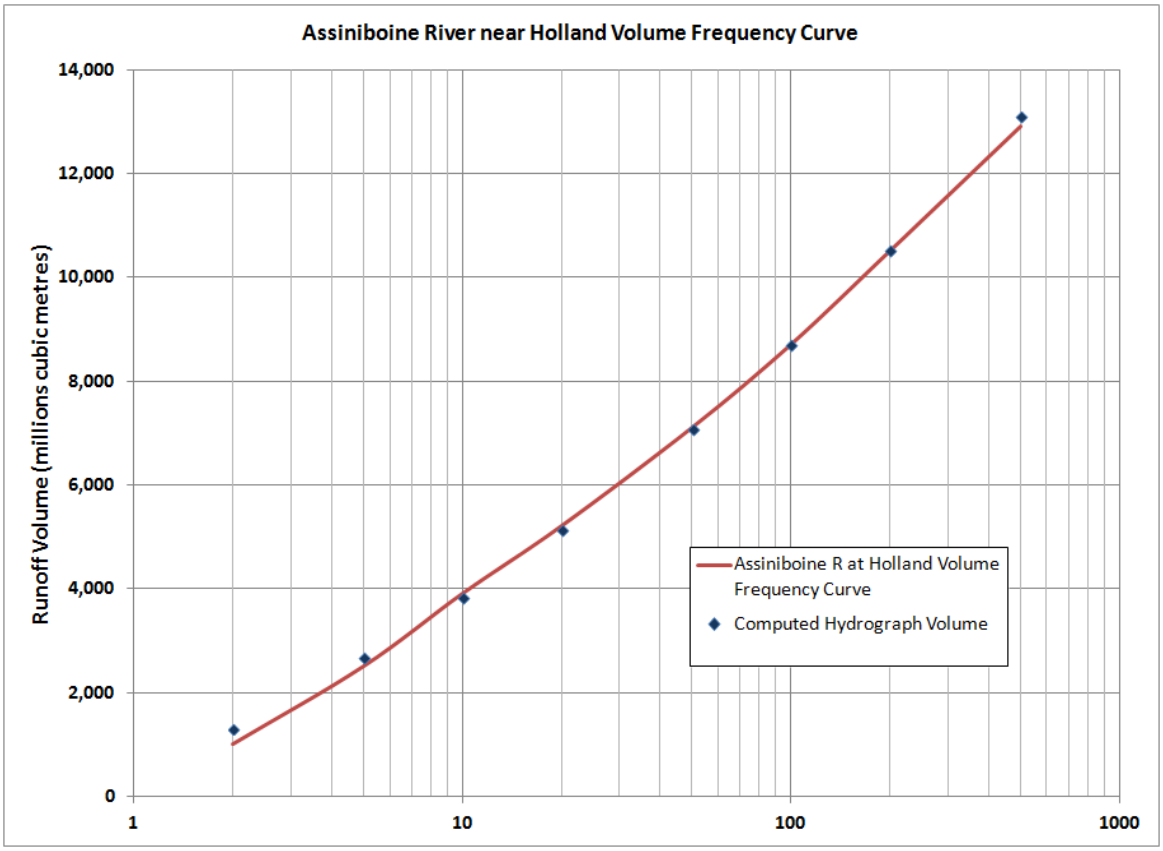
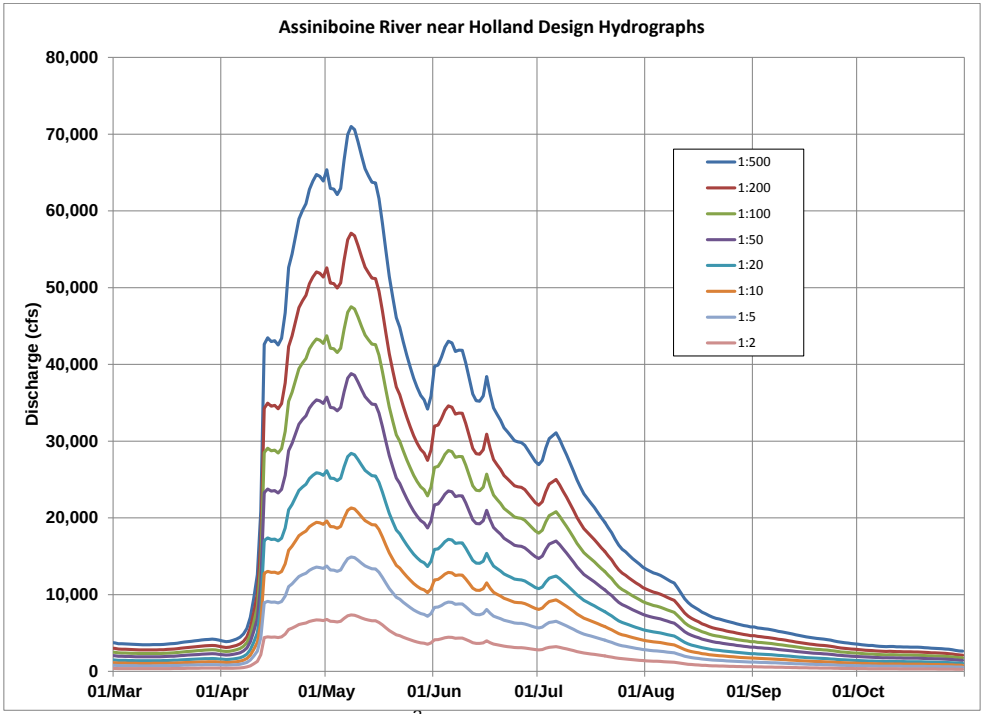


Figure 5.20
Assiniboine River near Holland – Design Flood Hydrographs



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

In addition to the various comparisons outlined above for the volumes and flows of the various design hydrographs to the computed frequency curves outlined in Section 5.2, graphics were prepared showing flood peaks vs flood volumes at the three locations where design hydrographs were developed. As shown on Figures 5.21, 5.22 and 5.23, the relationship of peak flows vs flood volumes from the design hydrographs were very similar to the period of record. This supports the conclusion that the design hydrographs are representative and appropriate for this level of study.

Figure 5.21
Assiniboine River near Brandon
Volume vs Daily Average Peak Discharge

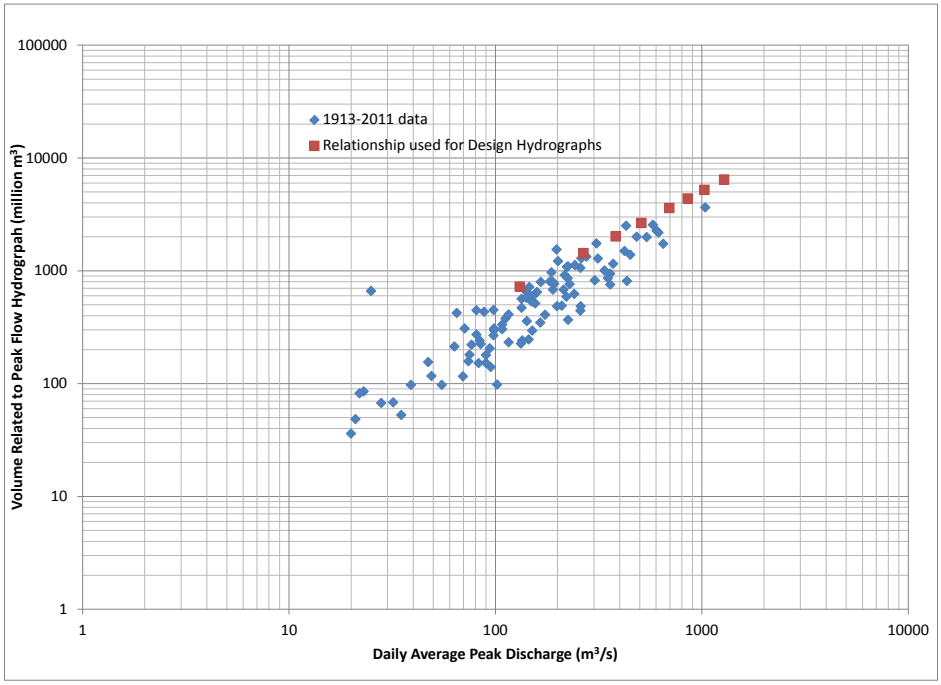


Figure 5.22
Souris River at Wawanesa
Volume vs Daily Average Peak Discharge

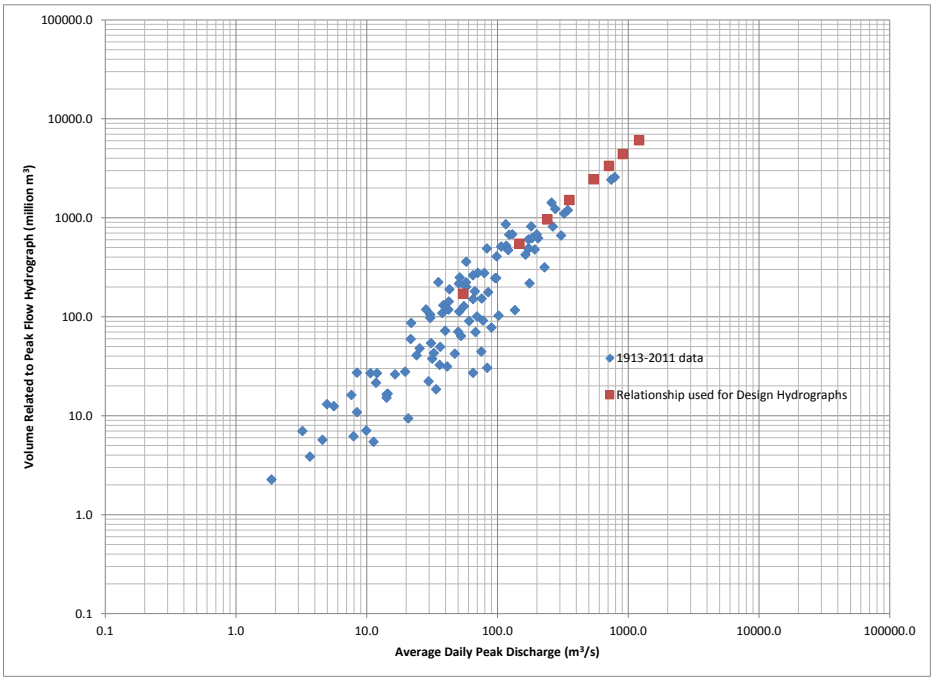
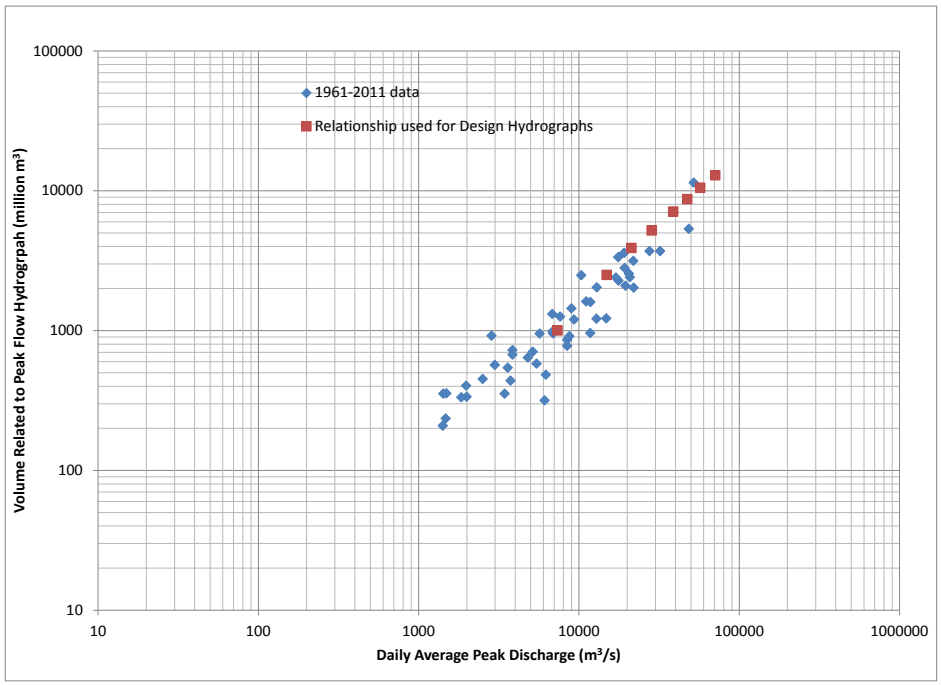


Figure 5.23
Assiniboine River near Holland
Volume vs Daily Average Peak Discharge



5.4.2 Conclusions

The design hydrographs developed for this study are typical for the Assiniboine River and Souris River during large hydrologic events. The annual peak discharge used in frequency analyses is taken as the recorded maximum daily discharge from all events in a given year. Conversely, the volume of the hydrograph corresponds to the total volume from multiple events if they occur subsequently.

The Souris River hydrographs can be scaled based on drainage area if required to represent Souris River conditions in Manitoba upstream of Wawanesa. The Assiniboine River hydrographs near Brandon can also be scaled based on drainage area if required to represent Assiniboine River conditions between Brandon and St. Lazare.

DRAWINGS

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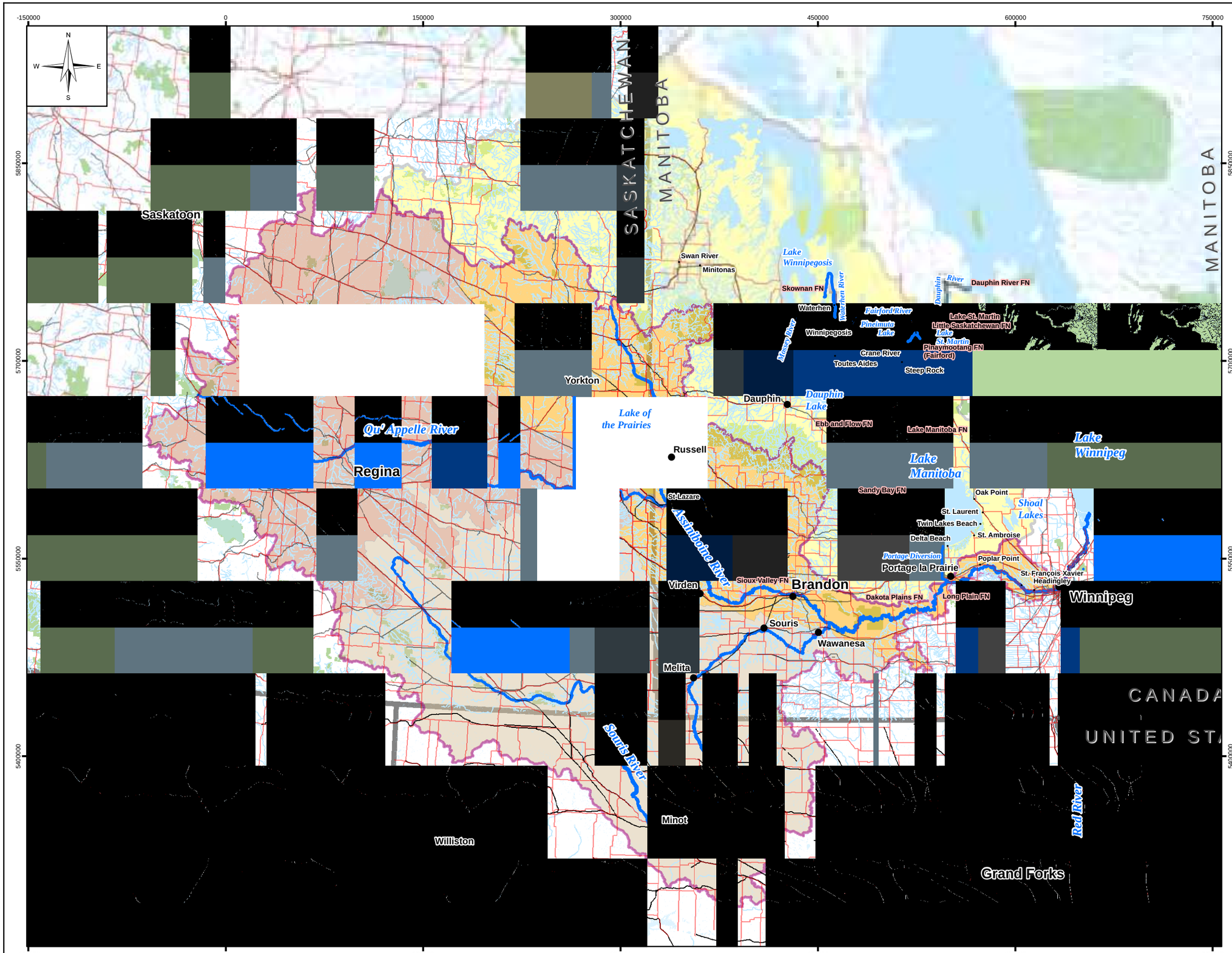

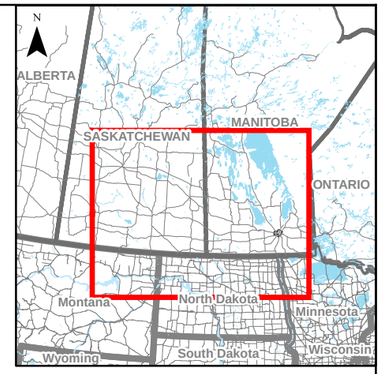

LEGEND:

- Minor Community
- Major Community
- Major Highway
- Railway
- Major River/Channel/Diversion
- Watercourse
- Waterbody
- Assiniboine River Basin
- Lake Manitoba Basin
- First Nation
- Provincial or National Park
- Province or State

DRAFT

25 0 25 50 75 100

Kilometres

SCALE: 1:2,750,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

MAP OF ASSINIBOINE RIVER & LAKE MANITOBA BASINS

JANUARY 2016 | DWG 5.1 | REV: 0

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11"x17" PLOT SCALE 1:1

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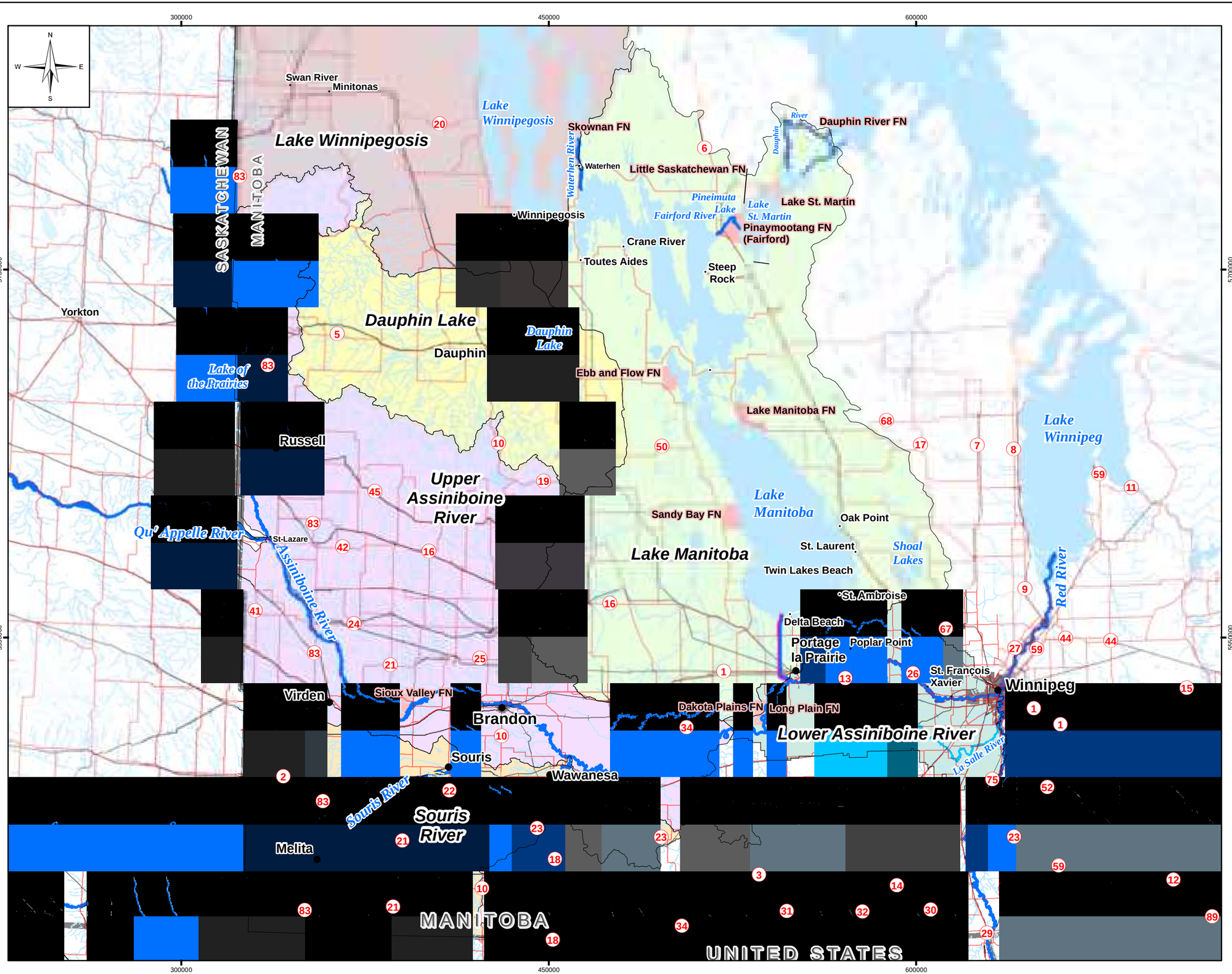

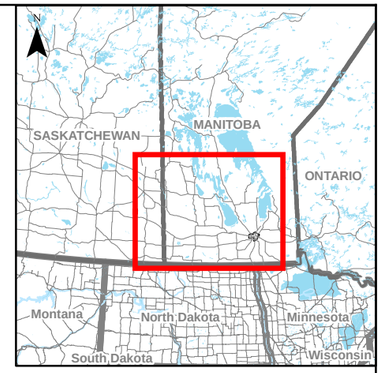

LEGEND:

- Major Highway
- Railway
- Major River/Diversion/Channel
- Watercourse
- Waterbody
- First Nation

BASINS

- Dauphin Lake
- Lake Manitoba
- Lake Winnipegosis
- Lower Assiniboine River
- Souris River
- Upper Assiniboine River

20 0 20 40 60
Kilometres

SCALE: 1:1,500,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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KGS GROUP CONSULTING ENGINEERS

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AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

AREAS OF THE ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

JANUARY 2016 | DWG 5.2 | REV: 0

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6.0 OPTIONS FOR FLOOD MITIGATION

6.1 INTRODUCTION

As described in Section 1.0, a thorough evaluation of a wide array of diverse options to improve flood protection was carried out during the course of this study. The options were based on a variety of influences:

- The experience and judgement of the Study Team Members.
- The original terms of reference.
- Suggestions by the Technical Advisory Committee.
- Input from the experts' workshop on vulnerabilities and mitigation options.
- A literature review by KGS Group.
- Frequent discussions with stakeholders throughout the study process.

The options evaluated and discussed in this report are:

- Installation of gates on the Shellmouth Dam.
- Diking the Upper Assiniboine River Valley.
- Land purchases along the Upper Assiniboine River Valley.
- Diking in the City of Brandon.
- Diking in the communities of Souris and Melita.
- Six different large storage reservoirs located on the Assiniboine and Souris River Basins.
- Twenty-one small dams on the Assiniboine and Souris River Basins.
- Three options for upgrades to the Lower Assiniboine River provincial dikes.
- Extension of the Lower Assiniboine River provincial dikes.
- Individual flood proofing, neighbourhood diking and purchase of flood prone lands downstream of the Lower Assiniboine River provincial dikes.
- Individual and community flood protection in the Lower Assiniboine River.
- Emergency diversion at the Hoop and Holler Bend.
- Various options to increase the capacity of the Portage Diversion.

- Various options to eliminate the Portage Diversion Failsafe.
- Construction of outlet channels on Dauphin Lake.
- River channel improvements in the Mossy River.
- Storage opportunities upstream of Dauphin Lake.
- Diversion channels from Lake Winnipegosis to Cedar Lake and Lake Winnipeg.
- Construction of a control structure on the Waterhen River.
- Storage upstream of Lake Winnipegosis.
- Various flood mitigation options for Shoal Lakes.
- Six options for increased outlet capacity from Lake Manitoba.
- Two options for increase outlet capacity from Lake St. Martin.
- Development controls.
- Modifications to land use.
- Restoration of wetlands.

A range of capacities was evaluated and varied depending of the option. In total, over 70 different measures were evaluated and are shown on Drawing 6.1 (located at the end of Section 6.0).

Each of the measures listed above are described in detail in Sections 9.0 through 20.0, and are ordered in the report by geographic location. These options were initially considered as individual projects acting in isolation, and economic evaluations of the options were undertaken. Following the review of the relative effectiveness of the individual options, combination(s) of attractive options and their economic performance(s) were considered. This is discussed further in Sections 22.0 and 23.0.

6.2 SPECIAL CONSIDERATIONS

6.2.1 Viable Projects in Upstream Jurisdictions

The terms of the reference requested that a review of possible projects on river tributaries upstream of Manitoba be carried out. KGS Group initially reviewed the available literature on

such projects and discussed their potential with knowledgeable sources in those areas. The following are salient points that emerged from that review:

- There are no existing projects such as reservoirs or channels in Saskatchewan that could be modified in operation to better benefit Manitoba in any significant way.
- There are no new projects in Saskatchewan that appear to have any potential for benefiting Manitoba. One exception was considered to be the Victor Dam on the Qu'Appelle River. This project has been addressed in further detail and is described in Section 9.5.
- Controls on development of drainage works in Saskatchewan are complex issues and have been addressed and discussed in Section 21.0.
- Restoration of wetlands is also a complex issue, whether the wetlands are located in Manitoba or in upstream jurisdictions. The issue is addressed in further detail in Section 20.0.
- There are potential reservoir sites on the upper Souris River in North Dakota that could be developed to reduce flood potential for all points downstream. The US Army Corps of Engineers (USACE) prepared detailed reports in the mid-1960s on various options, the leading of which were the Burlington Reservoir and the Des Lacs Reservoir. Both options were reported by the USACE to have relatively high benefit / cost ratios and were promoted at that time. There was considerable resistance to these options from the environmental perspective; and ultimately, the US elected to support the Rafferty and Alameda Reservoirs in Saskatchewan. Those dams were built and have been operating for many years.
- It is possible that the rules of operation for the Darling Reservoir, or even expansion of the dam and reservoir, could benefit Manitoba. However, it is unlikely that Manitoba could have the influence on affairs in North Dakota to expect such a development to come to fruition in the near future.
- The City of Minot, North Dakota, is currently pursuing federal support for funding an extensive upgrade to dikes in Minot to enhance protection there. This would have no benefit for Manitoba; and, depending on the option that is ultimately selected, could actually have modest negative impacts on Manitoba.

6.2.2 Temporary Protection Measures

As a general rule, temporary measures are not as reliable as permanent measures for flood protection. Temporary measures almost invariably carry with them the risk that:

- The measure cannot be installed in time to prevent flooding, or
- Hasty construction can lead to failures that could inflict more damage than would have occurred if the measures were not in place.

For these reasons, temporary measures are rarely considered as a long-term flood mitigation strategy to achieve a pre-determined flood protection standard such as the 1:200 year event or the highest flood on record defined by the Province of Manitoba. They are usually only considered in emergency situations during extreme flood events when the capacity of the permanent measures in place are at risk of being exceeded or when the infrastructure in place can no longer provide a reliable level protection on their own. However, there may be locations where physical limitations, environmental and socio-economic concerns, or high costs preclude permanent protection works to achieve a flood protection level that meets the provincial standard. In those locations, temporary works can be economical and effective to extend the flood protection level.

Upon review of the current level of protection in the different areas of the study (Section 4.2), it became apparent that only the Lower Assiniboine River was clearly deficient in achieving the provincial flood protection level criteria of 1:200 years or the highest flood on record. However, experience from the 2011 and 2014 floods demonstrated that temporary measures should not be relied upon as a long term flood mitigation strategy in this area. The risk of failure of these works at relatively low discharges, high frequency events, is clearly too high to be sustainable over the longer term.

Four other areas were identified in Section 4.2 as having programs or commitments in place to protect infrastructure to the highest flood on record that exceeds the 1:100 year event but not the 1:200 year event. These are:

- Dauphin Lake.
- Brandon.
- Melita.
- Souris.

Permanent measures have been identified in the report in all four of these areas to increase the level of protection to the provincial criteria of at least 1:200 years. However, temporary measures could also be considered in those areas if the cost of the options or environmental and socio-economic concerns were deemed unacceptable by the Province.

Notwithstanding the risks associated with temporary measures, they can be effective when installed correctly and planned for appropriately to extend the level of protection of permanent

measures for low frequency events beyond the 1:200 year criteria. Examples where temporary measures could prove to be economical would be:

- Temporary dikes on the Assiniboine River dikes – where extensions are planned and placed on top of dikes designed with an adequate top width and appropriately engineered structures, the risk of failure would be relatively low and the benefits significant. Since these temporary upgrades would occur at a frequency of less than 1:200, the cost of temporary protection would be low in the long term. This would be similar to the approach adopted by the City of Winnipeg Primary dikes where it has been concluded that raising the primary dikes is prohibitively expensive, and a plan to provide temporary clay dike extensions is deemed the most cost effective means of providing enhanced protection for extreme events.

- Community dikes and individual dike protection – temporary measures could also be used to increase the protection for communities and individuals during extreme events. This would be similar to the plans to top up secondary dikes in the City of Winnipeg with clay or sandbags.

- Wave protection on Lake Manitoba – with the ongoing programs and commitments in place to increase the flood protection level around Lake Manitoba, as well as with the preferred permanent measures identified in this report, the flood protection level on Lake Manitoba will exceed the 1:200 year event. Increasing the protection beyond this level with permanent works has been demonstrated to be impractical or prohibitively costly. However, temporary protection for extreme events to protect residences and property would be cost effective. The effectiveness of geotubes was demonstrated in the fall of 2011 and again in 2014 when water levels were high on the lake. As a temporary measure, these are an effective means of increasing the flood protection for extreme events on Lake Manitoba.

DRAWINGS

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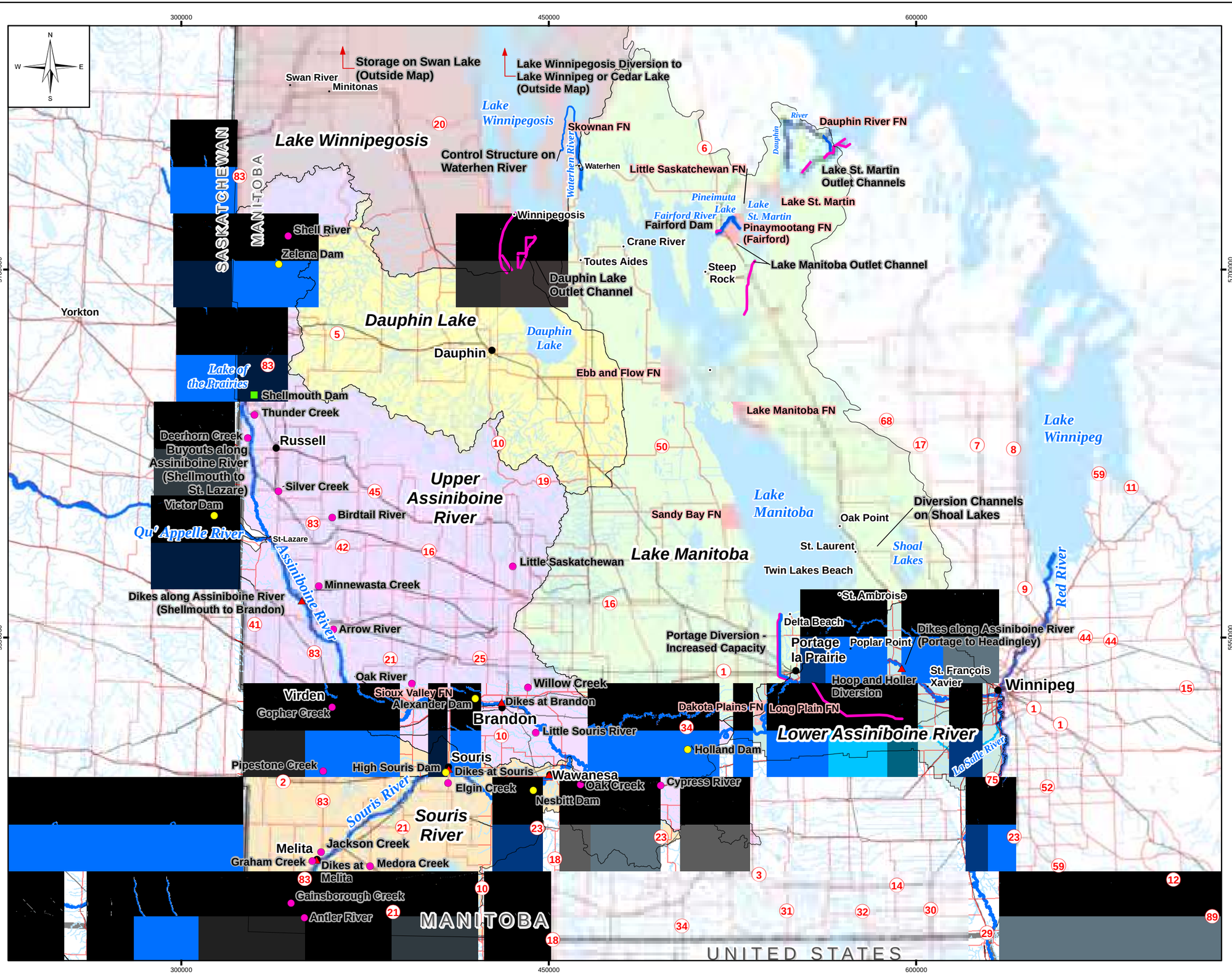

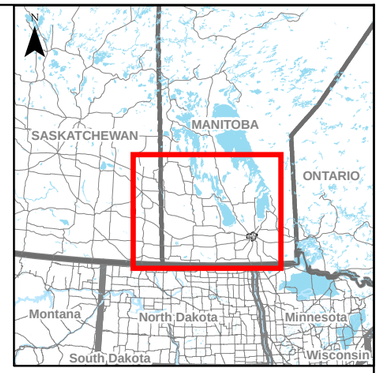

LEGEND:

- Major Highway
- Railway
- Major River/Diversion/Channel
- Watercourse
- Waterbody
- First Nation

BASINS

- Dauphin Lake
- Lake Manitoba
- Lake Winnipegosis
- Lower Assiniboine River
- Souris River
- Upper Assiniboine River

Mitigation Options

- Existing Large Storage Reservoirs
- Other Options
- Proposed Small Dams
- Large Storage Reservoir
- Diversion & Outlet Channels

NOTE:
1. The following mitigation options are not specific to a single area and therefore not shown on the map:
   - Individual Flood Protection
   - Wetland Restoration
   - Development Control and Land Use Changes
2. All units are metric and in metres unless otherwise specified. Transverse Mercator Projection, NAD 1983, Zone 14. Elevations are in metres above sea level (MSL)

20 0 20 40 60
Kilometres

SCALE: 1:1,500,000 METRIC 11"x17"

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

MAP OF FLOOD MITIGATION OPTIONS

JANUARY 2016 | DWG 6.1 | REV: 0

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11"x17" PLOT SCALE 1:1

7.0 BASIS FOR ECONOMIC EVALUATIONS OF OPTIONS FOR FLOOD MITIGATION

7.1 INTRODUCTION

A wide variety of flood protection works are being considered in the Assiniboine River & Lake Manitoba Basin (AR&LMB) Study. A list of options has been devised and economic analyses for at least the most effective options have been carried out. This section summarizes the basis upon which the economic assessments and comparisons were made.

7.2 AVERTED DAMAGES

The underlying principle for economic assessment of a flood protection measure in this study is the estimation of the damages due to flooding that would be averted by that measure. In simple terms, the benefits are the damages that would occur if the protection measure were not in place, minus the damages that would occur if the protection measure were constructed and operated effectively. Flood damages considered include, but are not necessarily limited to:

- Cost of pre-emptive works or actions that would be deployed if the protection measure were not implemented including military assistance, temporary protection measures such as sandbags, cleanup, and evacuation of flooded areas.
- Repairs / replacements for direct physical damages to residential, commercial and industrial buildings and contents, as well as damages to adjacent developments such as wells, septic fields, gardens, fences, walkways, and outbuildings.
- Temporary relocation and subsistence costs for affected residents.
- Temporary services may be required until direct physical damages have been repaired (e.g., hauled water or septic).
- Repair / replacement of damaged vehicles that could not be removed from the inundated zone in time.
- Agricultural losses including:
    - Repair or replacement of agricultural structures such as barns, silos, granaries.
    - Repair or replacement of damaged farm equipment and vehicles.
    - Direct damages to pre-existing crops.

- Reduction (or elimination) of income from crops due to delay in access to arable land.
- Purchase of feed needed for cattle or other farm animals displaced from flooded pasture land.
- Rental costs of pasture land for displaced cattle or other farm animals (in lieu of purchased feed).
- Repair of eroded arable land, or acknowledgement of lost future income from irreparable damages to land.
- Miscellaneous damage to crops or arable land such as de-nitrification or soil salinization due to standing water (may be multi-year effect) and clean-up of flood deposited debris.

- Miscellaneous impacts on farm infrastructure including, for example:
  - Effects on timely access to non-flooded fields. Because the farmer may be busy dealing with the flood emergency, crops in non-flooded fields are not planted and/or managed in a timely manner (e.g., seeding, spraying, cutting hay). Machinery, cropping supplies, fuel, and other vital services such as facilities for making repairs and performing regular maintenance to farm equipment may be isolated within a ring dike surrounded by flood waters and, therefore, inaccessible, which means wasted inputs and/or more costly and less timely alternatives.
  - Many farm families have off-farm jobs, therefore, they need to maintain this separate source of income despite a flood event. The employer may or may not be able to release the employee to deal with the flood emergency on the farm.
  - The farm families may need to be relocated from the flooded area even when they are protected by a ring dike, since it is common for the electricity and gas to be shut off and/or because there is excessive danger from potential breaches of the dike(s).
  - Emergency care of livestock, if located within the ring dike. This includes:
    - Feed may be inaccessible for intensive operations such as hog or poultry barns.
    - Mature hogs or chickens may need to be delivered to market.
    - For a dairy operation, milk needs to be delivered regularly several times a week.
    - For a feedlot or a cow / calf enterprise, regular feeding and care is needed but will be more difficult, time-consuming and costly during a flood event.
    - The farmer may also have a livestock enterprise located out of the flood zone. In such an event, routine care and management may be much more difficult because of limited access.
- Livestock that is either relocated or isolated within a ring dike, such as hogs or dairy animals, would be more subject to reduced weight gain and/or increased risk of disease.

- Additional costs due to interruption of normal transportation routes in and around the affected area(s), resulting in additional travel costs for local residents. School bus routes, access to medical facilities, etc. would be affected.

Repair / replacement of damaged public infrastructure, including institutional buildings (government offices and buildings, churches, schools, etc.), and public facilities such as sewage treatment plants, water treatment plants, power generation plants, electrical distribution equipment and systems, communication equipment and systems, roads, parks, etc.

Consideration was also given to the relative effects of intangible aspects such as stress and anxiety by acknowledging the number of people affected, the duration of stress imposed and generally comparing the event to other catastrophes.

InterGroup Consultants completed a literature review and discussed this subject with senior personnel who had recent experience working on other similar projects to determine if the academic research and/or practitioner experience has changed markedly since the Red River Floodway environmental assessment process.

Although research is progressing to better assess and understand the potential or realized stress and anxiety effects associated with flooding, the subject continues to be debated. As such, InterGroup was unable to single out a proven or defendable quantitative model applicable to the current study.

Stress and anxiety was previously considered as part of the Flood Protection Studies for Winnipeg, and stress associated with flooding was found in families at two levels: 1) immediately after the flood, and 2) for those with issues prior to the flood, they had limited capacity to cope or manage in the flood disaster resulting in long-term family crisis. This stress appeared to increase and was more chronic and long-standing for those families who felt the damages they suffered were the “result of flooding done to them” (artificial flooding), as opposed to an “act of nature”.

In addition to having physically related consequences, flood damage and the threat of flood damage are sources of stress and anxiety for property owners and residents. Stress and anxiety are important impacts and often serve as indicators of the severity of a flood event. For

residents and property owners, primary sources of stress from a flood event are the potential and actual damages to people and property. The challenge in quantifying this variable is the fact that stress and anxiety and its measurement is highly dependent on their individual perceptions and abilities to cope.

It is important to note that people engaged in event response and fighting the rising waters are also subject to stress and anxiety. Decision-makers and emergency service providers such as municipal officials, provincial agency representatives, federal agency representatives and the staff of non-profit organizations experience stress from the demands of their duties during flood events.

Given the challenges in quantifying this form of flood event specific damage (including other damages such as business interruption losses) into a stage-damage model, an additional percentage has in the past been included to reflect this uncertainty. For the current study, similar to the Flood Protection Studies for Winnipeg, an additional allowance equivalent to 20% of the building and agricultural damage estimates was included to account for stress and anxiety.

Typical units of measure and typical unit prices for damages of the types described above are listed in Table 7.1. Typical unit prices for damage types have been quantified from experience gained in 2011, or other recent flood events, and adjusted for escalation where appropriate. The typical values were also updated based on feedback from MIT and local stakeholders. Note that the final methodology selected and level of detail for the estimation of flood damages varied depending on the data available for a given area or flood protection option. This is discussed in greater detail in the flood damage model sections of the report.

Table 7.1
Typical Value of Damages

Item Description	Unit of Measure	Typical Value
Flood response and fighting costs (e.g., pre-emptive works, temporary diking, military response, etc.)	$ per structure or $ per discharge – varies depending on available data	Relationship developed based on data from 2011 Flood
Buildings and structures (e.g., residential, commercial, industrial, institutional, etc.)	% of replacement value or % of assessed value as a function of flood depth	Typically 40-150% of assessed (or replacement) value for cases with water levels above first floor
Damage to public infrastructure and utilities (e.g., streets, roads, ditches, culverts, etc.)	$ per submerged linear km of road	$12,000 to $22,000, varies based on total length of road flooded
Damage to crops planted prior to flooding (e.g., wheat, canola, oats, barley, flaxseed, etc.)	$ per hectare as a function of expected yield – highly variable depending on type of crop	$800 on average
Damage to crops planted prior to flooding – specialty crop (e.g., potatoes, onions, etc.)	$ per hectare – highly variable depending on type of crop	$10,000 on average
Delay in crop seeding leading to reduction in crop yield	% of expected yield loss – highly variable depending on type of crop and duration of flood	50 -100% of an average season – case by case basis
Forage crop losses and grasslands	$ per hectare – can be for more than one year depending on duration of inundation	$300
Other miscellaneous agricultural damages (e.g., de-nitrification or soil salinization due to standing water, and cleanup of flood deposited debris, loss in crop inventory)	% allowance of total agricultural damages	50%
Interruption of transportation – rail line	$ per hour	$10,000 – secondary line $100,000 – mainline
Interruption of transportation – road – fuel costs	$ per km per vehicle	$0.50
Interruption of transportation – road – driver time and other delays	$ per hour per vehicle	$50
Stress and anxiety	% allowance of agricultural and building damages	20%
Business losses	% allowance of total damages	5%
Other unidentified additional damages	% allowance of total damages	0 - 25% - case by case basis

Loss of income due to temporary closure of businesses in the inundated zone is real. However, it is a debatable component of flood damages from the perspective of economic assessment. In the U.S., for example, the official position of the government (through planning and economic ranking of potential flood protection works by the USACE) is that business losses are not

permitted in economic evaluations. The theory from the perspective of the country as a whole is that business losses attributed to the flood event are almost certainly compensated by increased production and income from comparable businesses in other unaffected jurisdictions in the country. So, from the viewpoint of the justification on a country-wide basis and selection of the most deserving projects for federal support, this component is not deemed to be valid. This was also the approach supported by the Independent Joint Commission (IJC) and the Province of Manitoba in the estimation of damages that were considered in the justification for the expansion of the Red River Floodway.

For the current AR&LMB Study, the Province of Manitoba has directed that business income loss be included in the analysis but also tracked separately in case it must be removed for a future federal funding submission.

Loss of income from agricultural production was treated separately and differently from loss of income from other businesses. Loss of crop production is unlikely to be readily compensated by increased production from other areas of the country during the same growing season. It should, therefore, be deemed a loss to the country as a whole. As such, agricultural losses were deemed as valid components of flood damages that must be considered in benefit / cost analyses.

7.3 BASIS OF ESTIMATING AVERTED FLOOD DAMAGES

The methodology selected for estimation of the financial implications of averted flood damages depended on the level of accuracy deemed appropriate for the protection measure under study. Some measures justified only a cursory examination in the preliminary screening process. Other measures justified more detailed approaches. Each of four optional levels of assessment was available for application, depending on the circumstances for the particular protection measure:

1. Estimates of damages based on judgment or anecdotal evidence of historical events and damages incurred in the region of interest or in nearby regions with similar characteristics.
2. Estimates based on per capita, or per unit area, statistics for known events in inundated zones in this or similar regions.
3. Methodical estimation of damages using GIS maps and databases combined with inundation limits estimated by hydraulic models.

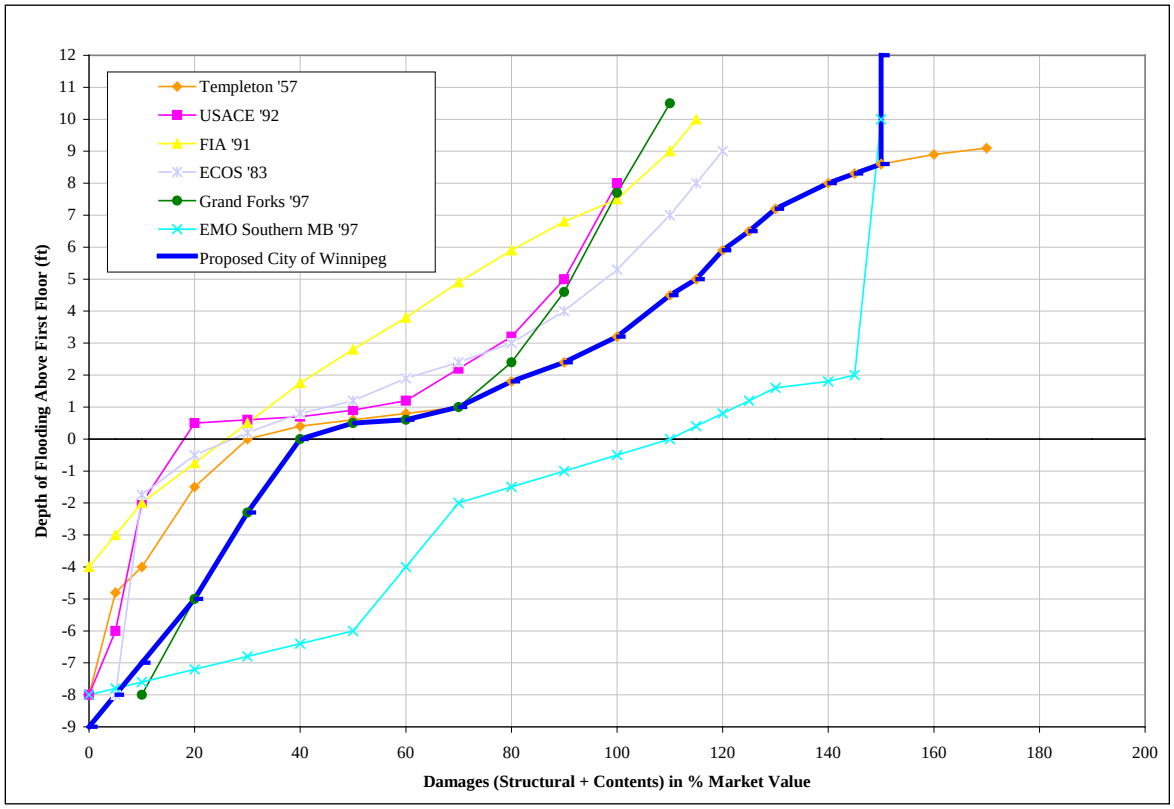
4. In instances where detailed data was available and the protection measure appeared to have sufficient justification, application of principle of risk and uncertainty embodied in the USACE software "Flood Damage Analysis" (FDA). Details of the methodology and principles applied in that software are available on the USACE website.

Selection of the appropriate method to deploy was made on a case by case basis, largely influenced by the availability of data. The current value of damage estimates prepared as part of previous studies were calculated based on appropriate inflation data. As a minimum, the adjustment from past years was equal to the Consumer Price Index (CPI) as documented by the Bank of Canada.

7.4 DEPTH-DAMAGE RELATIONSHIPS

A key tool in assessing potential flood damages to residential or rural structures and their contents is the application of a depth-damage relationship. This relates the damages to a residential structure as a fraction of its assessed or replacement value, dependent on the depth of flooding incurred at the structure. A variety of relationships were evaluated by KGS Group in the Flood Protection Studies for Winnipeg. A preferred relationship that captured the best of several data sources was developed and supported by the IJC, the Province of Manitoba and the City of Winnipeg. The relationship that was ultimately applied in that study for Winnipeg is shown in Figure 7.1. These values were used as required for structures in the study areas of the AR&LMB.

Figure 7.1
Proposed Depth-Damage Relationship for Residential Structures



Depth-damage relationships for commercial and institutional structures and contents were developed by KGS Group for the Flood Protection Studies for Winnipeg in 1999 (KGS Group, 2000c). These values were updated to represent current dollars and used as required in the AR&LMB Study.

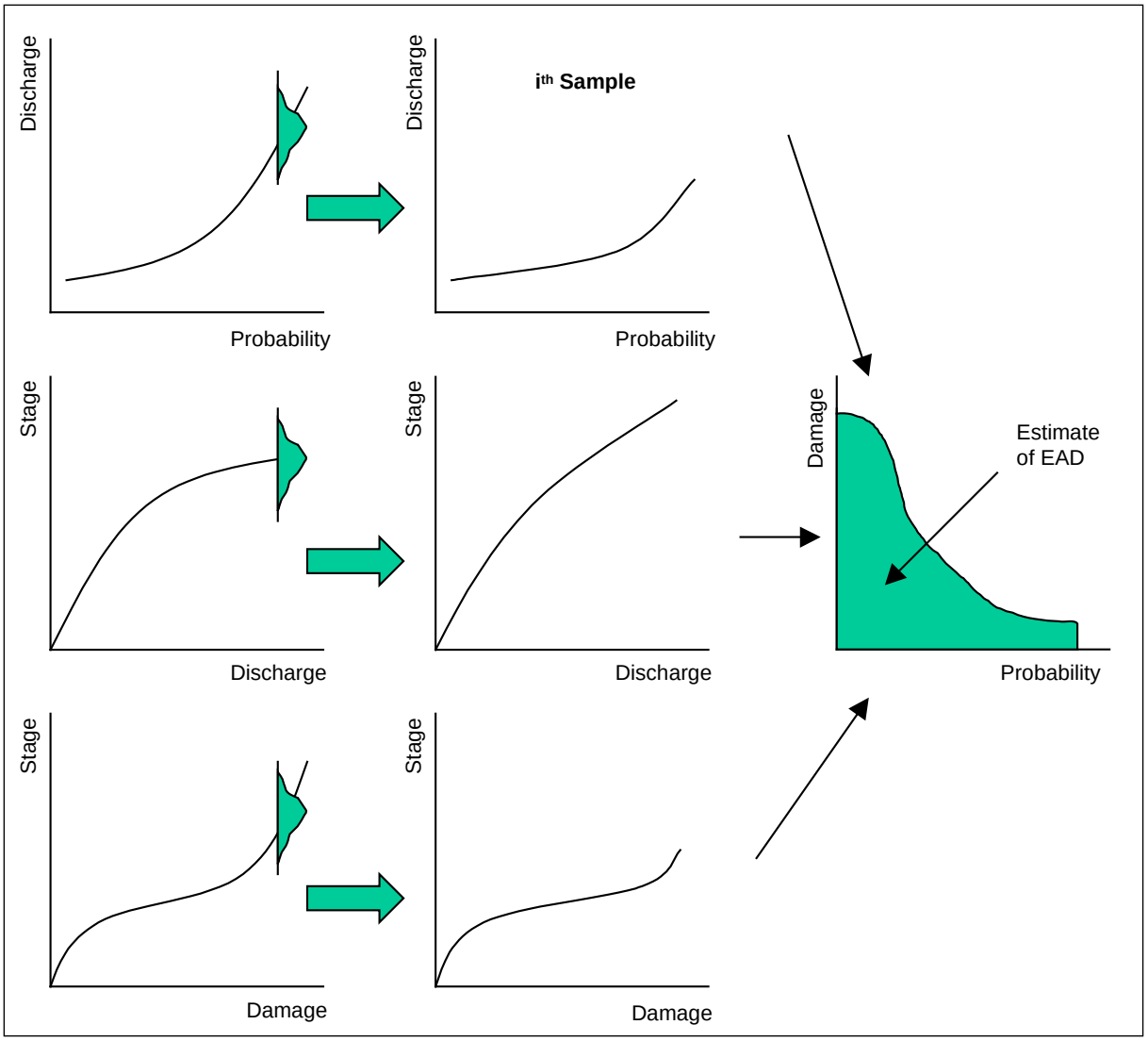
Damages to other facilities and infrastructure were based on data for those items, as appropriate. Damage estimates by KGS Group in the studies for the IJC in 1999 (KGS Group, 2000b) were also referenced.

7.5 EXPECTED ANNUAL DAMAGES

The calculation of Expected Annual Damages (EAD) for each scenario, with and without a particular mitigation scheme, is a cornerstone of the estimation of benefits. Figure 7.2 demonstrates the principles of derivation of EADs for a particular condition.

Once the EAD for a particular scenario is developed for the existing condition (i.e., with no flood protection or with an existing level of flood protection), it is used as a comparative basis for other EADs computed with each case having various mitigation options included. The differences between these represent EADs that can be analysed in a consistent fashion across the range of mitigation options being considered. The full range of flood potential from moderate floods to floods at or in excess of the 1:500-year flood must be included in this process.

Figure 7.2
Derivation of Expected Annual Damages



7.6 TIME HORIZON

All projects were evaluated on the basis of a 50-year time horizon extending into the future after the completion of the construction of the proposed measure under study. Historical flood records were deemed to be representative of river flows and flood potential in the future (this is further discussed in Section 8.0).

Residual damages (present value of damages after the 50-year time span) were considered in some circumstances, if the life of the measure was clearly expected to be considerably longer than 50 years, and if that aspect could affect comparisons with other more short lived projects.

7.7 PERIOD OF FLOOD RECORD

The basis of the flood damage frequency curves shown in Figure 7.1 were the historical records of flood flows. These were variable in length depending on the data available for the area under study. As previously discussed in Section 5.2.1 of this report, the primary basis for estimation of EADs was the full extent of flood records available for a site or region where a flood protection option was being considered. However, a sensitivity analysis was also performed for several options using shorter, more recent records of floods that reflect the recent few decades of wet conditions.

7.8 COST ESTIMATION

Preparation of costs was done on a consistent basis using unit prices representative for current construction. Table 7.2 outlines the key unit measures and prices used in this study. These are considered base costs for typical conditions; therefore, premiums were added to some of the estimates to reflect unique conditions such as constructing dikes within a marsh, abnormal hauling distances, widening an existing control structure, etc. The unit prices included mobilization / demobilization, but did not include indirect costs such as engineering, construction administration, owners' costs, and interest during construction (if applicable). These costs and this list were refined and expanded as the study progressed.

Table 7.2
Basis for Estimating Costs at Conceptual Level of Study

Construction Item	Unit Price for Study Purposes
Common excavation	$10 per cubic metre
Common excavation in the wet	$50 per cubic metre
Dikes (borrow excavation & placement)	$20 per cubic metre
Rock excavation (limestone)	$25 per cubic metre
Rock Riprap (hauled from off-site quarry)	$90 per cubic metre
Revegetation with cover crop (grass & top soil)	$5,500/ha
Revegetation with native species (multi-year)	$8,000/ha to $15,000/ha
Control / Drop Structure (stoplogs, no bridge)	$10,000 per square metre (foot print area)
Bridge for PTH (no approaches or traffic control)	$7,000 per square metre (area of deck)
PTH Bridge approaches & traffic control	$3,000,000 lump sum
Bridge for municipal roads	$2,500 per square metre
Railway bridge (single track, 3.35 m deck width)	$10,000 per square metre
Railway bridge approaches & traffic control	$4,000,000 lump sum
Permanent access road – single-lane gravel	$750,000/km
Land acquisition / buyouts – non-arable land	$1,000 per hectare
Land acquisition / buyouts – arable land	$5,000 per hectare
Land acquisition / buyouts – prime arable land (irrigated for specialty crops such as potatoes)	$10,000 to $15,000 per hectare
Acquisition / buyouts of residences, farms, etc.	100-150% of assessed value
Environmental mitigation	0-15% of direct construction costs
Indirect costs (engineering, administration, approvals, owner's costs, etc.)	20% of direct construction costs
Contingency	30% of direct and indirect costs

Environmental mitigation costs included activities such as long-term monitoring, mitigation of wells affected by a project, compensation for damage to fish habitat, compensation to fishermen or any additional mitigation and compensation measures that may come from Regulator requirements determined during the advancement of the licensing and permitting process. These costs were estimated as a percentage of the direct construction costs with the percentage ranging from 0 to 15% relative to the potential environmental effects as follows:

- Negligible effort for environmental mitigation expected – part of contingency. (e.g., new dikes, upgrading of dikes in current location)

- Minor environmental mitigation costs expected – 5% of direct costs.
  (e.g., new outlet channel, diversion channel widening)
- Moderate environmental mitigation costs expected – 10% of direct costs.
  (e.g., new outlet channel / diversion with inter-basin transfer)
- Major environmental mitigation costs expected – 15% of direct costs.
  (e.g., river expansion / dredging, new large storage reservoir)

Costs associated with proposed best practices to mitigate effects during construction, such as control of surface water and groundwater (dewatering, ditching, silt fences and settling ponds) have been included as part of the unit price for excavation.

The standard top width for construction of dikes was assumed to be 4 m for all mitigation projects unless noted otherwise. Calculated areas for land acquisition included an additional allowance for the purchase of some complete parcels that could be bisected by a construction project right-of-way.

The current value of cost estimates prepared as part of previous studies was calculated based on appropriate inflation data. As a minimum, the adjustment from past years was equal to the Consumer Price Index (CPI) as documented by the Bank of Canada. However, construction costs typically exceed the CPI since it represents the price level of a “basket” of consumer goods and services. For example, excavation costs are heavily influenced by fuel prices. Therefore, the adjustment from past years was based on the Electrical Utility Construction Price Index (EUCPI) for dams, reservoirs and waterways, as documented by the Bank of Canada from 1956 to 1997. The average escalation rate over this period was approximately 6%. Since the EUCPI was terminated after 1997, the previous costs were escalated from 1997 onwards by utilizing an average annual escalation rate of 5%. This was considered conservative since escalation rates have decreased based on a review of construction projects completed after 1997.

7.9 PRESENT VALUATION

Present values of future EADs were computed using accepted economic principles with selected interest rates and with consideration for future potential growth in the area under study and inflation of costs over the time horizon.

Existing structures may require modification or replacement as part of a proposed flood mitigation project. The analysis considered that existing structures would require rehabilitation, even if no expansion / extension / rehabilitation of the works went ahead, due to the existing structures being part way through their useful lifespan. Therefore, the cost of the improvement schemes considered that there would have been spending needed even if the improvement had not occurred. Similar to previous studies such as the Floodway Expansion, this was addressed by performing a life cycle cost analysis to identify the present worth of various improvements or the "do nothing" options in this study.

Future operation and maintenance costs for flood protection works was estimated to be 1% of the direct construction costs (1% annually for the life of the project, calculated as present value).

7.10 RATES OF INTEREST / INFLATION / GROWTH

The parameters used in the economic evaluation of flood protection options in the AR&LMB Study were the same as for the studies of flood protection for Winnipeg in 1999 (KGS Group, 2000c). At that time, the following values were adopted by the three levels of government involved:

- Interest – 7% per year.
- Inflation – 3 % per year.
- Future Growth of Population and Economic Activities in the Region – 1% per year.

For comparison, studies done by the USACE for Fargo Moorhead (USACE, 2011) used an interest rate of 4.25% and did not acknowledge the effects of inflation (inflation rate of zero, as is required by US Government policy).

It should be noted that for the purposes of the estimation of present values, the difference between interest and inflation is the most important parameter. For example, the variation in present values from combinations of 5% interest and 1% inflation, to 9% interest and 5% inflation is nominal. The difference between interest and inflation has historically been dominantly about 4%.

7.11 SENSITIVITY ANALYSES

Economic analyses for flood mitigation options were based on the procedures described above. However, for a short list of preferred options, sensitivity analyses were carried out to address the following ranges of key parameters:

- Interest 5 to 10% per year.
- Inflation 2 to 4% per year, but with recognition of a spread between interest and inflation of 3 to 5%.
- Growth – zero to 2% per year.
- Time horizon 40 to 75 years.

7.12 ECONOMIC INDICATORS

Two parameters were computed for each mitigation scheme:

1. B/C ratio (i.e., present value of benefits divided by present value of costs).
2. Net benefits (i.e., present value of benefits minus present value of costs).

8.0 NUMERICAL MODELS

8.1 GENERAL

The process of evaluating flood mitigation alternatives for this study was complicated due to great differences in flood vulnerability and flood frequency across the Assiniboine River and Lake Manitoba watersheds. This was further complicated by the spatial variability of flood causal factors, which could be quite different from one flood to another and for different magnitudes of floods. At locations where only relatively short hydrometric records are available, the reliability of frequency analysis can be affected, especially for major events, which usually plot as outliers. Furthermore, flood hydrographs for rivers can display great differences in peak flow versus flood volume from one event to another, complicating selection of design hydrographs. Flood protection schemes are highly variable depending on local topography, channel capacities, shoreline characteristics, exposure and stage-damage relationships in general.

Many portions of the Assiniboine River & Lake Manitoba Basins display their own unique hydrology and flooding problems. Each study area was, therefore, treated separately with respect to flood protection schemes and the associated benefit / cost analysis.

This section identifies the key numerical hydrologic and hydraulic models used in the study, and explains their structure, assumptions and intended uses. Further details on the models have been described throughout this report as protection measures for each of the geographic areas was considered.

8.2 FLOOD ROUTING MODELS

8.2.1 Lake Winnipegosis

8.2.1.1 Introduction

Lake Winnipegosis is the major source of inflows to Lake Manitoba. It drains a watershed of 55,000 km^2 (21,200 mi^2) and has a surface area of 520,000 ha (300,000 ac). When water levels

are high on Lake Winnipegosis, the outflows through the Waterhen River are also high, and remain high often for long periods at a time. This often results in flooding on Lake Manitoba. Furthermore, Lake Winnipegosis has a small outlet relative to its drainage area. It can take months and even years for the lake to recede to its normal water level. During this period, the outflows through the Waterhen River remain high, resulting in prolonged periods of above normal inflows to Lake Manitoba. There is no control structure on the Lake Winnipegosis outlet, but during typical winter conditions ice in the Waterhen River reduces the flow capacity.

To estimate the effects of the flood mitigation alternatives located on Lake Winnipegosis later discussed in this report, the historic inflows to the lake were computed. The period of study selected was from 1961 to 2012. This included both wet and dry periods and coincided with the period when Lake Manitoba levels were regulated by the Fairford Water Control Structure. It should be noted that conditions in 2014 were not included in the model simulations since the analysis had already been completed prior to this period. Nonetheless, the effects in 2014 of the flood mitigation alternatives located on Lake Winnipegosis are discussed in Section 14.0 of the report.

8.2.1.2 Inflow Available for Outflow

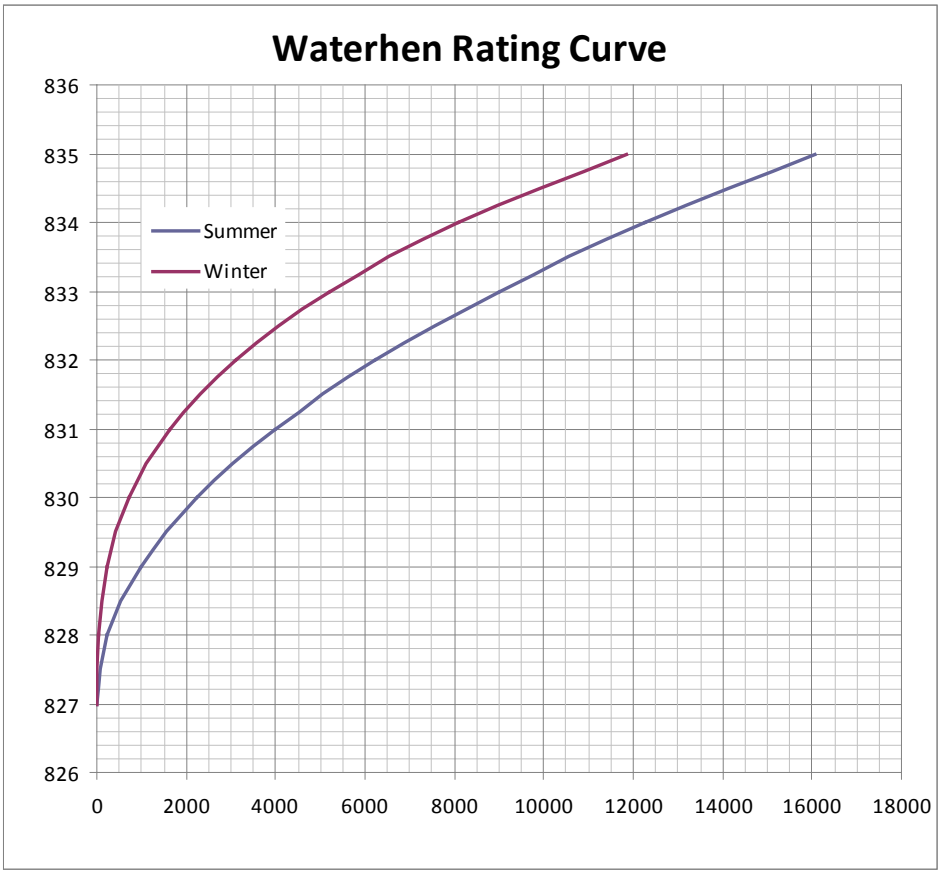
The Inflow Available for Outflow (IAO) method was used to compute historic inflows. This procedure is based on the continuity equation:

Inflow = Outflow + Change in storage

The daily outflows from Lake Winnipegosis have been recorded at Station 05LH005 Waterhen River near Waterhen since 1950. Daily lake levels have been recorded at Station 05LH001 Lake Winnipegosis at Winnipegosis since 1913. Because of the size of the lake, there are significant fluctuations in the recorded daily levels, which reflect wind effects rather than changing inflows. Elimination of this fluctuation was achieved in the computation of the IAOs by averaging the daily mean water levels over 15 days. This approximated the lake levels as they would have been without wind effects.

The computed daily inflows were then reduced by the amount being diverted from the lake, and the resulting flows were routed through the lake using the rating curves for the outlet at the Waterhen River. These curves related the recorded lake levels versus the recorded outflows. The relationship was stable during the open water period, but varied over the winter depending on ice conditions on the Waterhen River. For the current simulation, the winter rating curve was based on a best fit of all available winter lake level and river flow data. Figure 8.1 shows the estimated rating curves used in the numerical model.

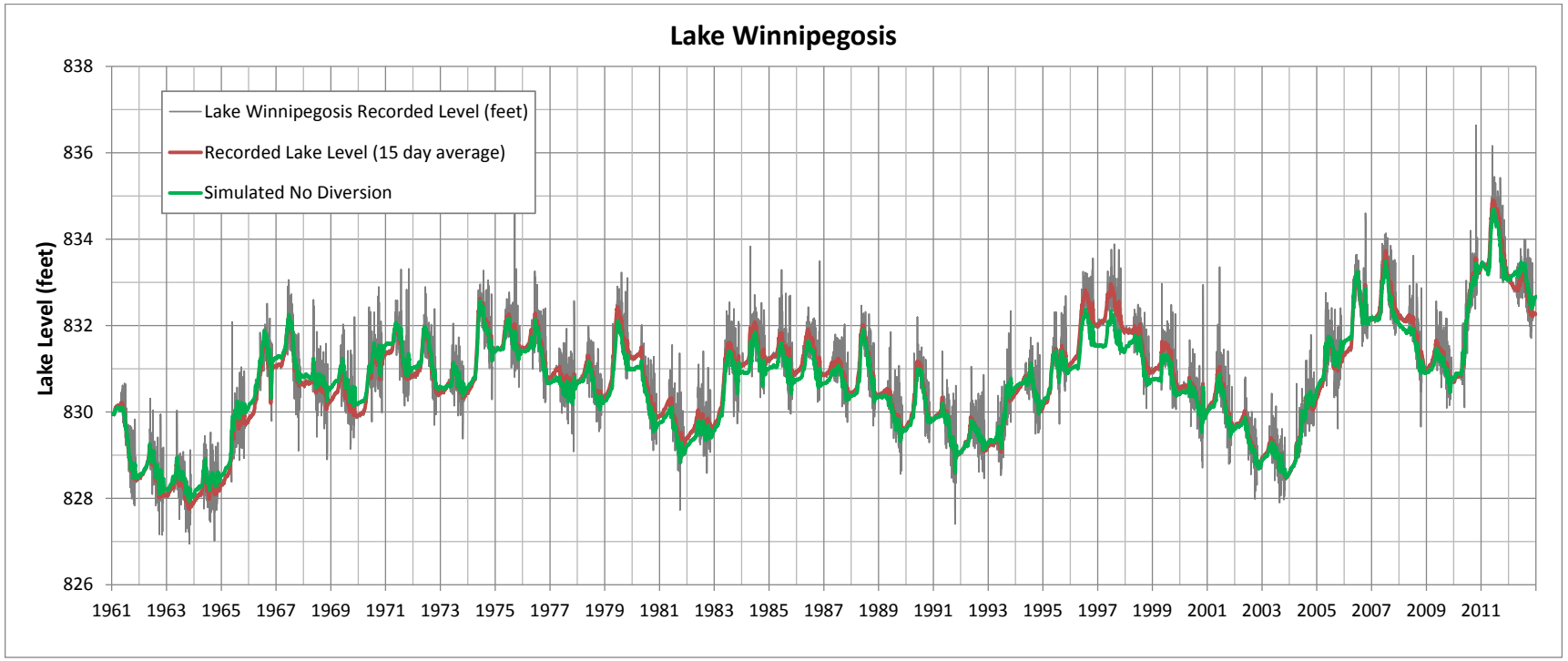
Figure 8.1
Outflow Rating Curve for Lake Winnipegosis



Notes: Water levels can be converted to metres by multiplying imperial values by 0.3048.
Flows can be converted to m^3/s by dividing imperial values by 35.3.

To test the accuracy of the simulation, the daily lake levels were simulated assuming existing conditions (i.e., no diversions or flood control structures). This should ideally result in a duplication of the recorded levels. The results are shown in Figure 8.2.

Figure 8.2
Lake Winnipegosis Simulated Historic Levels



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

The simulated levels generally matched the smoothed recorded levels. The deviations mainly occurred during the winter months because of changing ice conditions. The model was deemed to be suitably calibrated and verified to permit a good representation of the effects of changed conditions that would result from control schemes of various kinds.

Comparisons of lake levels under various diversion and storage schemes were based on the simulated rather than the recorded levels to eliminate the differences caused by changing ice conditions.

8.2.1.3 Extension of Lake Winnipegosis Routing Model to Downstream Lakes

To simulate the downstream effects of the potential flood mitigation alternatives located on Lake Winnipegosis, the Lake Winnipegosis routing model was extended through Lake Manitoba and Lake St. Martin. On Lake Manitoba, the model was developed to apply the current operating rules for the Fairford Structure.

Initially the structure was operated to maintain a constant water level of 247.55 m (812.17 ft) as often as possible. A minimum outflow of 14 m^3/s (50 cfs) was established to sustain a live stream downstream of the structure. However, over time it was recognized that stabilization of water levels on Lake Manitoba resulted in increased fluctuations of water levels on Lake St. Martin. It was also recognized that maintaining a stable lake level was damaging the shoreline marshes. In response to this, the Manitoba government adopted, in 2003, the “Minimal Log Change” that was recommended by the Lake Manitoba Regulation Review Advisory Committee. Under this operation, Lake Manitoba would be permitted to fluctuate normally between the levels of El 247.0 and El 247.7 m (El 810.5 and El 812.5 ft) with no stoplog adjustments. If the lake levels exceeded that range, the structure would be opened to controlled flooding. If the levels dropped below that range, the minimum outflow would be set to 22.7 m^3/s (800 cfs).

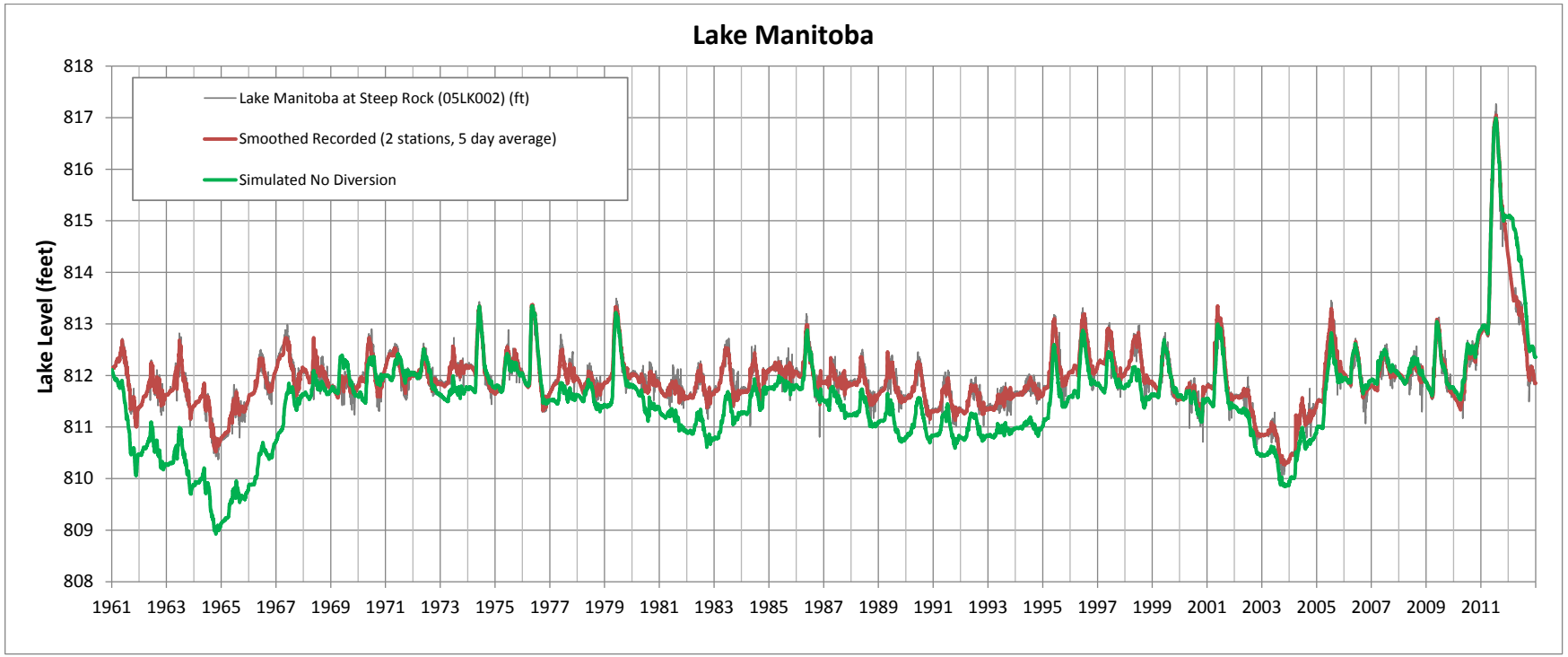
After the 2011 Flood, the operation of the control structure was again reviewed by the Lake Manitoba / Lake St. Martin Regulation Review Committee. They recommended two minor adjustments to the rules of operation:

1. After a period when lake levels have been outside the desirable range, permit the lake to reach the middle of the range before adjusting the stoplogs. For example, after a flood event, stoplogs should not be replaced until the lake has receded to mid-range or 247.3 m (811.5 ft). This would permit the marshes and beach ridges to re-establish.

2. Because the 2011 Flood was so extreme, the committee also recommended that the range of operation be reduced by half a foot to El 246.9 – 247.5 m (El 810.0 – 812.0 ft) for a period of five years. After that time, the range would revert to El 247.0 to 247.7 m (El 810.5 to 812.5 ft).

For the current study, the first recommendation was included in the operating simulations; but the range was set to El 247.0 to El 247.7 m (El 810.5 to El 812.5 ft) because this study was required to address the long-term effects. Also, the simulation did not include a permanent emergency channel from Lake St. Martin to Lake Winnipeg. Figure 8.3 shows the simulated water levels of Lake Manitoba under the current regime of operation.

Figure 8.3
Lake Manitoba Simulation of Minimum Log Change Operation



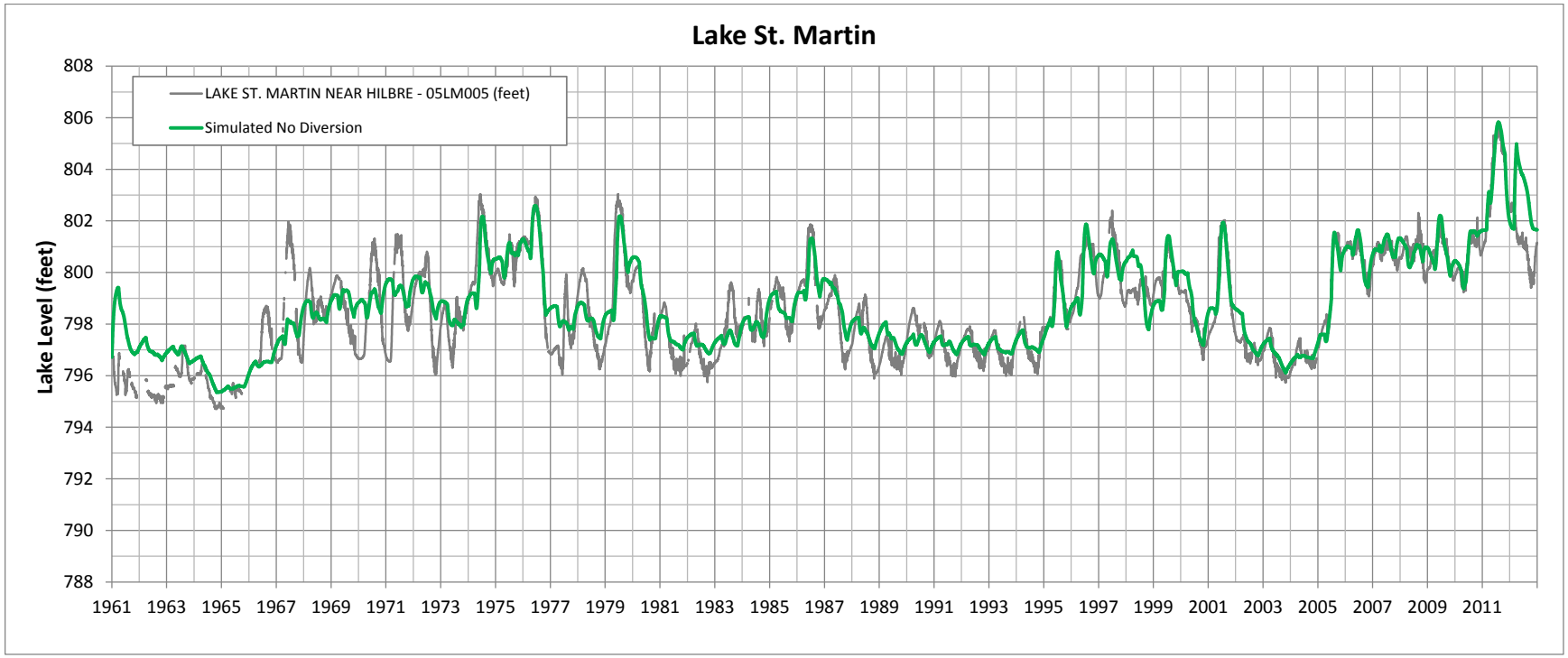
Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

The graph shows that the simulation matches the recorded levels more closely after 2003 when the current rules were adopted. The simulated levels tend to be lower than the recorded levels particularly during dry periods. The primary reason is that initially the minimum outflow was set at 1.4 m^3/s (50 cfs). After 2003, the minimum outflow was increased to 23 m^3/s (800 cfs). The effect of this increased minimum flow is most evident over the period from 1961 to 1965 when inflows were very low.

The simulated levels reproduce the 2011 Flood accurately, but the rate that the lake levels recede after the flood peak is more gradual in the simulation because the emergency outlet from Lake St. Martin is not included in the model.

There is currently no control structure at the outlet from Lake St. Martin. Therefore, the lake levels fluctuate with changing inflows based on the natural outlet rating curves. Figure 8.4 shows the simulated Lake St. Martin levels under current operating rules.

Figure 8.4
Lake St. Martin Simulation of Current Operation



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Again the simulated levels match the recorded levels from 2003 to the present, except for 2012. In the simulation, the emergency channel that was in place in 2012 was not included, so the benefits to Lake St. Martin were not achieved.

8.2.2 Lake Manitoba and Lake St. Martin

The Monte Carlo approach adopted and described in subsequent sections of this report was used to generate thousands of synthetic annual flood volumes for the Assiniboine and Lake Manitoba drainage basins. These volumes were consistent with the statistical characteristics of the historical record of floods. However, the synthetic volumes must, in turn, be analyzed to estimate the peak still water levels (wind effects eliminated) for Lake Manitoba that would contribute to governing the extent of flood damage for each event. The conversion from synthetically generated flood volumes to peak annual still water levels was carried out with a numerical model as described below.

8.2.2.1 Description of Flood Routing Model

A numerical model was developed for flood routing to analyze and compute the water levels on Lake Manitoba and Lake St. Martin and the outflows from these lakes. The numerical model was formulated as a spreadsheet and calculates daily changes in still water levels and outflows based on predetermined stage-discharge relationships at the lake outlets and on the available storage volume as a function of lake level. The model had as input the net inflow to Lake Manitoba based on flood hydrographs developed for local Lake Manitoba inflow. A discussion on the flood hygrographs is given in Section 8.2.2.2.

Stage-storage relationships used in the model were developed from measured surface areas of the lakes. The stage-storage relationships for Lake Manitoba and Lake St. Martin are provided on Figures 8.5 and 8.6, respectively.

Figure 8.5
Lake Manitoba Stage-Storage Relationship

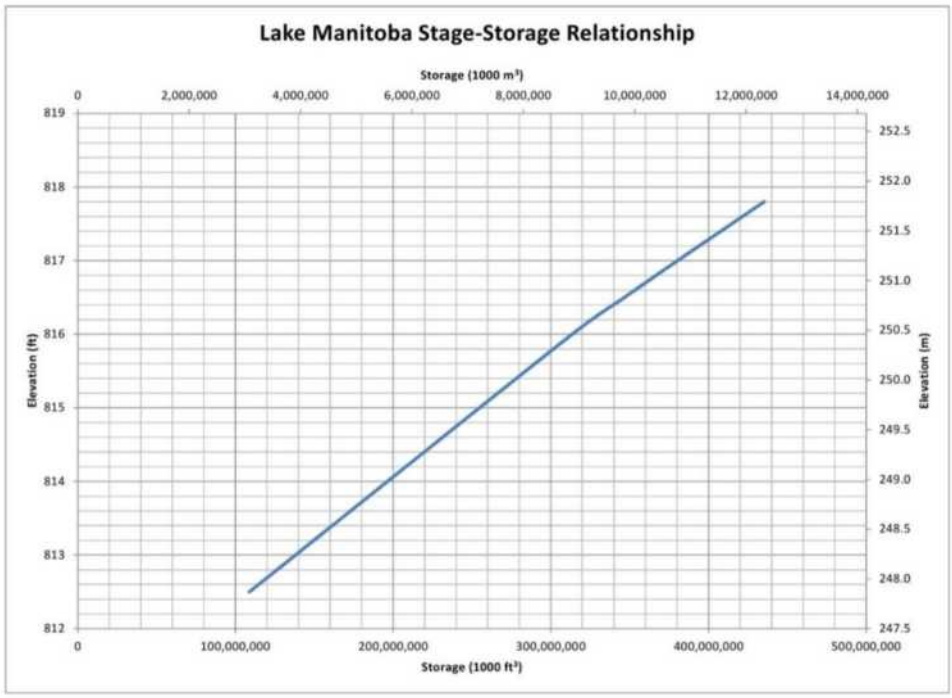
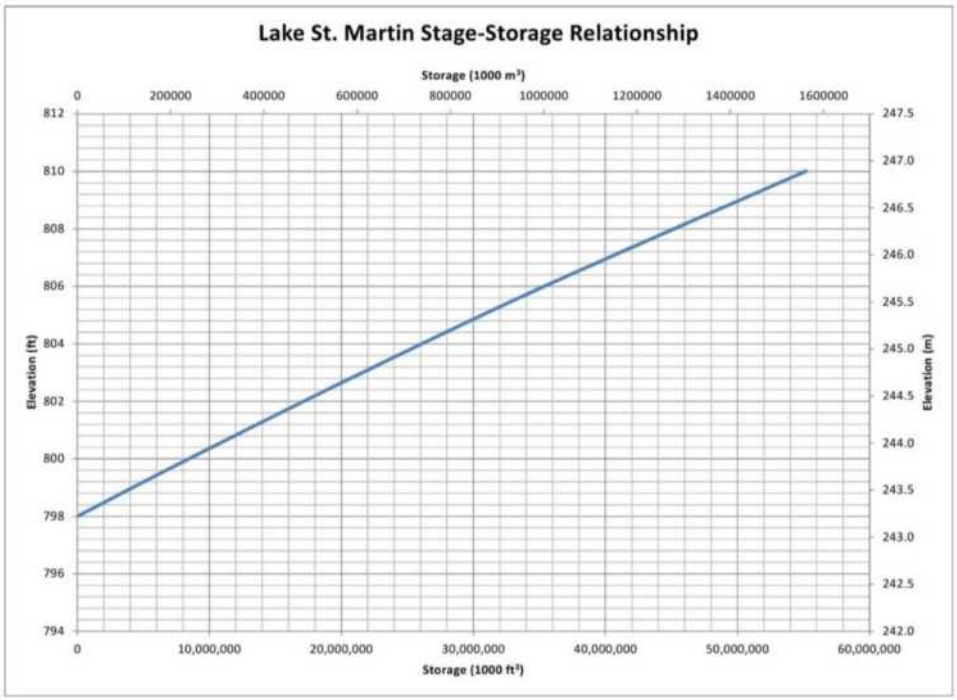
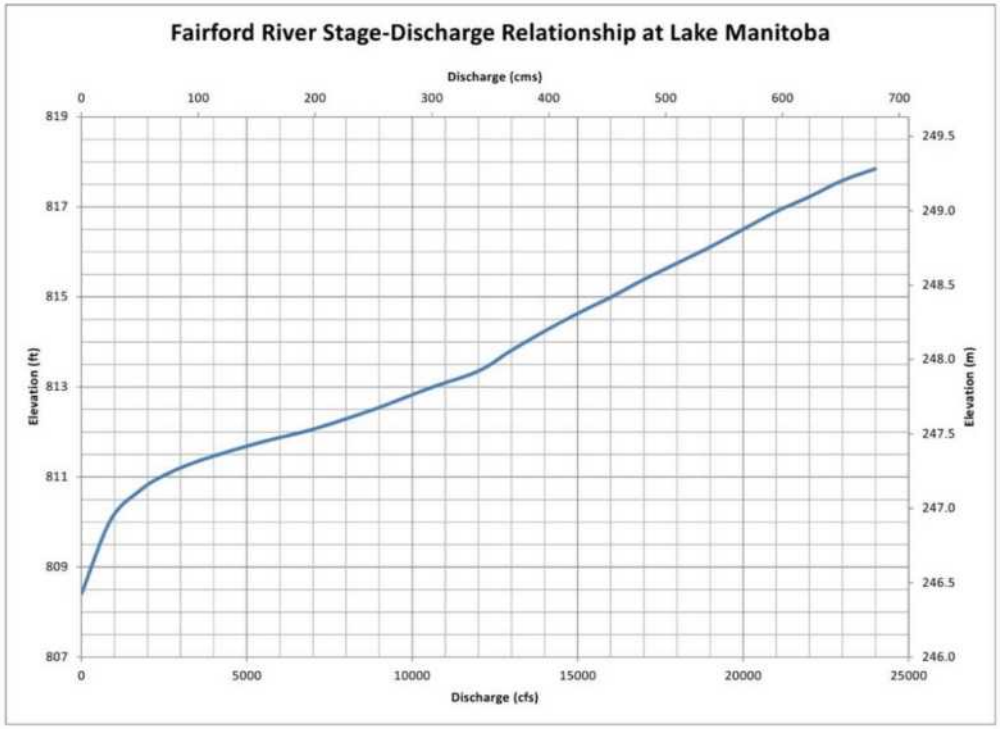


Figure 8.6
Lake St. Martin Stage-Storage Relationship



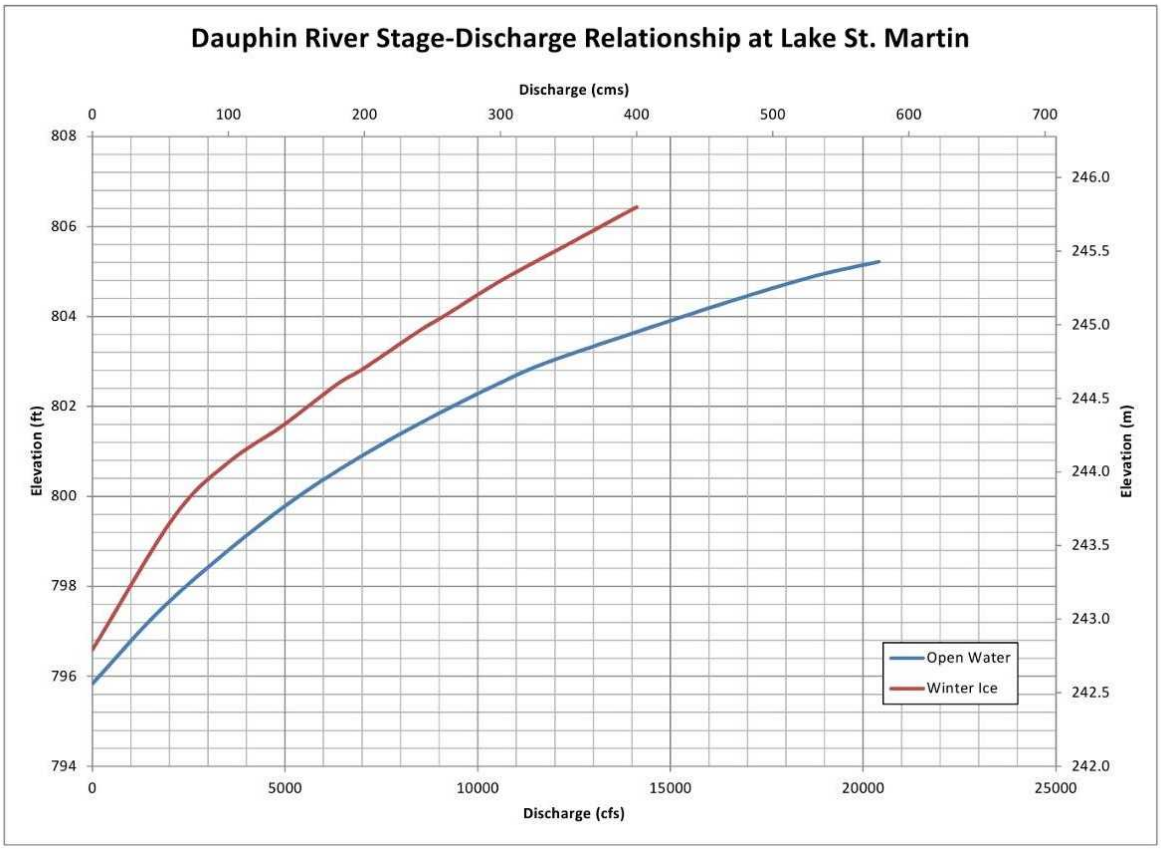
The existing stage-discharge relationship for outflow from Lake Manitoba was based on historical water levels and flow data from Water Survey of Canada (WSC) records and augmented by results from a backwater model analysis of the Fairford River. The model assumes that the Fairford River Water Control Structure (FRWCS) would be fully open. Assuming a fully open FRWCS is representative of how the structure is actually operated at high water levels on Lake Manitoba. Partial gate openings would reduce the outflow from Lake Manitoba (typically only occurs when water levels are well below the flood stage), which are not being considered for this study. Partial gate openings have also been used in the past to control flows during the winter months to limit frazil ice production on the Dauphin River and the risk associated with potential ice staging. This method of operation during the winter was not included in the model since the base case of the analysis assumes that the existing Lake St. Martin Emergency Channel would be made permanent, as later discussed in Section 15.3.2 of the report. With a permanent Lake St. Martin outlet channel, it was assumed that winter restrictions on the FRWCS would no longer be required. The stage-discharge relationship for Lake Manitoba is provided on Figure 8.7.

Figure 8.7
Stage-Discharge Relationship for Fairford River at Lake Manitoba



The stage-discharge relationship for outflow from Lake St. Martin for open water conditions in the Dauphin River was based on historical lake levels recorded by WSC on Lake St. Martin (Station 05LM005), and flow data recorded by WSC on the Dauphin River (Station 05LM006). The stage-discharge relationship for winter ice conditions was based on results from numerical models and actual winter observations of water levels in the Dauphin River. This stage-discharge relationship was developed by KGS Group for the Emergency Reduction of Lake Manitoba and Lake St. Martin Water Levels Project and is provided on Figure 8.8.

Figure 8.8
Stage-Discharge Relationship for Dauphin River at Lake St. Martin



A 15-month simulation period was selected for the flood routing modeling, with the first day of simulation starting on January 1 and the last day ending on March 31 in the following year. This simulation “window” of time was selected such that two winter periods were included in the model.

As depicted by the stage-discharge relationships for the outlet at Lake St. Martin, the formation of an ice cover in the river can reduce the outflow capacity from Lake St. Martin. For this analysis, ice restrictions in the Dauphin River at the outlet of Lake St. Martin were assumed from past experience to occur between December 15 and March 31.

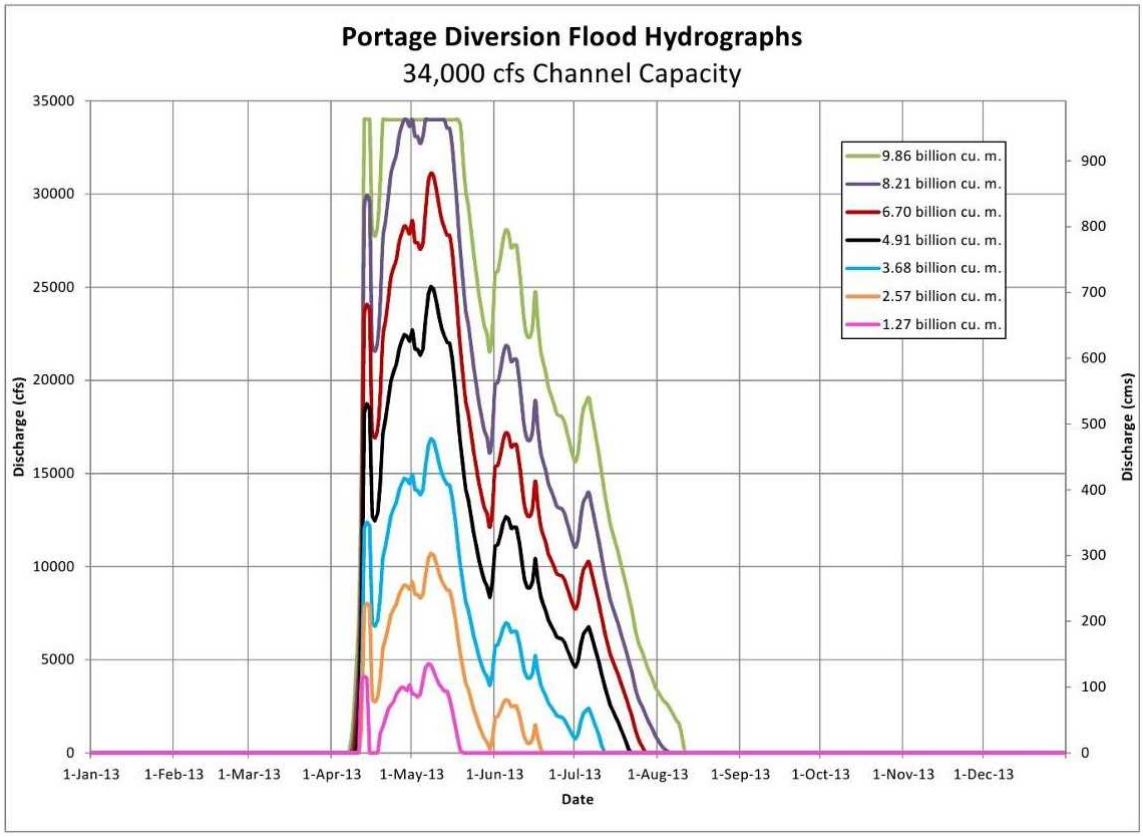
A sensitivity analysis was conducted to determine the effects on the computed lake levels from a variation in the assumed first day of simulation in the model. The results are documented in Section 8.2.2.4.

A starting water level of El 247.65 m (El 812.5 ft) was assumed for Lake Manitoba and El 244.15 m (El 801 ft) was assumed for Lake St. Martin for the baseline conditions. By selecting these water levels, it was assumed that the two lakes were already at the upper limits of their operating ranges prior to the flood.

8.2.2.2 Flood Hydrographs

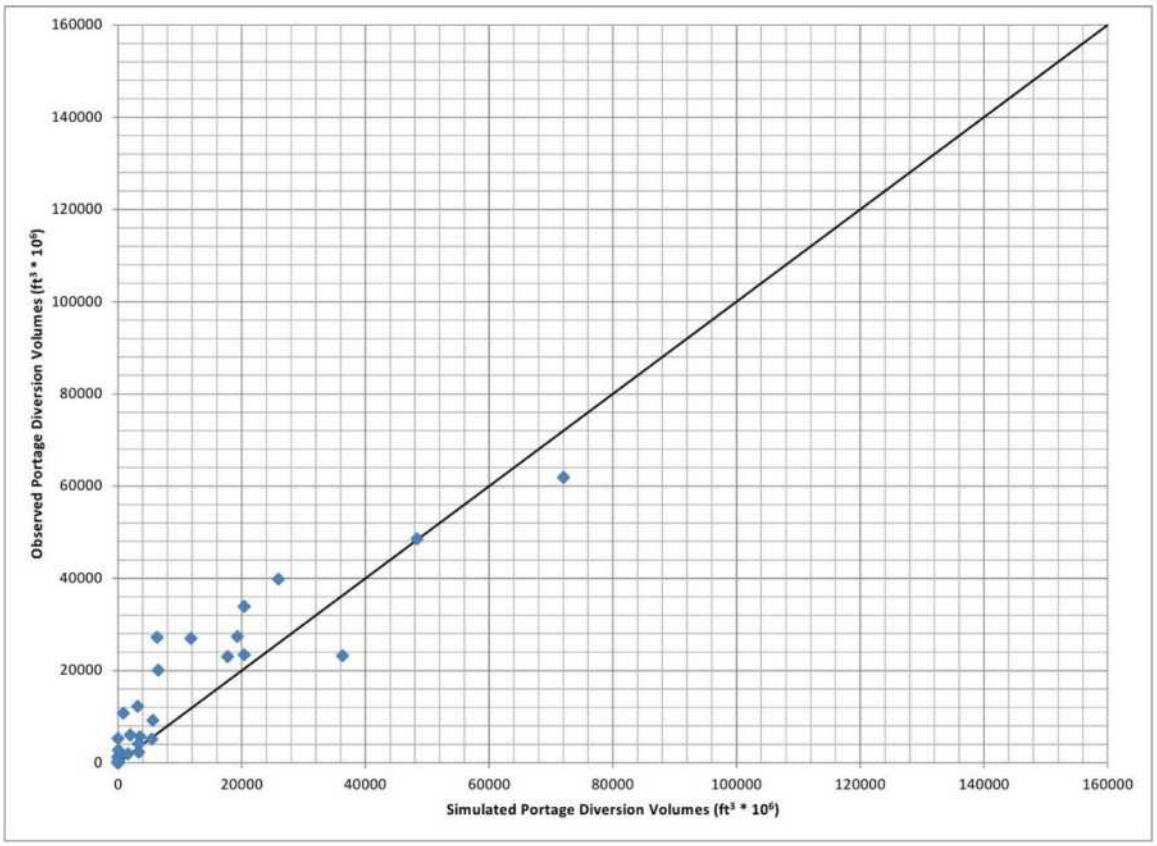
Flood hydrographs were developed for the releases from the Portage Diversion and for local Lake Manitoba inflow for the purpose of forecasting water levels on Lake Manitoba and Lake St. Martin and discharges from these lakes. Flood hydrographs for flow through the Portage Diversion were based on the flood hydrographs that were developed for the Assiniboine River near Holland as documented in Section 5.4. The diverted portions of the Assiniboine River hydrographs through the Portage Diversion were estimated based on knowledge of the current operation guidelines and experience with the operation of the Portage Diversion in the past. The key assumptions for this estimate were listed in Section 5.2.4. However, as per the terms of reference for this study, it was assumed that the maximum discharge capacity of the Portage Diversion was 963 m^3/s (34,000 cfs). The flood hydrographs for releases through the Portage Diversion are provided on Figure 8.9.

Figure 8.9
Portage Diversion Flood Hydrographs



To check the validity of the assumptions made for the Portage Diversion, the historic record of Assiniboine River flows near Holland from WSC Hydrometric Station 05MH005 was analysed to determine the computed amounts of diverted flow. These computed amounts were then compared with the actual recorded amounts of diverted flow. For this comparison, a maximum discharge capacity of 708 m^3/s (25,000 cfs) was adopted for the Portage Diversion for the period of record from 1970 to 2012, with the exception of 2011 when a maximum capacity of 963 m^3/s (34,000 cfs) was adopted. The resultant yearly volume of water diverted by the Portage Diversion was compared to the actual volume of water recorded by WSC Station 05LL019 (Portage Diversion near Portage la Prairie) and is provided on Figure 8.10. As shown on the figure, the computed volumes based on flows near Holland agreed reasonably well on average with the observed volumes passing through the Portage Diversion. On this basis, it was considered that the assumptions made for estimating the operation of the Portage Diversion were valid for the purpose of routing flood volumes on Lake Manitoba and Lake St. Martin, and for forecasting water levels and discharges from these lakes.

Figure 8.10
Comparison of Volumes Released through the Portage Diversion



The flood hydrographs for Lake Manitoba were developed partly from an analysis of the Inflow Available for Outflow (IAO) for Lake Manitoba. The IAO method is based on a simple water balance model, which solves for inflow during a time period when the rate of change in storage and the outflow during the same period are known (i.e., Inflow = Change in Storage + Outflow). The rate of change in storage is determined from observed water levels on the lake and a stage-storage relationship for the lake. The outflow is determined from recorded outflows through the outlet river. This method indirectly accounts for all sources of inflow including precipitation, evaporation, gauged and ungauged surface inflows, and other miscellaneous inputs such as seepage and groundwater inflow. All these inputs are included in the change in the volume of the lake as measured by the water level.

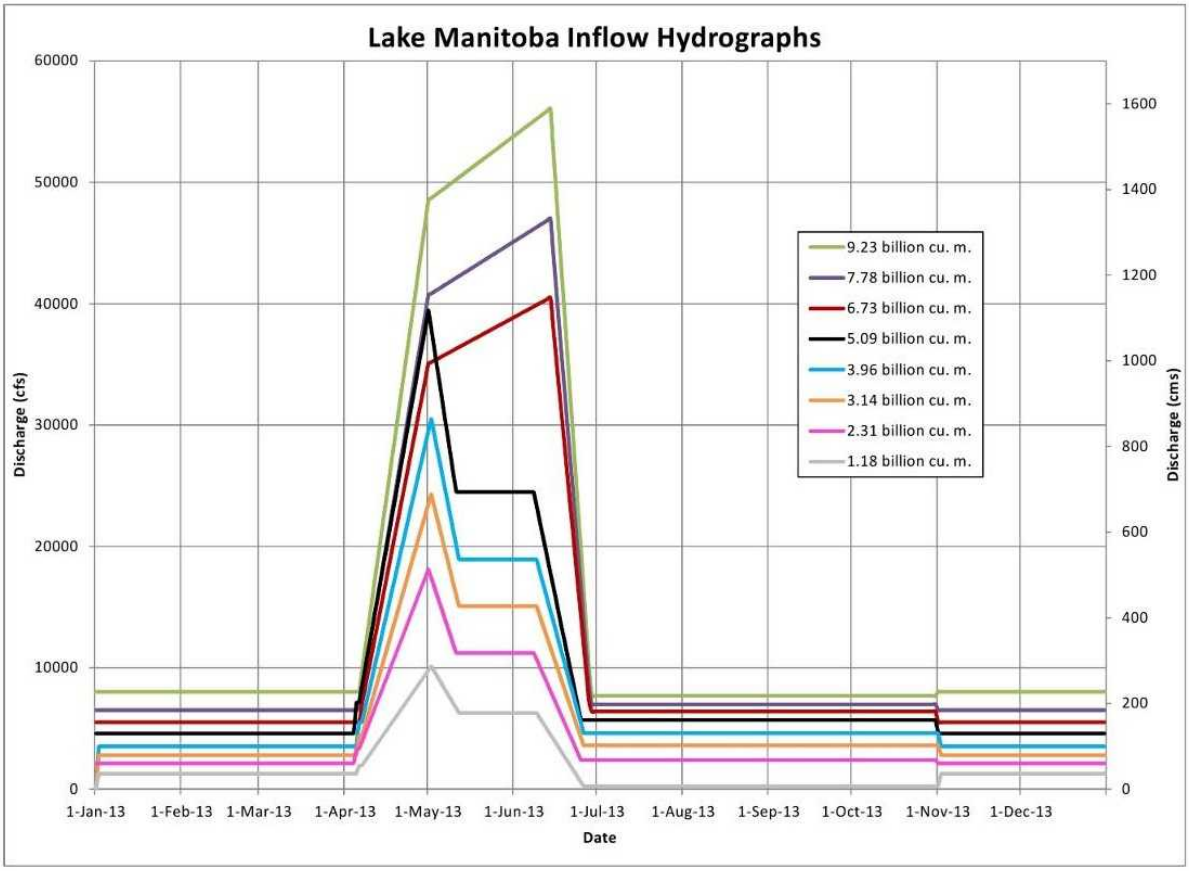
For this analysis, observed Lake Manitoba water levels were based on historic records at WSC Station 05LK002 (Lake Manitoba at Steep Rock) and Station 05LL012 (Lake Manitoba near

Westbourne); and the outflows were based on recorded flows in the Fairford River near Fairford at WSC Station 05LM001, between 1961 to 2012. The stage–storage relationship for Lake Manitoba is provided in Section 8.2.2.1.

Since the IAO consists of a record of all sources of inflow to the lake, the inflow portion from the Portage Diversion as recorded by WSC at Station 05LL019 was subtracted from the results to obtain a continuous record of local inflows to Lake Manitoba.

The annual IAO volumes were input to the USACE HEC-FFA flood frequency curve program to produce a frequency curve of flood volumes. The results of these frequency analyses were used to estimate a range in possible IAO volumes from a 1:500 year to 1:2 year return periods. Eight hydrographs were developed covering the range in flood volumes 1,180,000,000 to 9,230,000,000 m^3 (956,000 to 7,500,000 ac-ft). The IAO hydrograph shape for 2011 was used as a guide to determine the shape of the runoff hydrographs for large floods, with volumes greater than approximately 5,000,000,000 m^3 (4,100,000 ac-ft). The start of runoff was assumed to begin on April 1 with 86% of the peak occurring by May 1, with the peak inflow on June 14. The recession of the flood occurred by July 1. As a comparison, the 2011 Flood had a volume of approximately 6,300,000,000 m^3 (5,100,000 ac-ft). For lesser flood magnitudes, less than approximately 5,000,000,000 m^3, the hydrograph shape was based on the 2009 inflow flood. This hydrograph assumed the start of the hydrograph on April 1, with the peak occurring on May 1. Baseline flows during the winter period prior to and after the flood were based on a statistical analysis of average Waterhen River flows in the winter. For the summer and fall period after the recession of the hydrograph, the baseline flows were based on a statistical analysis of the IAO for that period. The flood hydrographs for local Lake Manitoba inflow are provided on Figure 8.11.

Figure 8.11
Inflow Hydrographs for Local Inflow to Lake Manitoba



It should be noted that the shape of the Lake Manitoba inflow hydrographs shown on the figure are not typical of the shape of a hydrograph for a river or stream like the Assiniboine River for example. That is because the various inflow sources to Lake Manitoba vary in magnitude and timing depending on the flood event. Due to the relatively large size and storage capacity of Lake Manitoba compared to its outflow capacity, the total volume of inflow was the most important factor in developing the hydrograph shape. In general, the majority of the inflow from the largest contributors on Lake Manitoba, which are the Portage Diversion, the Waterhen River, and local precipitation and evaporation, occur in the spring months of April, May and June. This trend is consistent with the hydrographs developed for the lake. Different shapes of hydrographs that would have the same total volume but be distributed differently over these three months would yield very similar Lake Manitoba levels once routed through the model.

8.2.2.3 Modeling Results

Over 25 different combinations of Portage Diversion inflow hydrographs and local Lake Manitoba IAO hydrographs covering the full range of possible flows were routed numerically through the model of Lake Manitoba and Lake St. Martin. Each scenario was for the baseline condition (i.e., maximum capacity of Portage Diversion is 963 m^3/s (34,000 cfs) and assumes no new outlet channel in place on Lake Manitoba), as well as for conditions with the proposed outlet channels.

Resultant hydrographs for Lake Manitoba and Lake St. Martin for a scenario, which results in a peak water level equivalent to a flood event with an approximate return period of 1:200 years, are provided in Figures 8.12 and 8.13. The return period of the flood event was defined based on the results of the Monte Carlo generation discussed later in Section 8.5.1 of this report.

Figure 8.12
Computed 1:200 Year Hydrograph of Lake Manitoba Water Levels

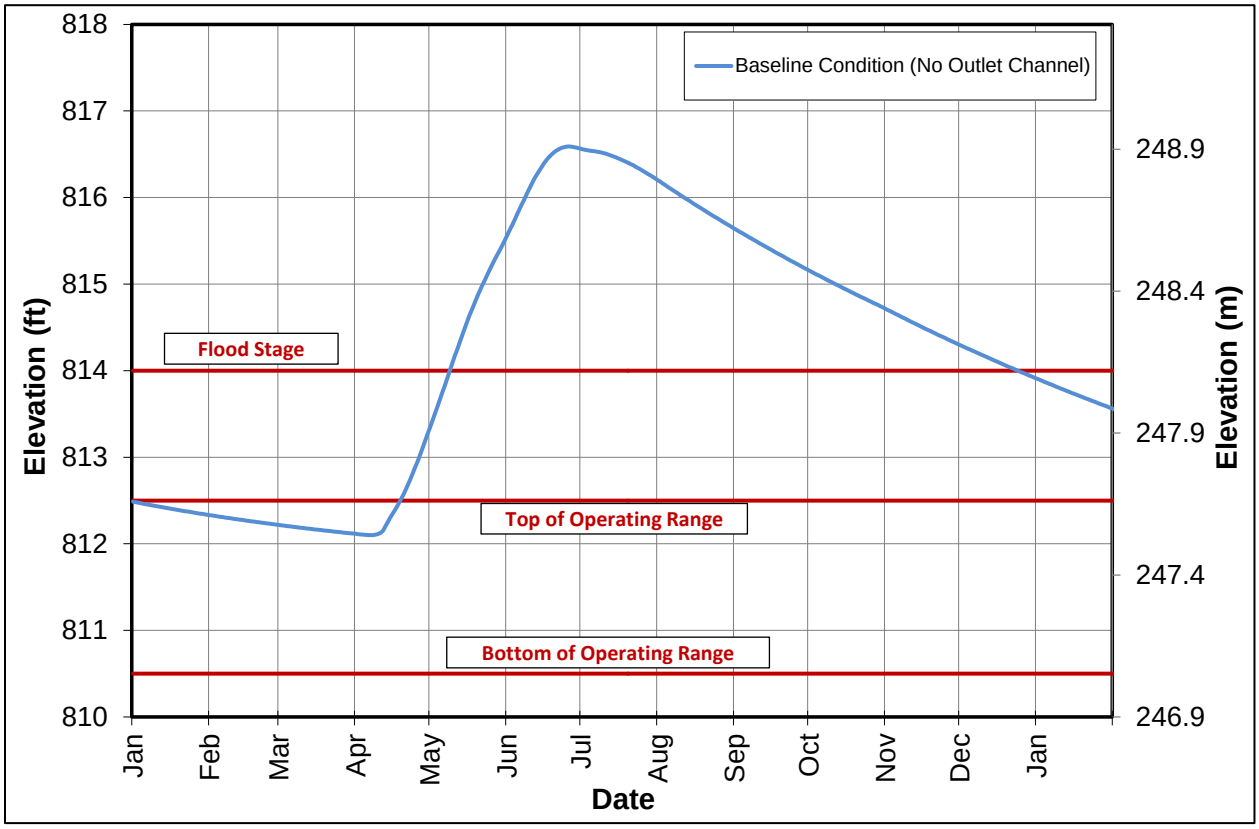
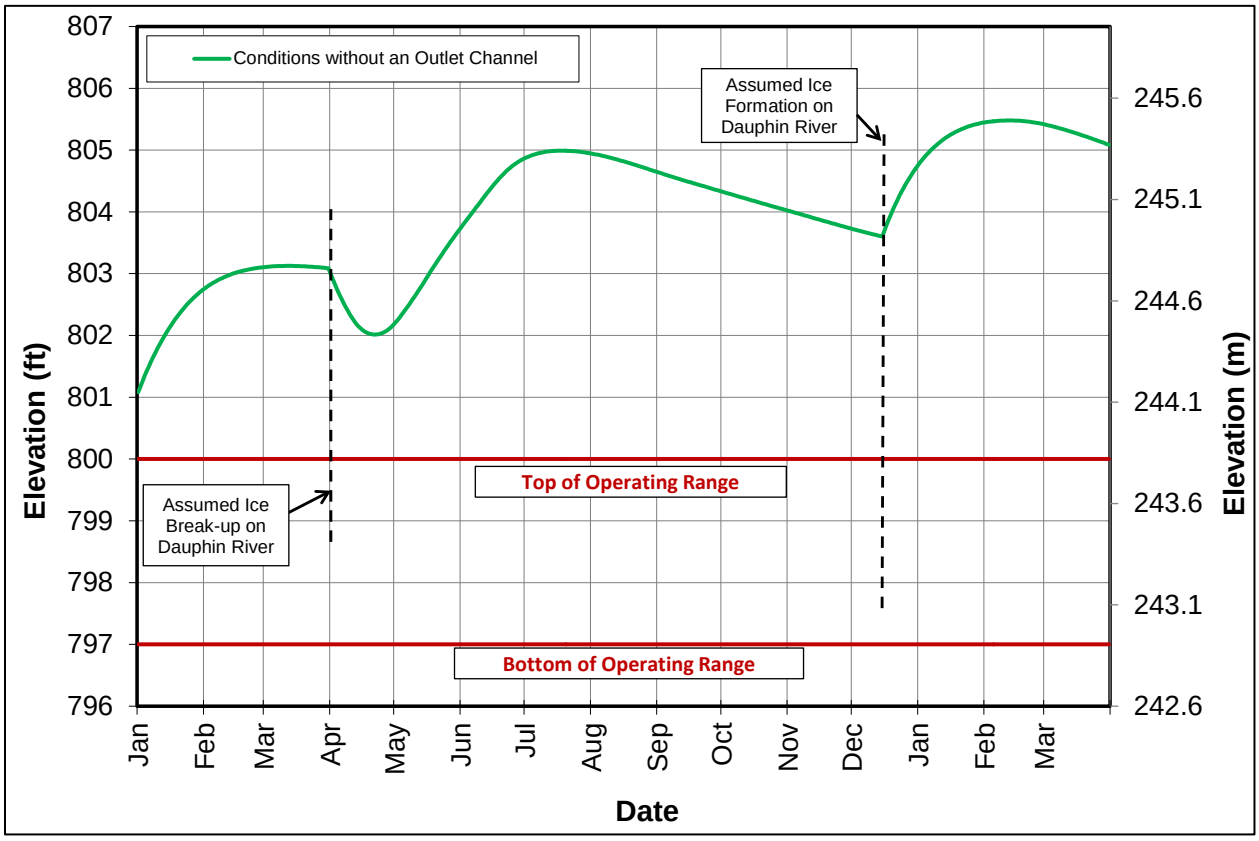
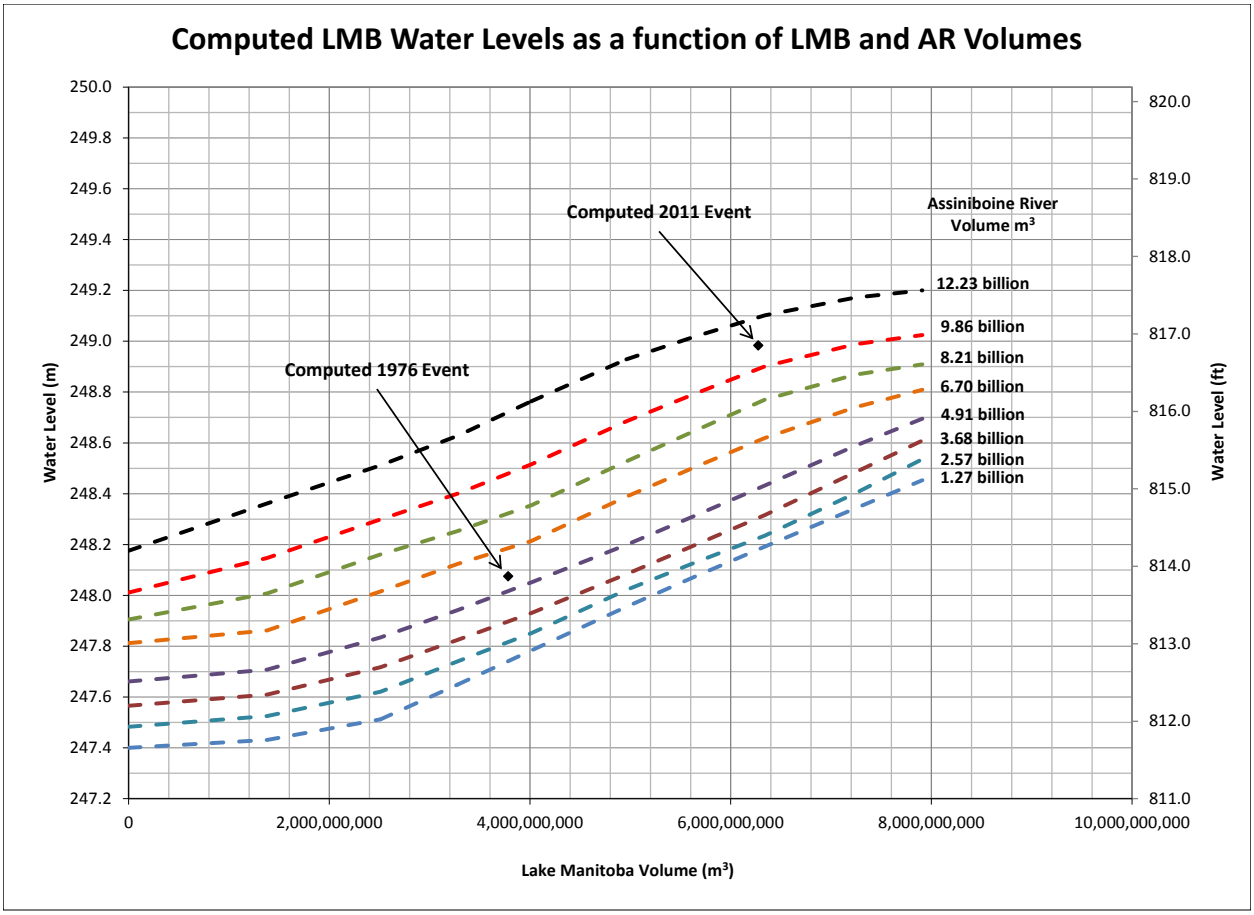


Figure 8.13
Computed 1:200 Year Hydrograph of Lake St. Martin Water Levels



The results of the flood routing were used to generate a relationship between Assiniboine River flood volumes, local Lake Manitoba inflow volumes, and peak Lake Manitoba water levels, as shown on Figure 8.14. By interpolating between lines on the figure, peak water levels on Lake Manitoba can be estimated for any combination of flood volumes from either of the two basins. On the graph, the computed peak water levels for the 2011 and 1976 flood events are shown for reference.

Figure 8.14
Computed Peak Water Levels on Lake Manitoba Water



Note: 1 billion = 1,000,000,000.

As discussed in Section 8.2.2.2, the maximum discharge capacity for the Portage Diversion was assumed to be 963 m^3/s (34,000 cfs) for this analysis. This is more than the original design capacity of the Portage Diversion, which was 708 m^3/s (25,000 cfs). To determine the effects of assuming a 963 m^3/s (34,000 cfs) Portage Diversion capacity instead of the existing 708 m^3/s (25,000 cfs) capacity, Portage Diversion hydrographs were first developed for the existing 708 m^3/s (25,000 cfs) discharge capacity based on the methodology outlined in Section 8.2.2.2. The releases through the Portage Diversion were then routed through Lake Manitoba. The process was then repeated using a discharge capacity of 963 m^3/s (34,000 cfs).

It was determined that the peak water level on Lake Manitoba with the existing discharge capacity of 708 m^3/s (25,000 cfs) in the Portage Diversion was approximately 0.03 m to 0.12 m

(0.1 ft to 0.4 ft) less than with an assumed discharge capacity of 963 m^3/s (34,000 cfs). The higher difference of 0.12 m (0.4 ft) occurs during larger floods when high flows on the Portage Diversion are sustained for longer periods. During smaller floods, an increase in capacity to the Portage Diversion has very little impact on Lake Manitoba since peak flows exceed 708 m^3/s (25,000 cfs) for only short periods. On Lake St. Martin, the corresponding peak water level was at most 0.06 m (0.2 ft) less.

8.2.2.4 Sensitivity Analysis

Sensitivity analyses on the computed peak water level on Lake Manitoba and Lake St. Martin were assessed for the following hydraulic assumptions:

- Starting water level.
- First day of simulation.

The results of the sensitivity analyses on the computed peak water levels are discussed below. From an economic perspective, the effects of changing any of the hydraulic assumptions discussed below were found to be insignificant with respect to the results of the Lake Manitoba damage model discussed in Section 8.6.2. Changes in the hydraulic assumptions had no effect on the conclusions of the economic analysis discussed in Section 22.0 of this report.

Sensitivity to Starting Water Level

As discussed in Section 8.2.2.2, the assumed baseline condition for the starting water level on Lake Manitoba was El 247.65 m (El 812.5 ft) on January 1, without the outlet channels. The exact water level at the start of a flood event, however, varies depending on the timing of the flood and antecedent conditions. To understand how the starting water level on January 1 would affect the peak lake level during a flood, flood hydrographs were also routed based on starting water levels of El 247.2 m (El 811.0 ft) and El 247.8 m (El 813.0 ft). The computed peak Lake Manitoba levels were shown to decrease by approximately 0.12 m (0.4 ft) when starting at an elevation of 247.2 m (811.0 ft) and increase by approximately 0.06 m (0.2 ft) when starting at an elevation of 247.8 m (813.0 ft).

Sensitivity to the First Day of Simulation

Flood hydrographs were also routed using April 1 as the first day of simulation in the numerical model. This was a sensitivity check to understand how variation of the assumed start date would affect the peak water levels on Lake Manitoba. The computed peak water level on Lake Manitoba was shown to increase by approximately 0.12 m (0.4 ft) when starting at El 247.7 m (El 812.5 ft) on April 1. If starting at a different water level than the baseline condition on April 1, the computed peak water level on Lake Manitoba was shown to decrease by approximately 0.15 m (0.5 ft) when starting at El 247.2 m (El 811.0 ft) and increase by 0.15 m (0.5 ft) when starting at El 247.8 m (El 813.0 ft).

8.2.3 Muskingum Flood Routing Models of Assiniboine and Souris River

8.2.3.1 Introduction

This section summarizes the methods and procedures used to calibrate two Muskingum Flood Routing Models: one is of the Assiniboine River between the Shellmouth Dam and Portage la Prairie; and the other of the Souris River from the Canada – United States border to the confluence with the Assiniboine River. Flood hydrographs were routed on the Assiniboine and Souris Rivers for flood return periods of 5 years to 500 years, using the design hydrographs developed for this study and as documented in Section 5.4 of this report. The models formed the basis for evaluating the flood attenuation benefits for the storage reservoirs discussed in Sections 9.5 and 12.3.

8.2.3.2 Hydrology

Inflow hydrographs for flood return periods of 5 years to 500 years developed at twelve locations along the Assiniboine River were used as input to the Assiniboine River model. The twelve locations are shown on Figure 8.15 and the corresponding drainage areas are summarized in Table 8.1. Hydrographs were also developed at eleven locations along the Souris River and were used as input to the Souris River model. The eleven locations are shown on Figure 8.16 and the corresponding drainage areas are summarized in Table 8.2.

Figure 8.15
Assiniboine River Muskingum Model Reaches and Location of Inflow Hydrographs

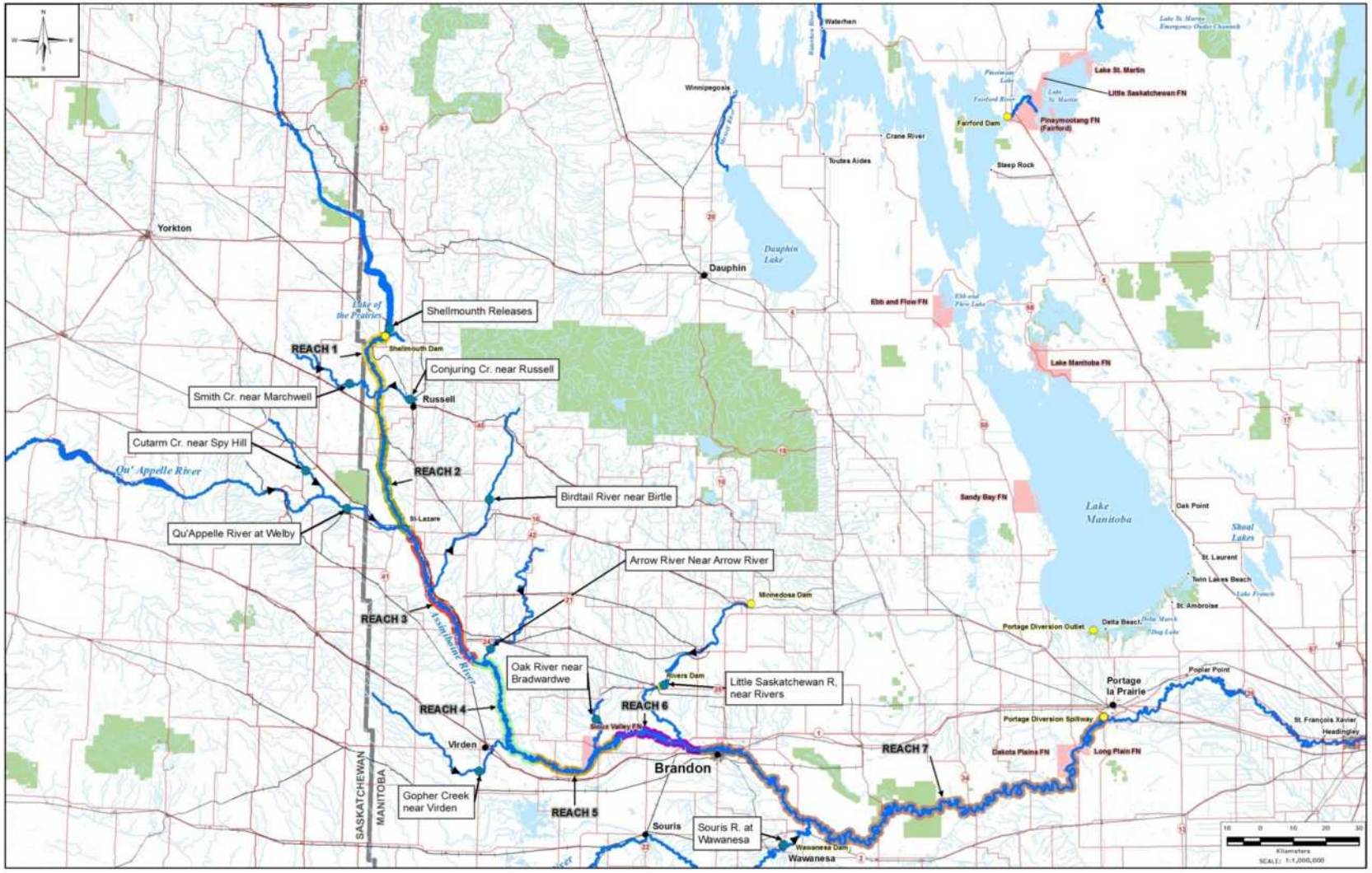


Figure 8.16
Souris River Muskingum Model Reaches and Location of Inflow Hydrographs

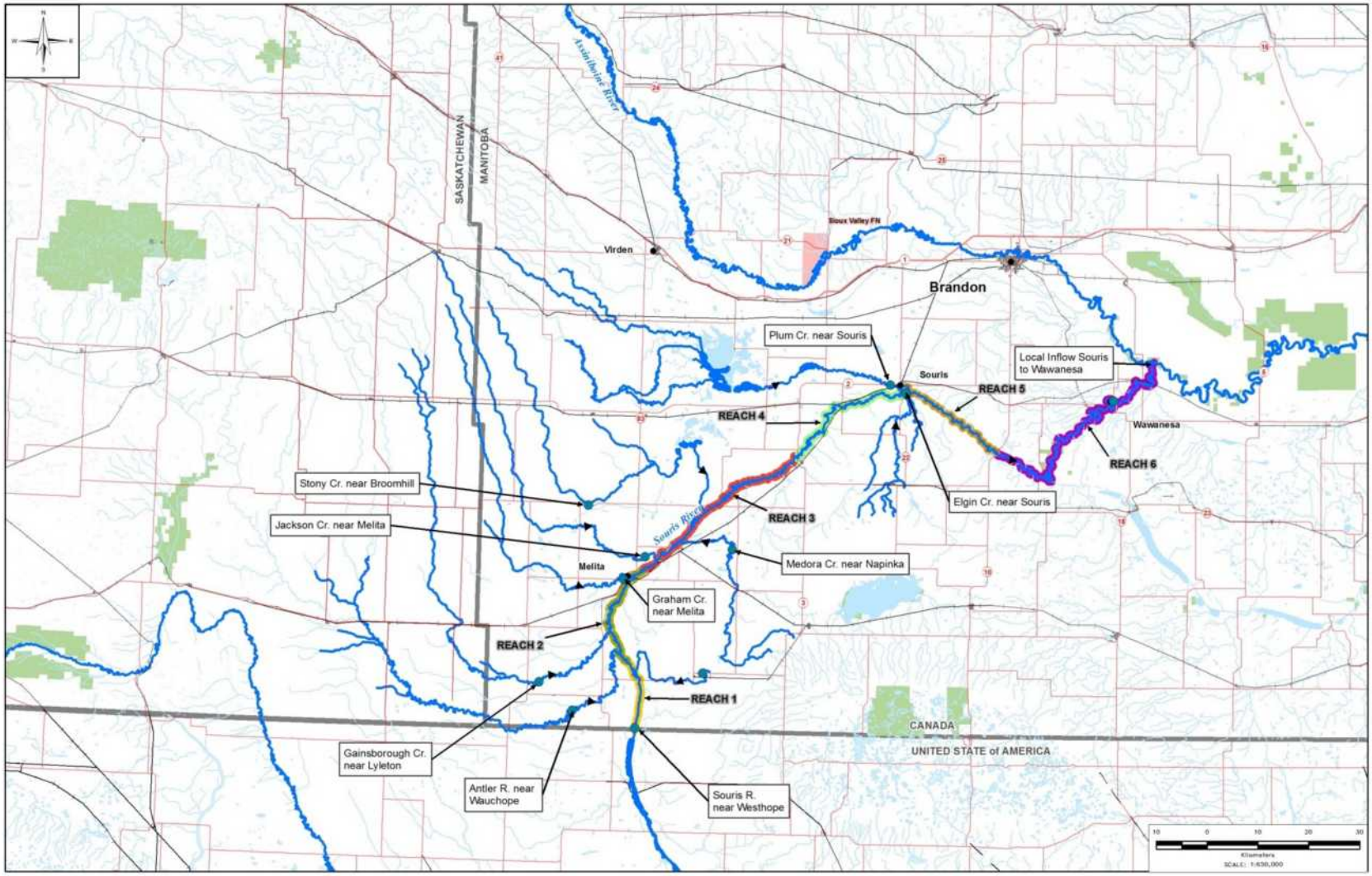


Table 8.1
Inflow Hydrograph Description for Assiniboine River

Station Name	Drainage Area		Lag Time for Modeling to Portage Reservoir
	(km^2)	(mi^2)	(days)
Conjuring Creek near Russell	85	33	10
Smith Creek near Marchwell	435	168	10
Cutarm Creek near Spy Hill	767	296	9
Qu'Appelle River at Welby	55,426	21,400	9
Birdtail Creek near Birtle	1,096	423	8
Arrow River near Arrow River	572	221	7
Gopher Creek near Virden	311	120	5
Oak River near Bradwardine	1,919	741	4
Minnedosa River near Rivers	3,665	1,451	4
Souris River at Wawanesa	61,642	23,800	2
Brandon to Portage la Prairie	5,276	2,037	0
Shellmouth Reservoir Releases	17,900	6,911	11

Table 8.2
Inflow Hydrograph Description for Souris River

Station Name	Drainage Area		Lag Time for Modeling to Portage Reservoir
	(km^2)	(mi^2)	(days)
Souris Flows near Westhope	43,693	16,877	7
Waskada Creek near Granmer	98	38	6
Gainsborough Creek near Lyleton	1,225	473	6
Antler River near Melita	3,212	1,241	6
Graham Creek near Melita	635	245	5
Jackson Creek near Melita	570	220	5
Stony Creek near Broomhill	562	217	4
Medora Creek near Napinka	298	115	4
Downstream Portion of Plum Creek near Souris	6,708	2,591	2
Elgin Creek near Souris	363	140	2
Local Souris to Wawanesa	1,033	399	1

Most of the Assiniboine River inflow hydrographs are located on tributaries of the Assiniboine River at Brandon or on the main stem of the Assiniboine River. These hydrographs were developed by scaling the design hydrographs for the Assiniboine River at Brandon (from Section 5.4) based on the ratio of the drainage areas. Similarly, the inflow hydrographs for the Assiniboine River from Brandon to Portage la Prairie (local inflow to Assiniboine River between the two points) were based on design hydrographs for the Assiniboine River near Holland. The drainage area used in the calculation for the Assiniboine River at Brandon was 90,270 km^2 (34,900 mi^2) and for the Assiniboine River near Holland was 160,000 km^2 (61,800 mi^2). The inflow hydrographs for the Souris River at Wawanesa were taken from Section 5.4 without change. Each of the hydrographs was then lagged by up to ten days relative to the hydrograph for the Assiniboine River near Holland. The lag adopted depended on the location along the Assiniboine River, so that the peak discharge could be properly accounted for in the flood routing model. The adopted lag times for each of the hydrographs are shown on Table 8.1.

The Souris River inflow hydrographs were developed by scaling the design hydrograph for the Souris River at Wawanesa from Section 5.4 based on the ratio of the drainage areas. The drainage area used in the calculation for the Souris River at Wawanesa was 62,025 km^2 (24,000 mi^2). Each of the hydrographs was then lagged by up to six days relative to the hydrograph for the Souris River at Wawanesa. The lag adopted depended on the location along the Souris River, so that the peak discharge could be properly accounted for in the routing model. The assumed lag time days for each of the hydrographs are shown on Table 8.2.

The method used for this analysis to develop the inflow hydrograph assumed that inflows to the Assiniboine and Souris Rivers were equally distributed across the entire watershed. It also assumed that each unit of watershed area contributed an equal volume of water to the two rivers downstream. It is understood that in reality, depending on the flood, inflows to the two rivers vary between tributaries and are not exactly proportional to the drainage area. However, for most tributaries, the average inflow would be similar. On this basis, the drainage area method was deemed acceptable for this conceptual phase of study. An alternative approach to capture the natural variability in distribution of flows within a watershed would require a laborious and complex analysis. For example, unit hydrographs might be developed at each of the tributaries; and they would be individually customized based on available data at nearby gauged stations. Such detailed analysis was not deemed to be warranted at this conceptual

phase of study. Additionally, since the purpose of the inflow hydrographs was to evaluate the hydraulic benefits of large storage reservoirs on the main stems of the Souris and Assiniboine Rivers, a change in the overall distribution of inflows between tributaries would be negligible when evaluating the relative impact of a storage reservoir to downstream flows. It was considered that adoption of the simple apportionment by drainage area would be acceptable for this level of study. Further discussion on the impacts of this assumption is included in Sections 9.5 and 12.3 of this report.

8.2.3.3 Model Descriptions

The Muskingum flood routing method was selected to route flows along the Souris and Assiniboine Rivers for this study. This method was selected for its simplicity and ability to route the large floods under investigation in this study. The method is also consistent with methods used by MIT in the past to route flows on large streams for flood forecasting purposes. Several members from the KGS Group project team were involved in the original development of these models; and a copy of the models were made available by a member of the KGS Group project team, with permission from senior management at MIT.

The Muskingum flood routing method is based on the relationship existing between inflow, outflow and storage along the reach of river. During the advancement and recession of a flood, the inflow does not necessarily equate to the outflow within a reach due to the effects of storage. The Muskingum model applies weighting coefficients to the inflow and outflow within a reach to account for the effects of storage and to determine how the flood translates downstream to the next reach. The weighting coefficients are based on two parameters (“k” and “x” values) that can be modified as a means to calibrate the model. The coefficients are derived by comparing the model output hydrographs to known hydrographs at a given location. The coefficients are adjusted until the output hydrographs are in agreement with the known hydrographs.

The Assiniboine River model was subdivided into seven different reaches, as shown on Figure 8.15. The Souris River model was subdivided into six different reaches, as shown on Figure 8.16. Each reach of the models had one to three of the inflow hydrographs described in Section 8.2.3.2 as inputs to that reach. These hydrographs were either routed through the reach

or applied at the end of the reach. Based on the inflow hydrographs, both models also compute an additional local inflow to each reach to account for the drainage areas that fell outside of the input hydrographs. The inclusion of additional local inflow is required to ensure that the entire flood volume is accounted for in the models. The seven reaches of the Assiniboine River model and their corresponding input hydrographs are summarized in Table 8.3. The six reaches of the Souris River and their corresponding input hydrographs are summarized in Table 8.4.

Table 8.3
Summary of Reaches in the Muskingum Model of the Assiniboine River

Reach Number	Location on Assiniboine River	Input Hydrographs
1	Shellmouth Reservoir to Russell	Conjuring Creek near Russell
		Smith Creek near Marchwell
		Shellmouth Reservoir Releases
2	Russell to St. Lazare	Cutarm Creek near Spy Hill
		Qu'Appelle River at Welby
3	St. Lazare to Miniota	Birdtail Creek near Birtle
4	Miniota to Virden	Arrow River near Arrow River
		Gopher Creek near Virden
5	Virden to Griswold	Oak River near Bradwardine
6	Griswold to Brandon	Minnedosa River near Rivers
7	Brandon to Portage	Souris River at Wawanesa
		Brandon to Portage la Prairie Local

Table 8.4
Summary of Reaches in the Muskingum Model of the Souris River

Reach Number	Location on Souris River	Input Hydrographs
1	Westhope to Coulter	Souris River near Westhope Waskada Creek near Cranmer
2	Coulter to Melita	Antler River near Wauchope
		Gainsborough Creek near Lyleton
		Graham Creek near Melita
3	Melita to Lauder	Jackson Creek near Melita
		Medora Creek near Napinka

Reach Number	Location on Souris River	Input Hydrographs
		Stony Creek near Broomhill
4	Lauder to Souris	Downstream Portion of Plum Creek
		Elgin Creek near Souris
5	Souris to PTH 10	N/A
6	PTH 10 to Wawanesa	Local Souris to Wawanesa

8.2.3.4 Model Calibrations and Results

The design hydrographs for the Assiniboine River upstream of Brandon and the Assiniboine River near Holland from Section 5.4 were used to calibrate the Assiniboine River model. The design hydrographs for the Souris River upstream of Wawanesa also from Section 5.4 were used to calibrate the Souris River model. The "k" and "x" values for each of the model reaches were adjusted progressively until the output hydrographs at Brandon, Holland and Wawanesa were in agreement with the corresponding design hydrographs. The range of "k" and "x" adopted for each of the seven Assiniboine River reaches and for each of the six Souris River reaches for the different flood scenarios modelled are summarized in Tables 8.5 and 8.6. The results of the calibration and output hydrographs are provided in Figures 8.17 to 8.37.

Table 8.5
Range of K and X Values Adopted for the Assiniboine River Model

	1	2	3	4	5	6	7
Reach Name:	**Russell**	**St. Lazare**	**Miniota**	**Virden**	**Griswold**	**Brandon**	**Portage**
K values:	1	1.5	3 - 3.5	1.5 - 2	1.5 - 2	1 - 1.5	1.5 - 2
X values:	0	0.1	0.4	0.2 - 0.3	0.2 - 0.3	0.3	0.2 - 0.3

Table 8.6
Range of K and X Values Adopted for the Souris River Model

Reach Number:	1	2	3	4	5	6
K values:	1	1.5	1	1	1	1
X values:	0.5	0.5	0.5	0.5	0.5	0.5

Figure 8.17
Assiniboine River Model Calibration Results at Brandon for 1:500 Year Flood Event

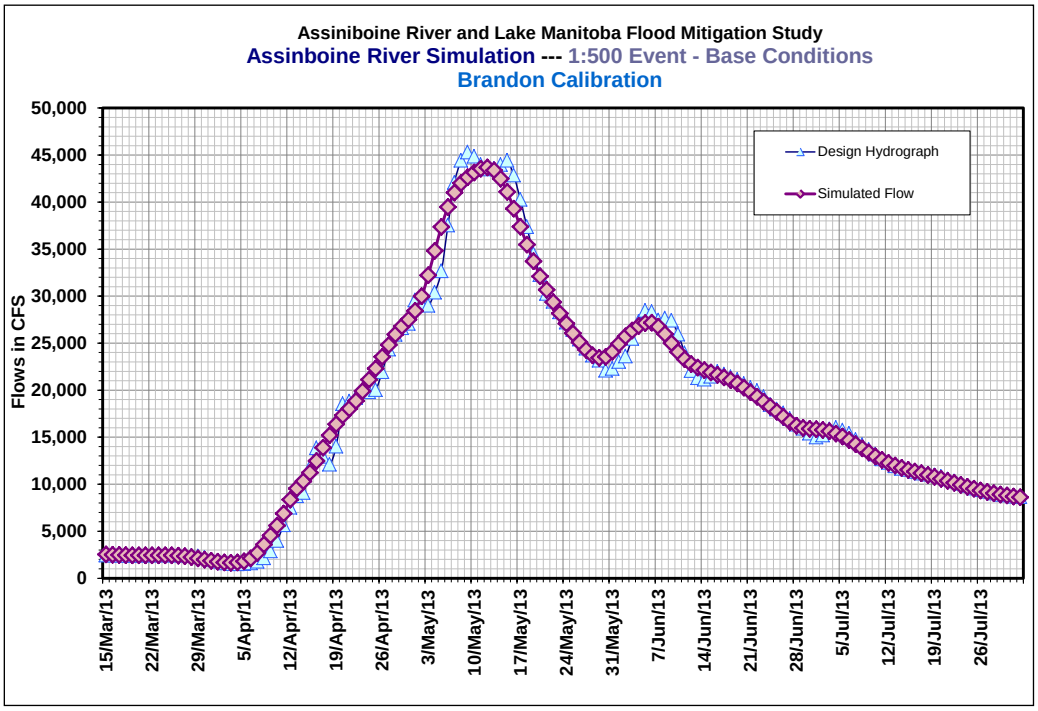


Figure 8.18
Assiniboine River Model Calibration Results at Holland for 1:500 Year Flood Event

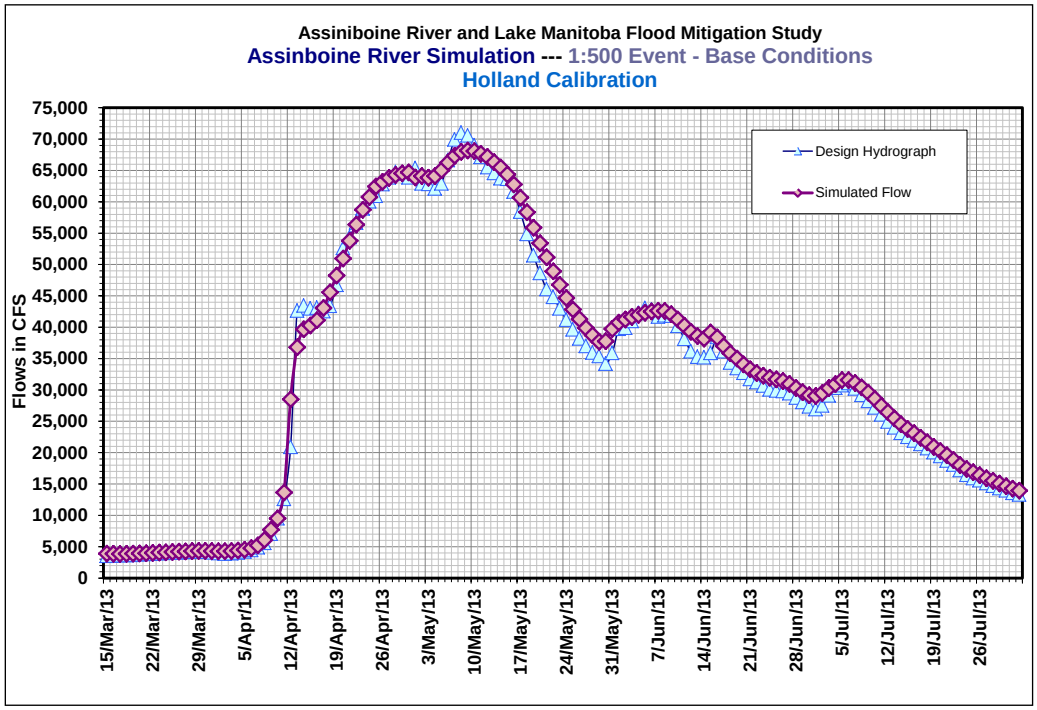


Figure 8.19
Assiniboine River Model Calibration Results at Brandon for 1:200 Year Flood Event

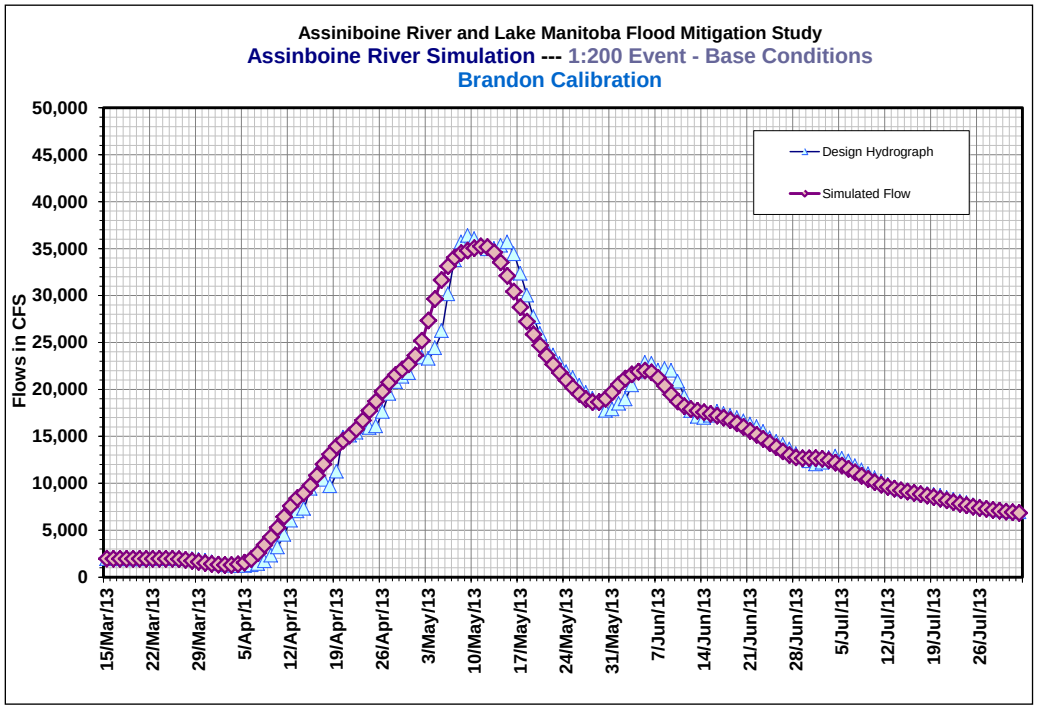


Figure 8.20
Assiniboine River Model Calibration Results at Holland for 1:200 Year Flood Event

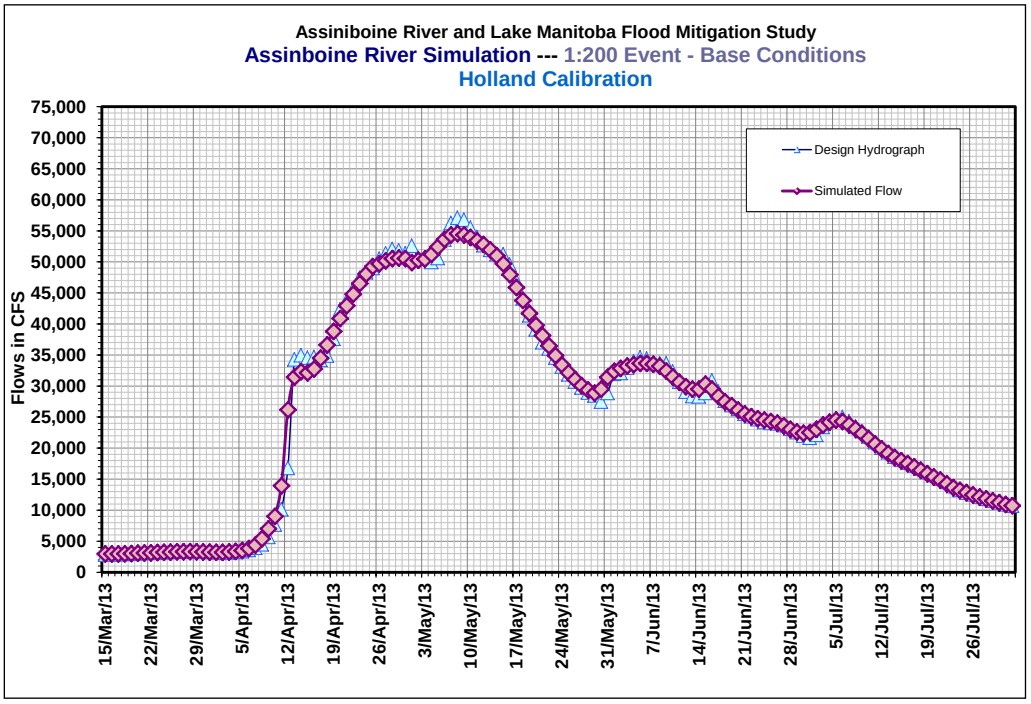


Figure 8.21
Assiniboine River Model Calibration Results at Brandon for 1:100 Year Flood Event

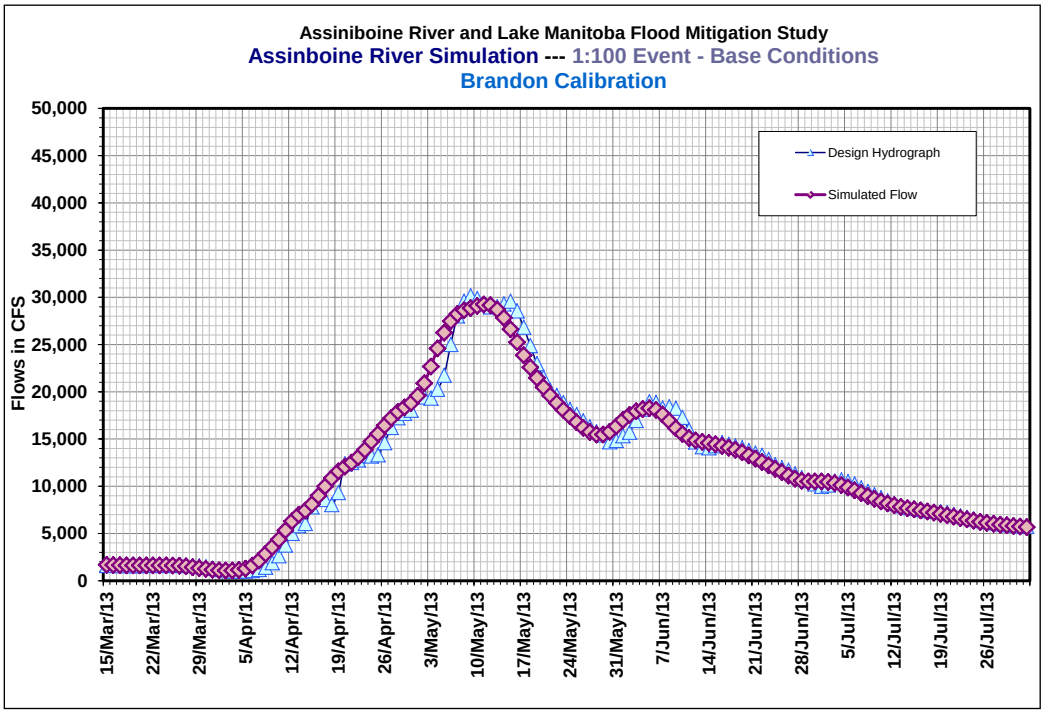


Figure 8.22
Assiniboine River Model Calibration Results at Holland for 1:100 Year Flood Event

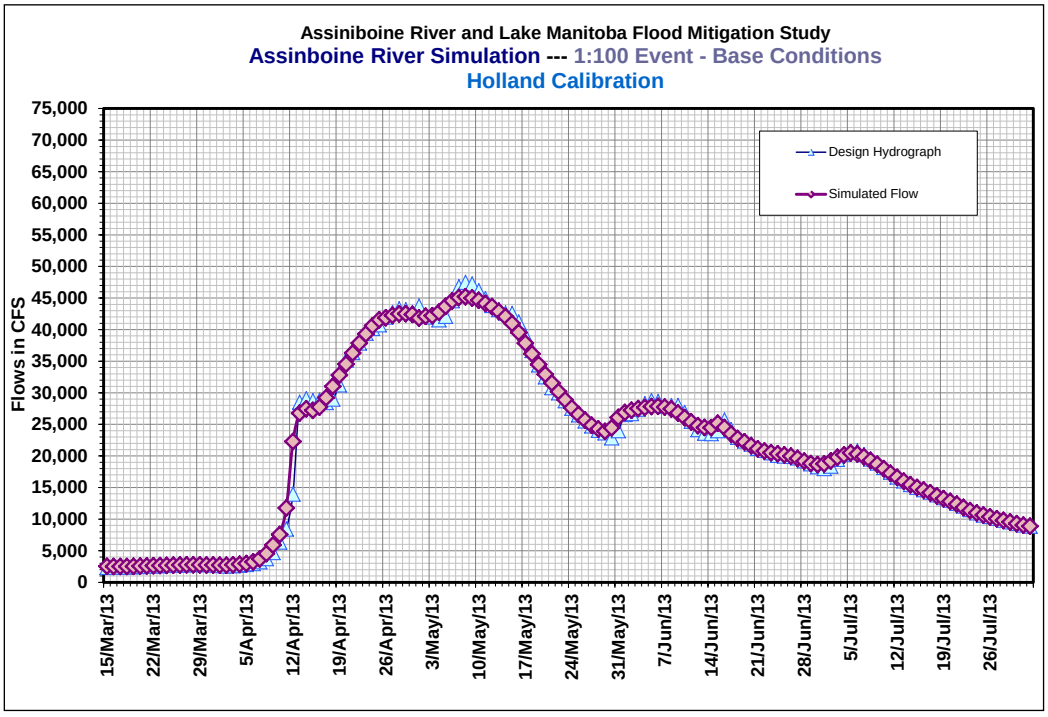


Figure 8.23
Assiniboine River Model Calibration Results at Brandon for 1:50 Year Flood Event

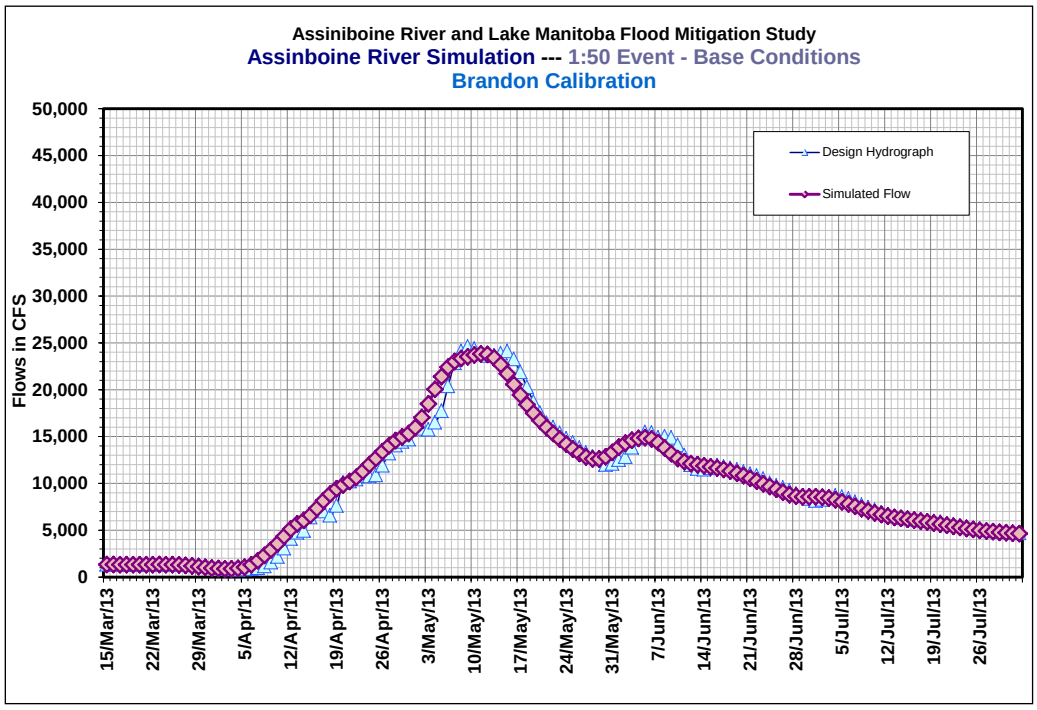


Figure 8.24
Assiniboine River Model Calibration Results at Holland for 1:50 Year Flood Event

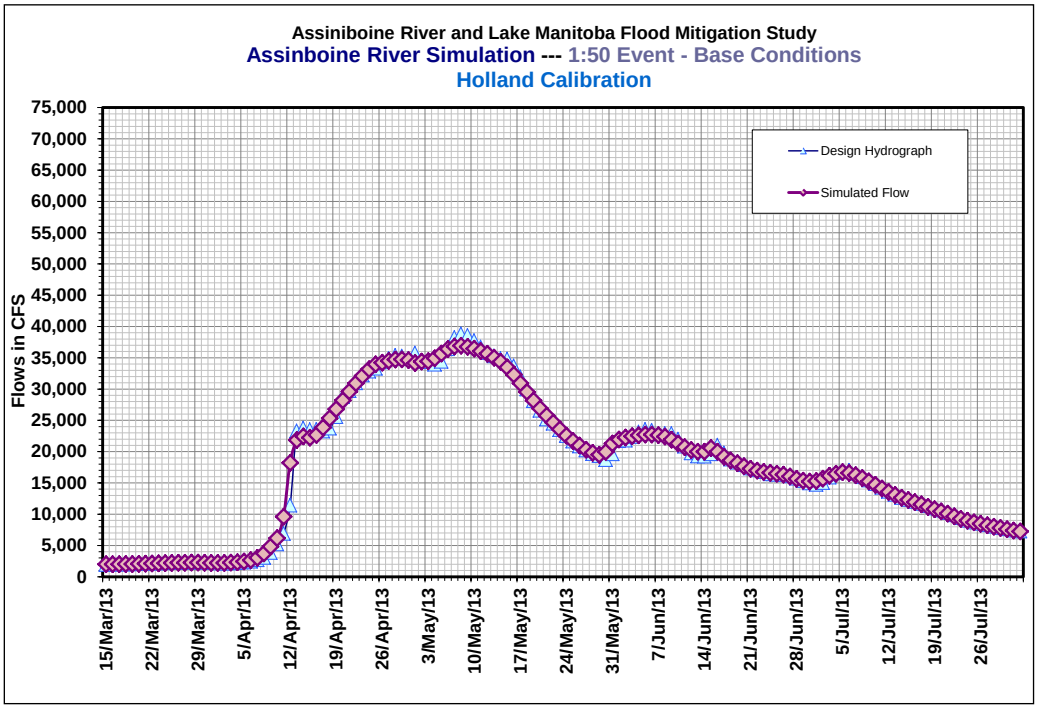


Figure 8.25
Assiniboine River Model Calibration Results at Brandon for 1:20 Year Flood Event

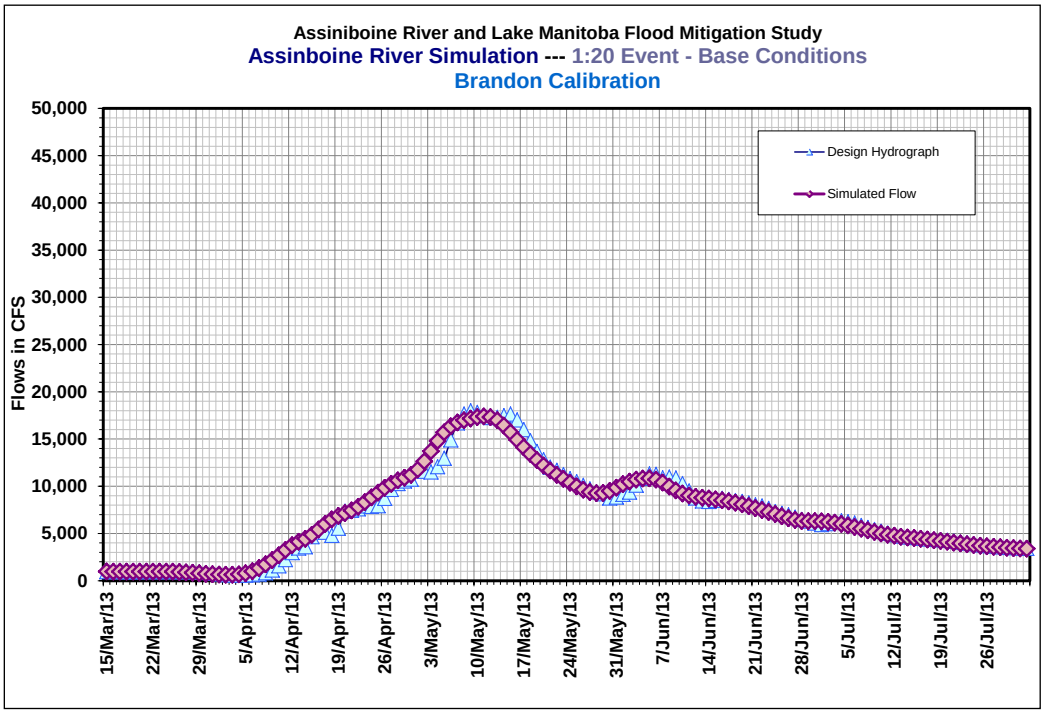


Figure 8.26
Assiniboine River Model Calibration Results at Holland for 1:20 Year Flood Event

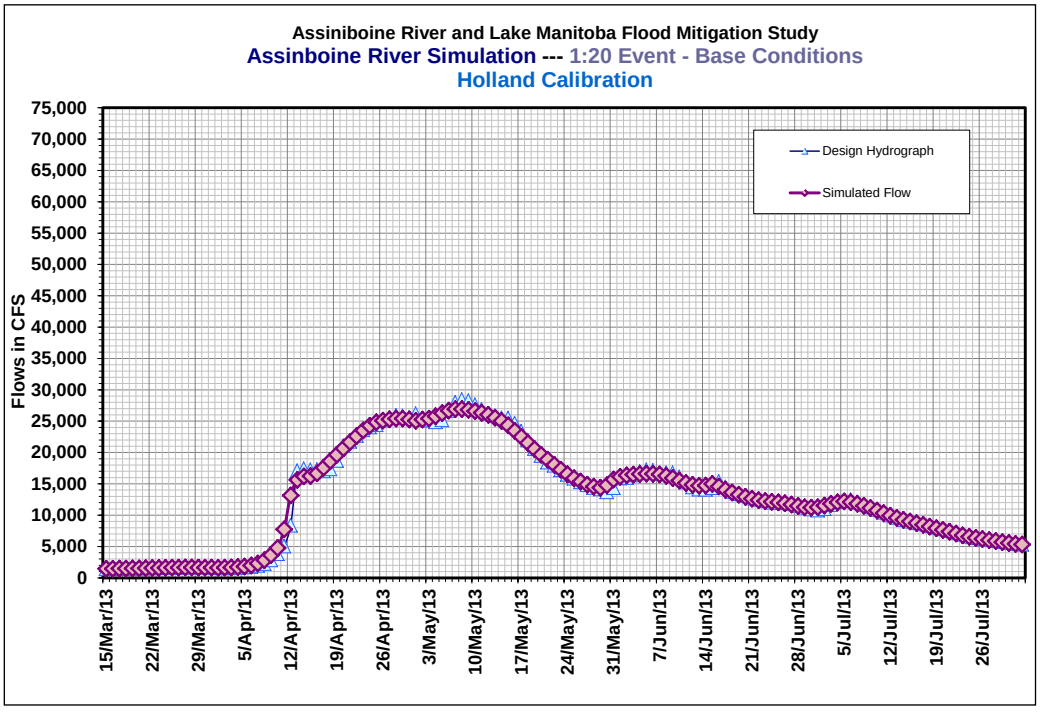


Figure 8.27
Assiniboine River Model Calibration Results at Brandon for 1:10 Year Flood Event

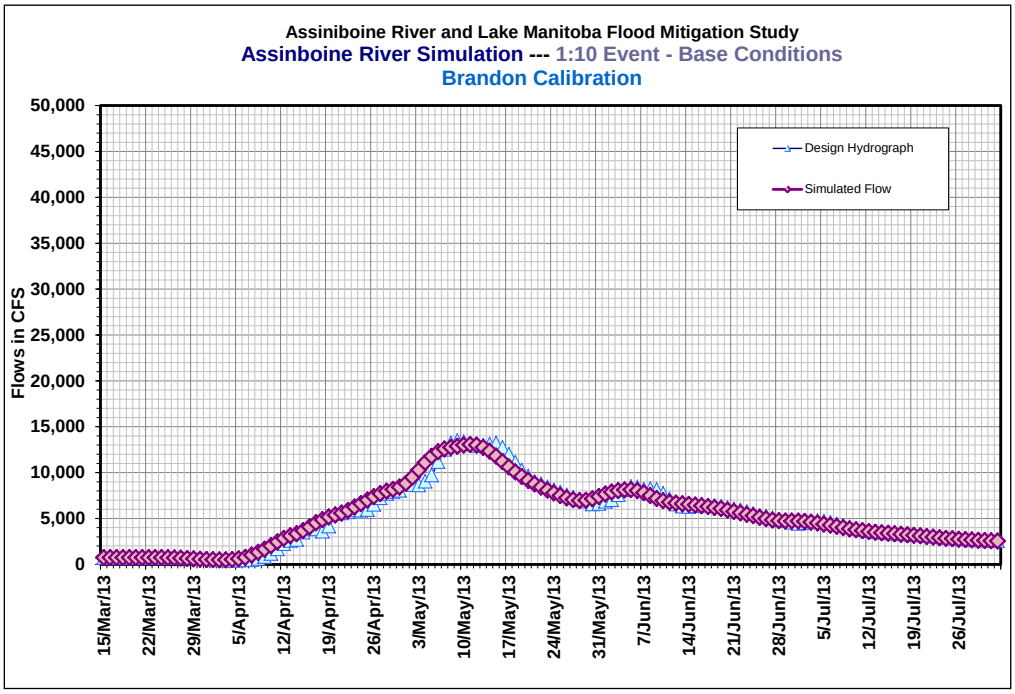


Figure 8.28
Assiniboine River Model Calibration Results at Holland for 1:10 Year Flood Event

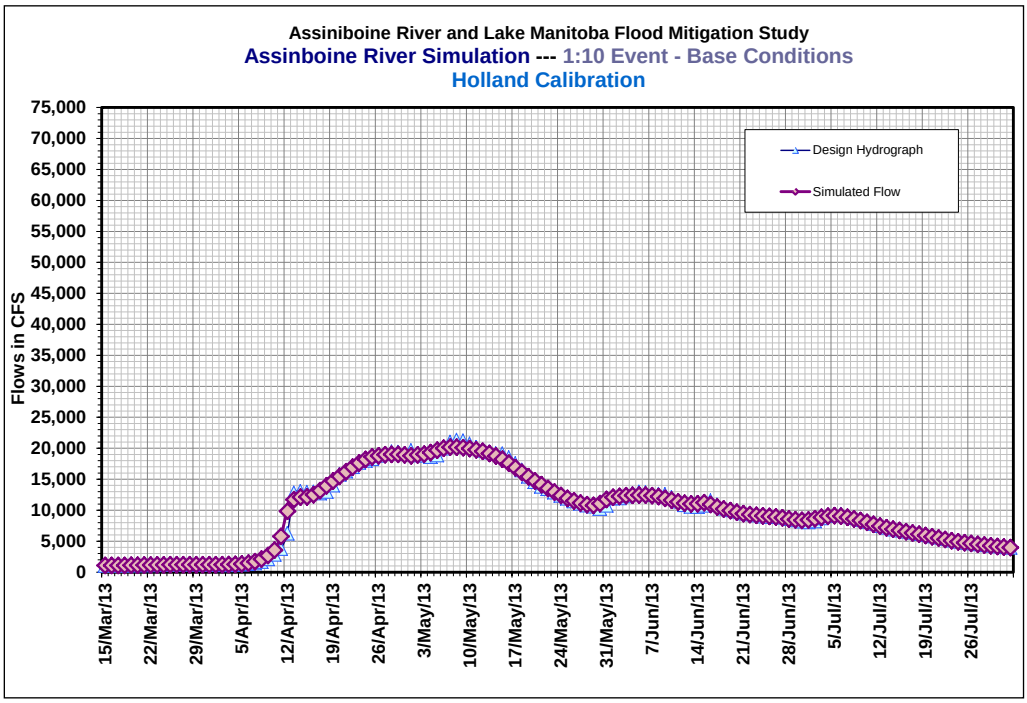


Figure 8.29
Assiniboine River Model Calibration Results at Brandon for 1:5 Year Flood Event

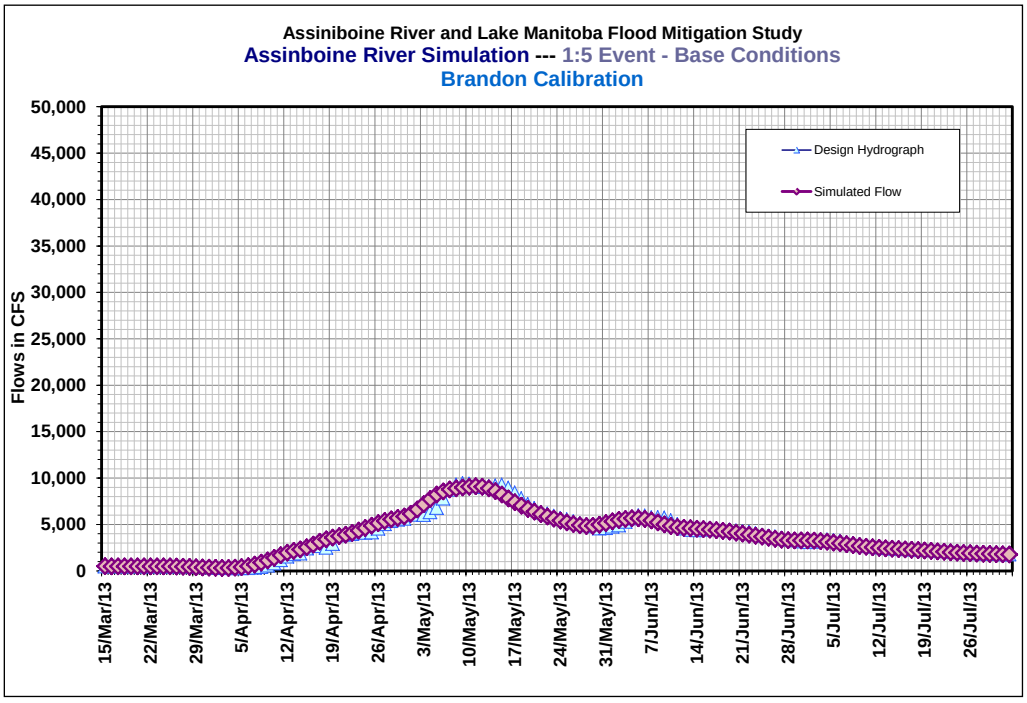


Figure 8.30
Assiniboine River Model Calibration Results at Holland for 1:5 Year Flood Event

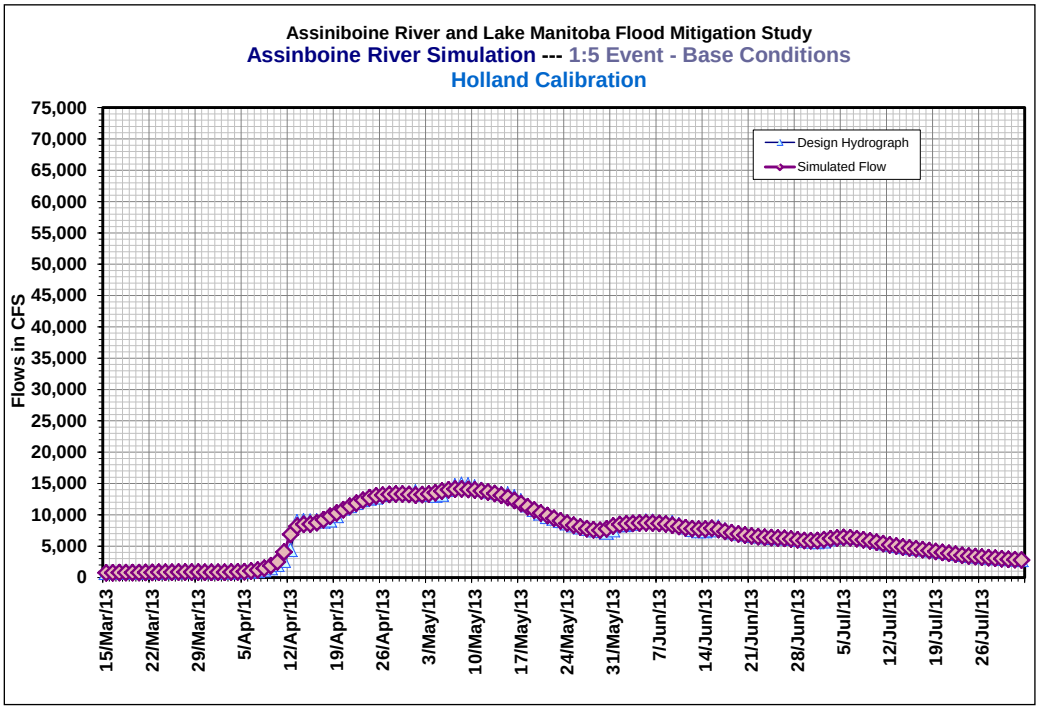


Figure 8.31
Souris River Model Calibration Results at Wawanesa for 1:500 Year Flood Event

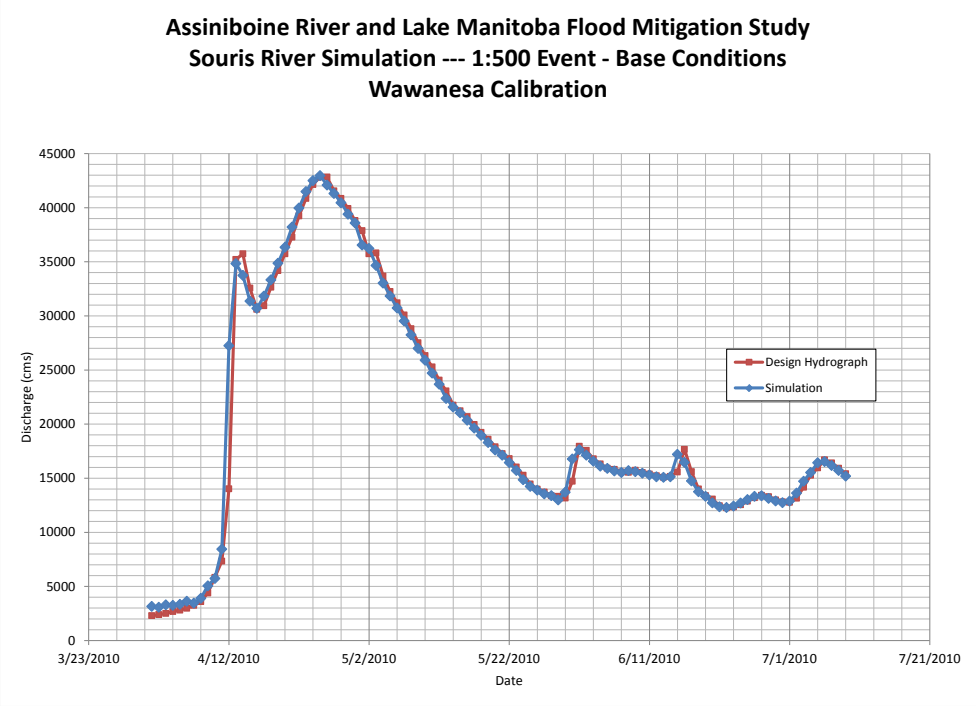


Figure 8.32
Souris River Model Calibration Results at Wawanesa for 1:200 Year Flood Event

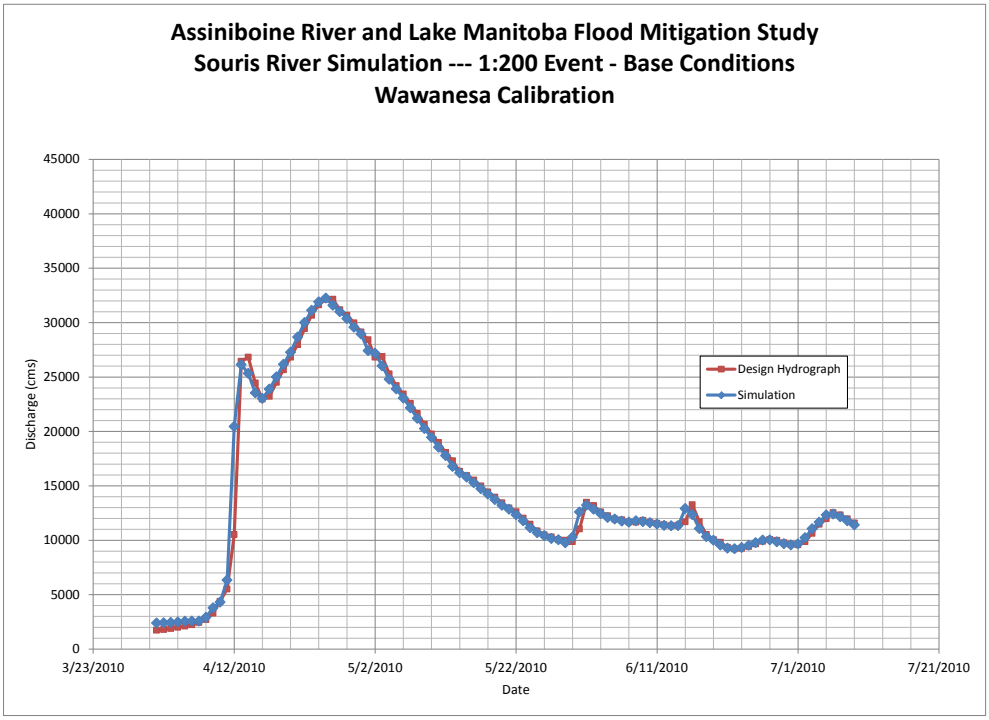


Figure 8.33
Souris River Model Calibration Results at Wawanesa for 1:100 Year Flood Event

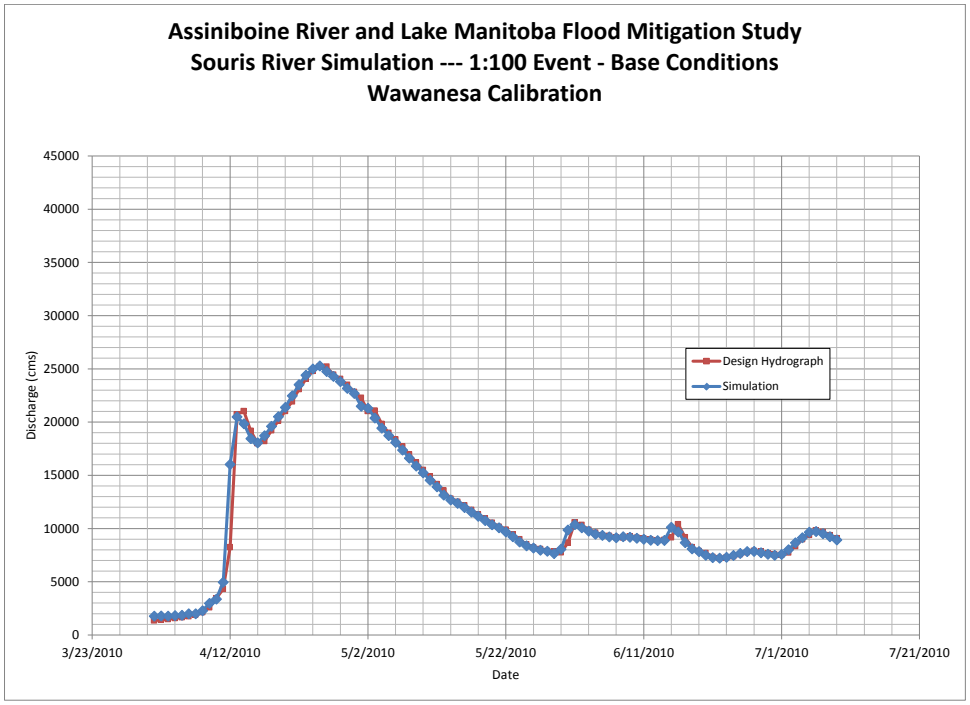


Figure 8.34
Souris River Model Calibration Results at Wawanesa for 1:50 Year Flood Event

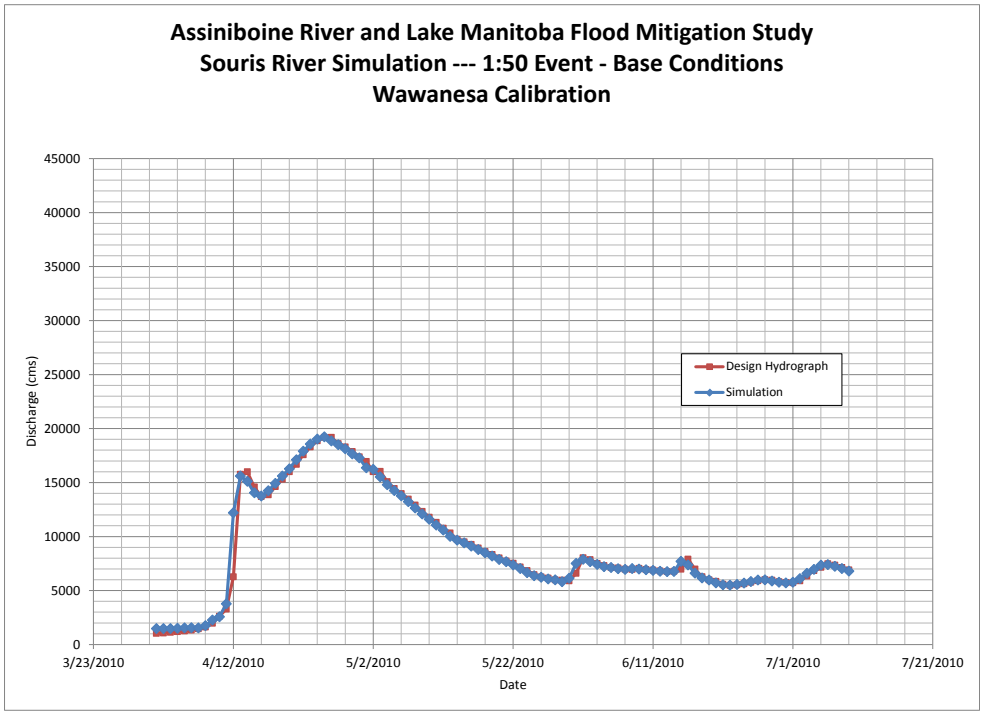


Figure 8.35
Souris River Model Calibration Results at Wawanesa for 1:20 Year Flood Event

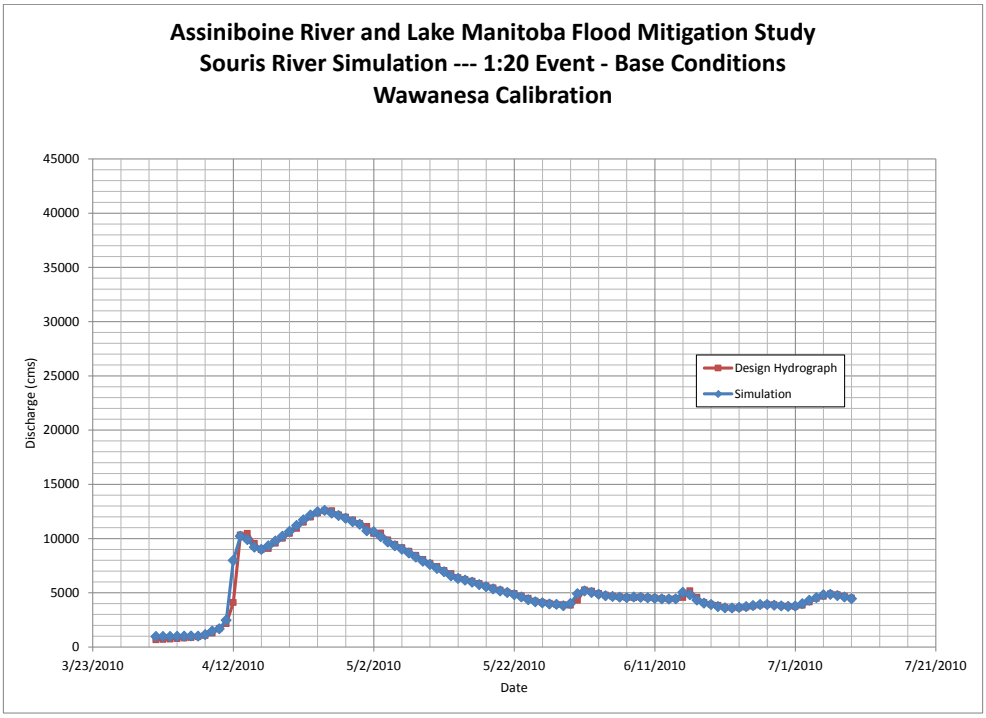


Figure 8.36
Souris River Model Calibration Results at Wawanesa for 1:10 Year Flood Event

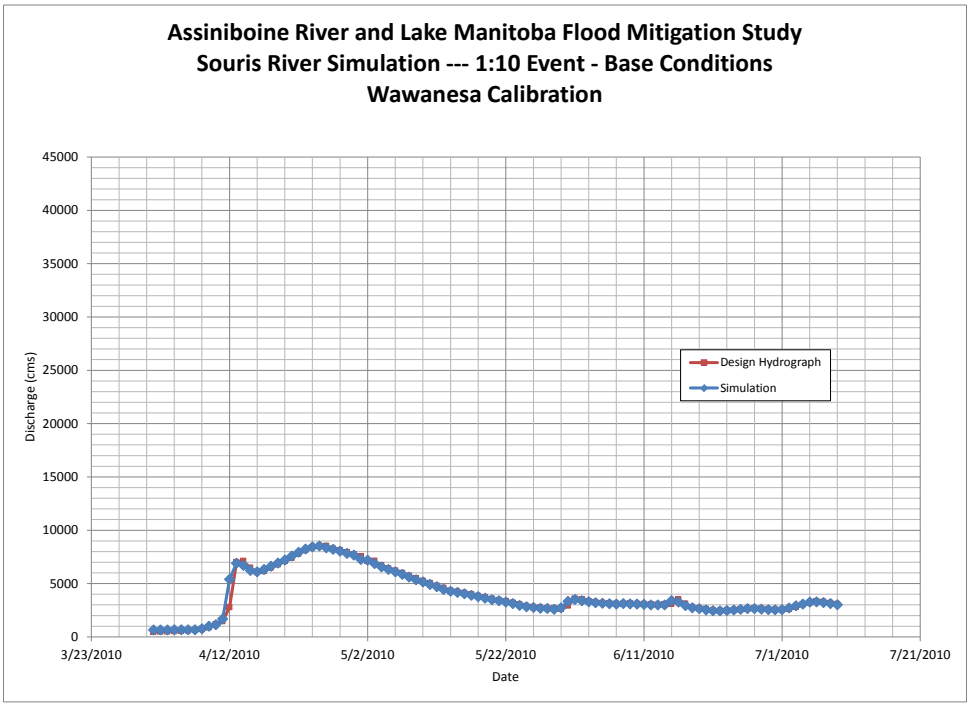
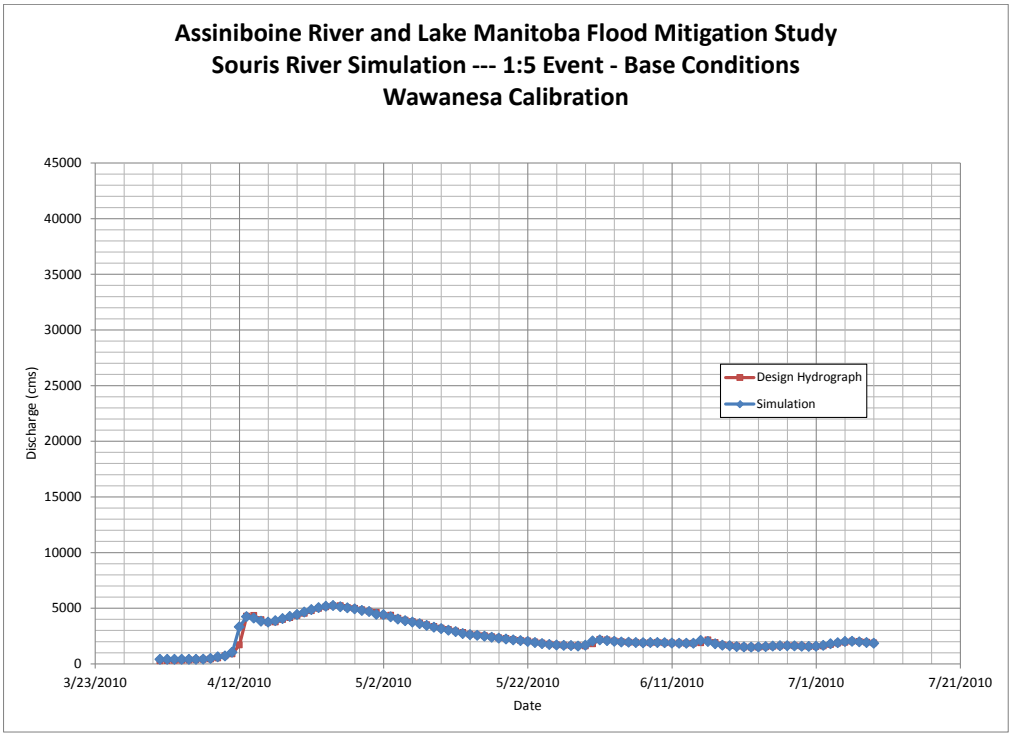


Figure 8.37
Souris River Model Calibration Results at Wawanesa for 1:5 Year Flood Event



As depicted on the figures, the calibration of the model was very successful. For all scenarios simulated, the computed flood peaks in the Souris River at Melita, Souris and Wawanesa were within 1% of the design conditions being simulated and the computed flood peaks of the Assiniboine River at Brandon and Holland were within 5% of the design conditions. The differences in volume between the computed conditions and design conditions being simulated were negligible.

8.2.3.5 Conclusions

The Muskingum flood routing model was successful in calibrating flood conditions in the Assiniboine and Souris Rivers. The computed hydrographs resemble closely the design hydrographs that were developed for the study. On this basis, it was possible to use the model for calculating flood storage attenuation on flows from potential large storage reservoirs on the Assiniboine and Souris River Basins.

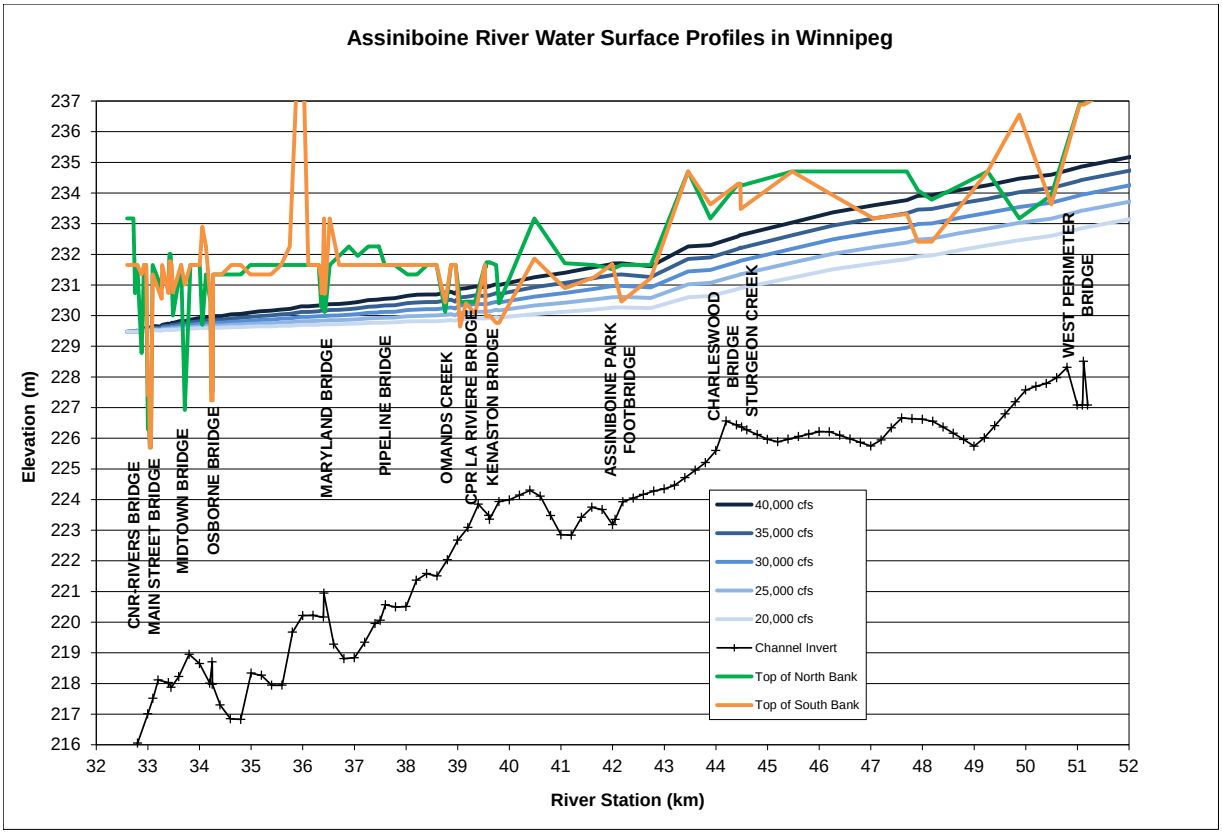
8.3 BACKWATER MODELS

8.3.1 Assiniboine River – Winnipeg

The City of Winnipeg Flood Manual (Acres, 2004) was used to estimate water surface profiles for the Assiniboine River during flood events up to a flow of 700 m^3/s (24,700 cfs). This flow is the maximum Assiniboine River discharge that can be used in the Flood Manual. For flows that exceeded 700 m^3/s (24,700 cfs), the HEC-RAS backwater model of the Assiniboine River that formed the basis for defining the water surface profiles in the Flood Manual was utilized. This model is approximately 26 km (16 mi) long and extends from the confluence of the Assiniboine and Red rivers to PR 334 in Headingley. The HEC-RAS backwater model was last calibrated after the 2009 flood event and was the most up to date model available at the time of the analysis.

Water surface profiles for a range of flow conditions were computed and are shown on Figure 8.38. The top of bank is shown on the figure and corresponds to the highest elevation on either side of the river taken from the model cross sections. The river stationing shown on the figure is in kilometres and based on the Flood Manual, which starts on the Red River at the St. Andrews Lock and Dam. The confluence of the Assiniboine and Red rivers is located between Station 32 and 33 km.

Figure 8.38
Assiniboine River Water Surface Profiles



A conservatively high water level on the Red River, equivalent to a 24.5 ft water level at the James Avenue Pumping Station Datum (JAPSD), was selected as the downstream boundary condition for the analysis. The results of the model were used to estimate potential damages in the City of Winnipeg in the event of a flood on the Assiniboine River and have been incorporated into the flood damage model for the City of Winnipeg, later discussed in Section 8.6.3 of the report.

8.3.2 Assiniboine River – Portage la Prairie to Winnipeg

Model Calibration

The hydraulic analyses of the Lower Assiniboine River were based on two separate HEC-RAS models representing two portions of the Assiniboine River:

1. The "upstream" model, which extends from Portage la Prairie to approximately St. Francois Xavier.
2. The "downstream" model, which extends approximately from Baie St. Paul to Headingley.

The "upstream" model was developed by Hatch and provided by MIT. Two versions of this model were used for the study: one was based on pre-2011 Flood bathymetric data; and the other was based on post-2011 Flood bathymetric data. The pre-2011 Flood model was used in the early stages of the study and was later updated with the results of the post-2011 Flood model.

Prior to obtaining the post-2011 Flood model, concern was expressed that scouring and deposition may have altered the bathymetry of the Assiniboine River channel. However, Section 4.4.2 of the 2011 Flood Review Task Force Report (MFRTF, 2013) suggested that, if there are sediment-related changes to the channel, they are local in extent and do not significantly affect water levels in the river. On this basis, it was assumed that the pre-2011 bathymetry was adequate for this study and that the model could be calibrated to the 2011 Flood event. Therefore, the pre-2011 "upstream" model was calibrated to the 2011 Flood event by adjusting the Manning's "n" value within the main channel until the results of the model were in agreement with the rating curves at PR 248, PR 430, and PR 240. Comparisons between the model results and rating curves at the three bridges have been provided on Figures 8.39, 8.40 and 8.41.

Figure 8.39
Pre-2011 HEC-RAS Model Calibration at PR 248

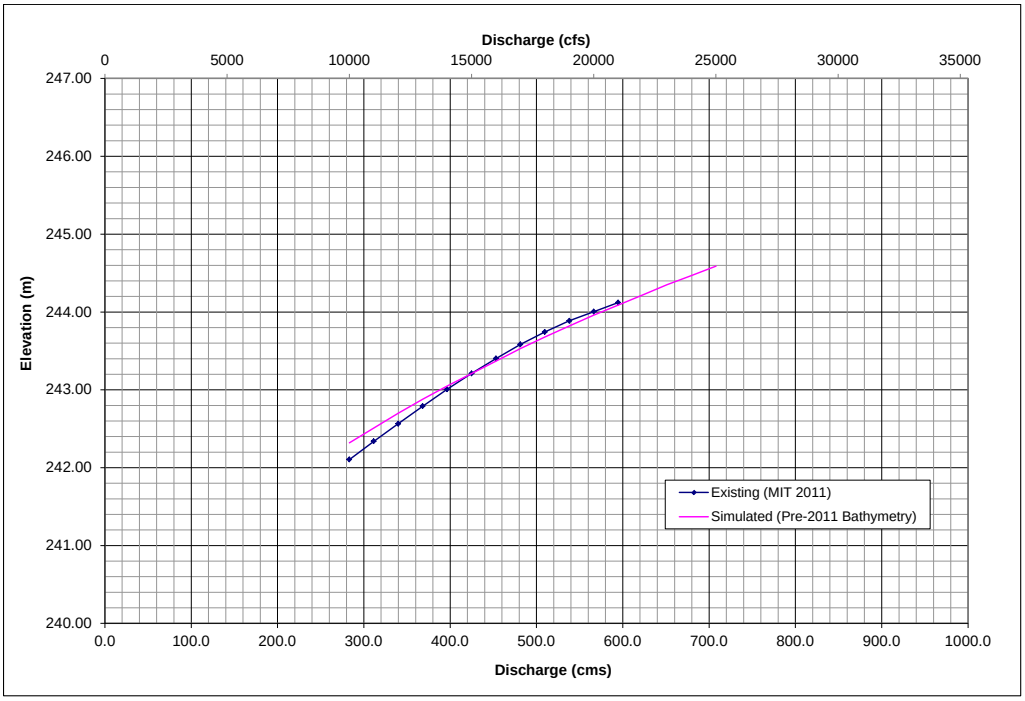


Figure 8.40
Pre-2011 HEC-RAS Model Calibration at PR 430

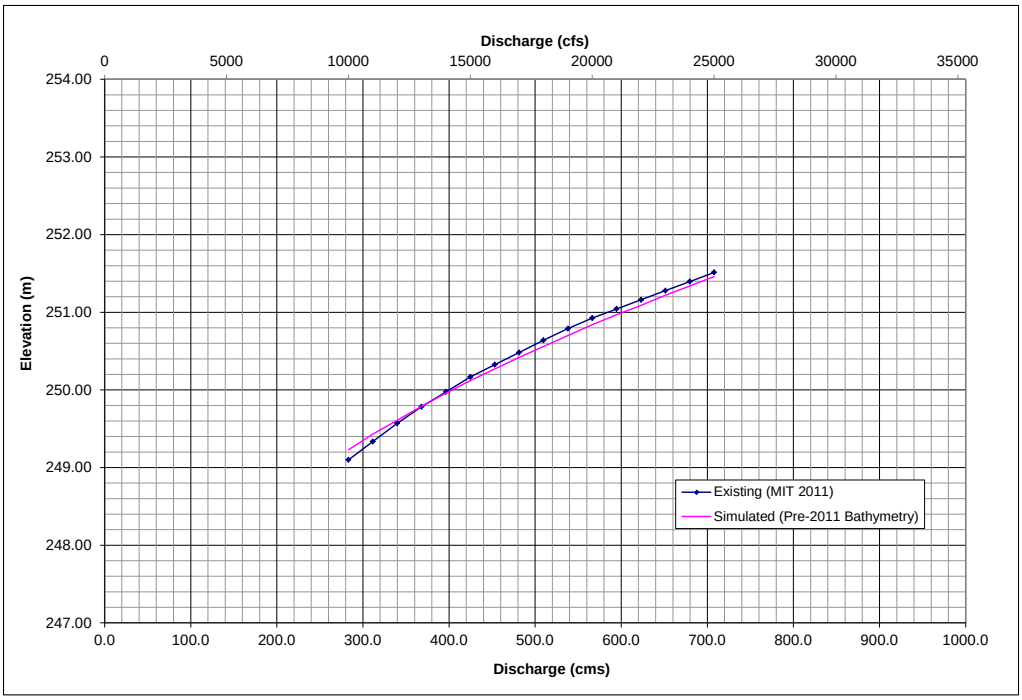
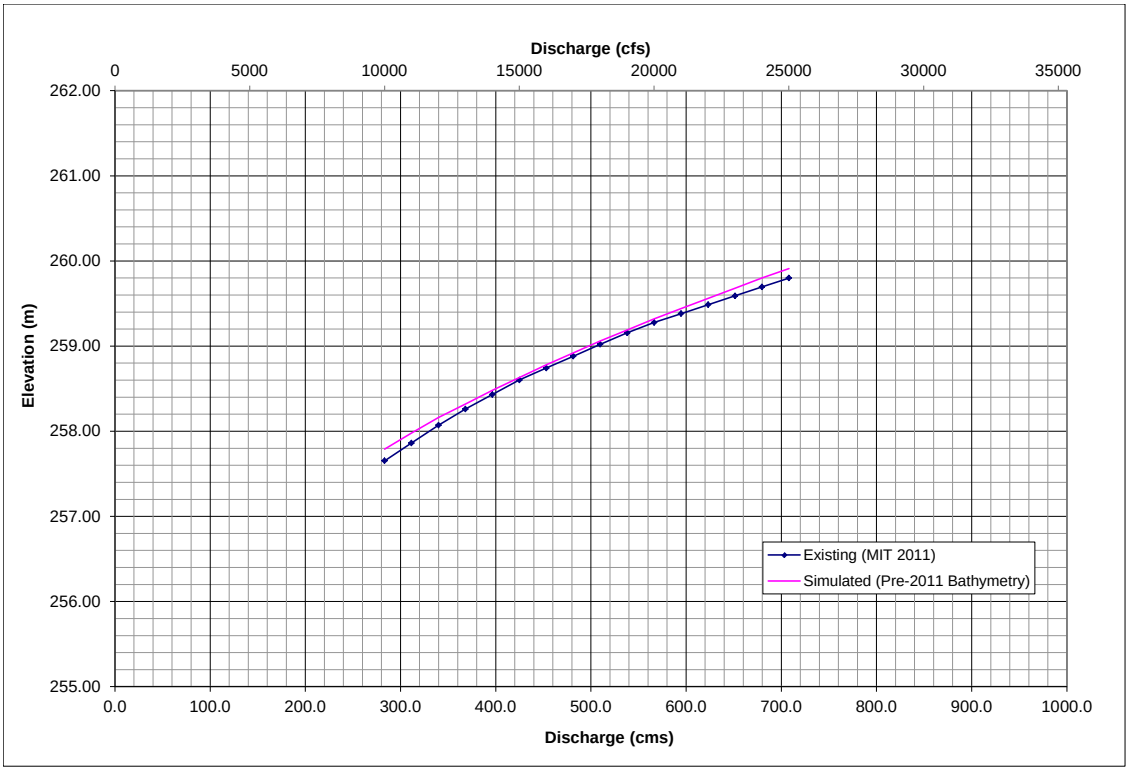


Figure 8.41
Pre-2011 HEC-RAS Model Calibration at PR 240



As shown on the figures, the calibration was good with water levels computed to be within 0.1 m (0.3 ft) of the rating curve on average. The final Manning's "n" value ranged between "0.0265" and "0.031" within the main channel and was adjusted by a maximum of ± 0.006 compared to the original model values.

As the study progressed, results from the post-2011 "upstream" model became available; and MIT indicated that the model showed higher water levels in the Baie St. Paul area for a 1976 type flood, when compared to 1976 water level surveys. On this basis, analyses of the Lower Assiniboine River used the post-2011 model from that point on. Since the post-2011 model had already been calibrated to the 2011 Flood, changes to the model were not necessary for the study.

The "downstream" model developed by KGS Group (1999) was calibrated on two occasions for the study. Similar to the "upstream" model, results based on the first calibration were used in the early stages of the study and were later updated with results based on the second calibration.

The first calibration was completed at the beginning of the study based on the limited amount of data available after the 2011 Flood and required adjusting the Manning's "n" value in the reach that extended between St. Francois Xavier and Baie St. Paul. Downstream at Headingley, the 2011 rating curve was very similar to the 1999 rating curve. Therefore, only the upstream portion of the model was modified such that at Baie St. Paul the "downstream" model would be consistent with the "upstream" (or the "Hatch") model.

The second calibration occurred in the winter of 2015, based on new data from MIT made available from the 2011 and 2014 floods, which showed that the model was underestimating the water levels by about 0.3 m to 0.5 m in some locations between Baie St. Paul and Headingley. On this basis, the model was re-calibrated by adjusting the assumed Manning's "n" values and by updating the downstream boundary condition using a 2014 rating curve at the Water Survey of Canada Gauge Station 05MJ001 (Assiniboine River at Headingley) as shown on Figure 8.42. Also, the river stationing and distances between cross sections were updated to be consistent with current river conditions, although changes were not made to the river channel geometry in the model as this was accounted for by adjusting the Manning's "n" values. Results of the 2015 calibration are shown on Figure 8.43.

Figure 8.42
Rating Curve at WSC Gauge Station 05MJ001 (Headingley)

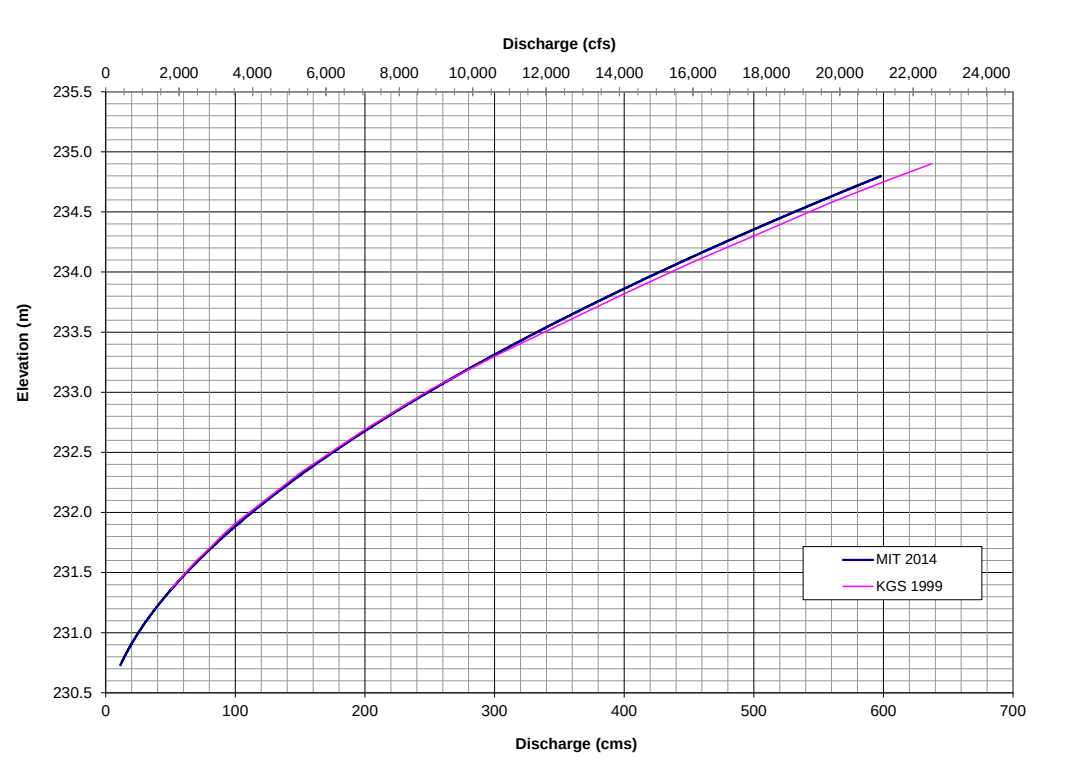
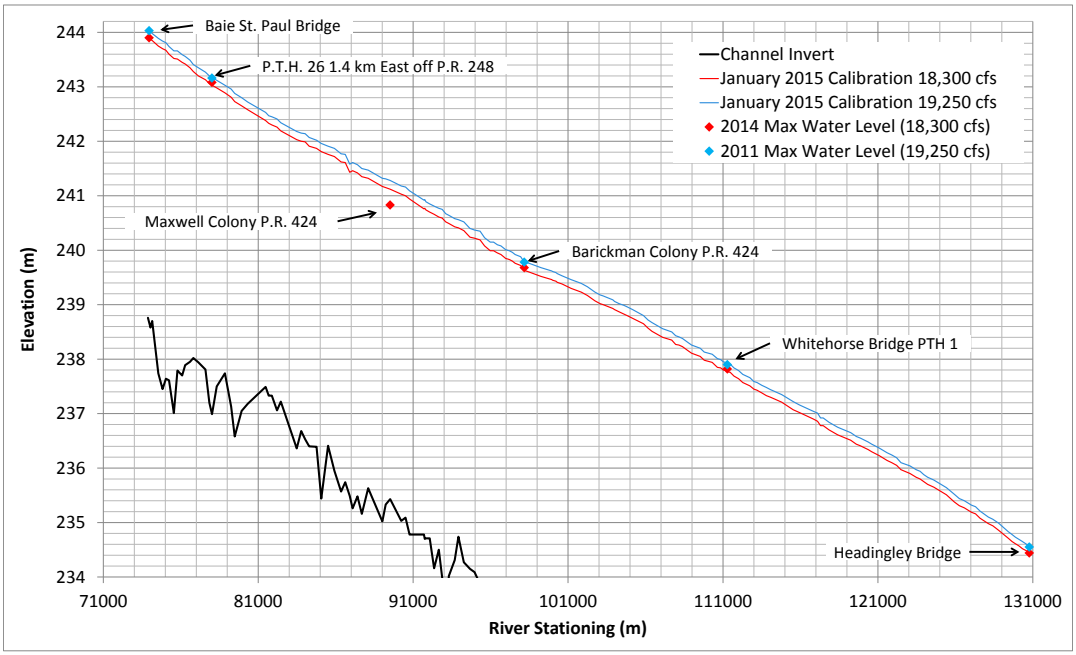


Figure 8.43
2015 Calibration of "Downstream" Model



As shown on Figure 8.43, the results of the 2015 calibration were good, except near Maxwell colony where the model was overestimating the water levels by about 0.2-0.3 m. However, since the higher levels are conservative for the purpose of cost estimating, additional efforts were not undertaken to further re-calibrate the model at this conceptual level of analysis. Although this updated model forms the basis of the updated analyses between Baie St. Paul and Headingley, the Province should consider collecting new bathymetric data in this reach of the river to develop a new and current backwater model, similar to what was done upstream from approximately Portage la Prairie to Baie St. Paul.

8.3.3 Assiniboine River – Brandon

A backwater model developed previously by Bruce Harding as part of a 2007 study for Manitoba Water Stewardship, and calibrated for the City of Brandon AECOM Report (AECOM, 2013) formed the basis of the analyses for the City of Brandon that are later discussed in Section 9.6 of this report. The limits of the backwater model extend for a distance of 30.7 km (19 mi) upstream (river centreline distance) from a location approximately 2.1 km (1.3 mi) downstream of the CN Bridge (Mile 33.6 Carberry Subdivision) just east of Brandon. The backwater model, which is required to enable the accurate calculation of water surface profiles for a river, was developed using the one-dimensional US Army Corps of Engineers River Analysis System HEC-RAS Model.

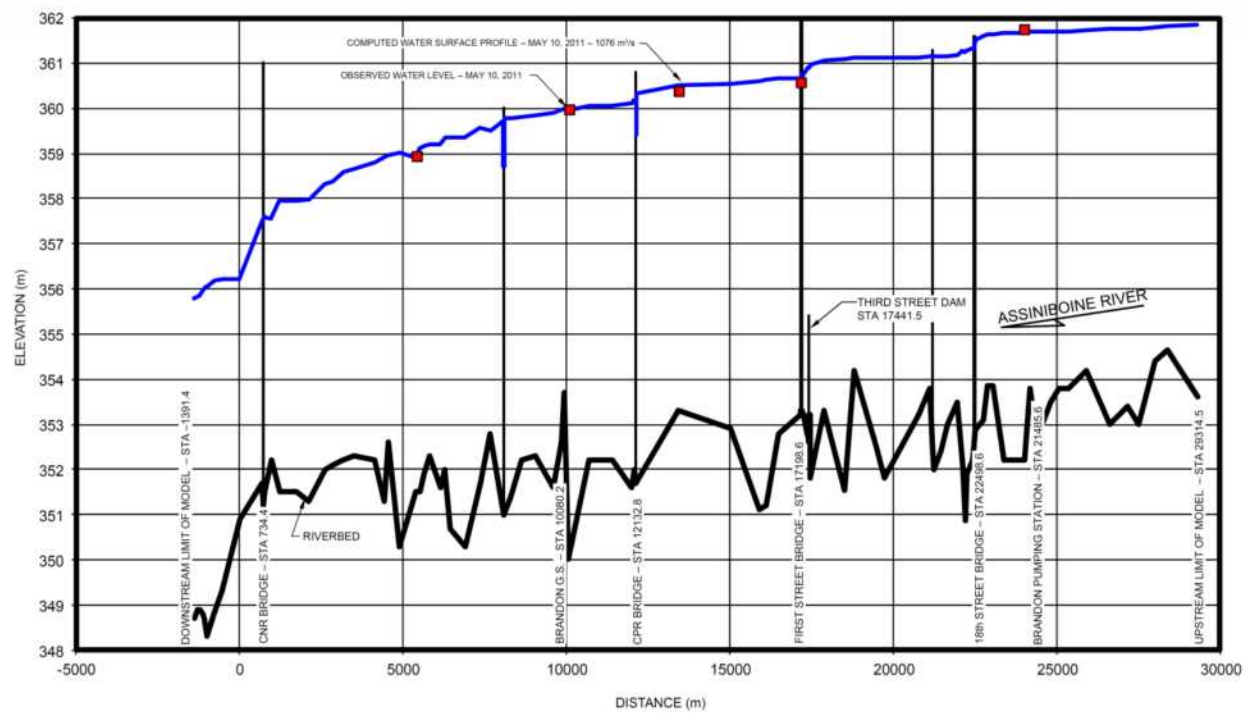
The cross sections for the backwater model were primarily developed using the bathymetric and topographic data collected as part of the 1981 Flood Risk Mapping Study (Manitoba, 1984a). Additional surveyed cross sections were incorporated from many sources including Manitoba Infrastructure and Transportation, the City of Brandon and Wardrop Engineering.

The backwater model also includes all of the present structures (circa 2011) that cross the Assiniboine River within the study area. This includes all of the bridge crossings, the 3rd Street Dam and flood protection dikes (permanent and temporary). The structures included in the model are identified on Figure 8.44.

The City of Brandon dike, located south of the 18th Street Bridge, would have the greatest effect on the hydraulic assessment for this study. In 2011, temporary dikes were also constructed at

PTH 110, 1st Street and 18th Street to prevent overtopping of the road embankments. The other dikes include the PFRA dike (immediately west of the city limits), the Brandon Recreation Centre Golf Course dike and the Kasiurak Bay dike. For purposes of this hydraulic calculation, it was assumed that the City of Brandon dike and the Kasiurak Bay dike would be of sufficient height and stability to contain all flood events. The Golf Course and PFRA dike are low height dikes that provide protection during more frequent floods and were, therefore, modelled such that they would overtop during large flood events.

Figure 8.44
Calibrated Peak 2011 Flood Profiles



The model was not re-calibrated for the current study. Rather, the calibration conducted by Bruce Harding as part of AECOM's report (AECOM, 2013) was adopted without change. Bruce Harding's calibration process involved the adjustment of hydraulic parameters to enable a match between observed and computed water surface profiles for a given flood event. The key parameter adjusted in the calibration phase was the hydraulic roughness coefficient or Manning's "n" value for the cross section, which included the overbank and main channel areas. The 2011 observed data was incorporated into the calibration process, and it was determined that the model required adjustment in the higher flow range (greater than 1%) to ensure that a

good fit was achieved. The water surface profiles estimated by the calibrated model at the peak of the 2011 Flood are presented on Figure 8.44 against observed water levels.

8.3.4 Portage Diversion

An existing backwater model of the Portage Diversion system was provided to KGS Group by Manitoba Infrastructure and Transportation (MIT). The hydraulic model was developed by Hatch using the U.S. Army Corps of Engineers HEC-RAS software and is documented in a report by Hatch (2011a).

Since the model was developed prior to the 2011 Flood on the Assiniboine River Basin, the calibration of the model was checked with hydrometric data collected by KGS Group for MIT at PTH 1 and PR 227 during the 2011 Flood to ensure the validity of the model results. The results showed a good match between the measured and computed water levels. This is shown on Figures 8.45 and 8.46. On this basis, it was determined that the model was representative of existing conditions in the Portage Diversion; and, therefore, no adjustments to the model calibration were necessary for this conceptual study. A profile of the Portage Diversion at its design flow of 708 m^3/s (25,000 cfs) is provided on Figure 8.47. This model formed the basis of the analyses for upgrades to the Portage Diversion later discussed in Section 11.0 of this report.

Figure 8.45
PTH 1 Rating Curve – Surveyed vs Computed Water Levels

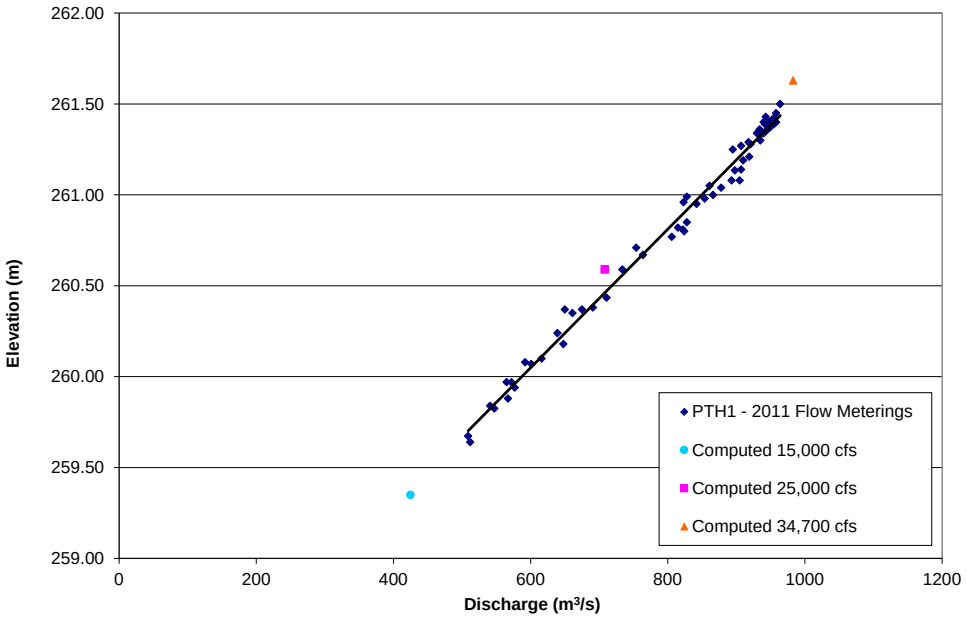
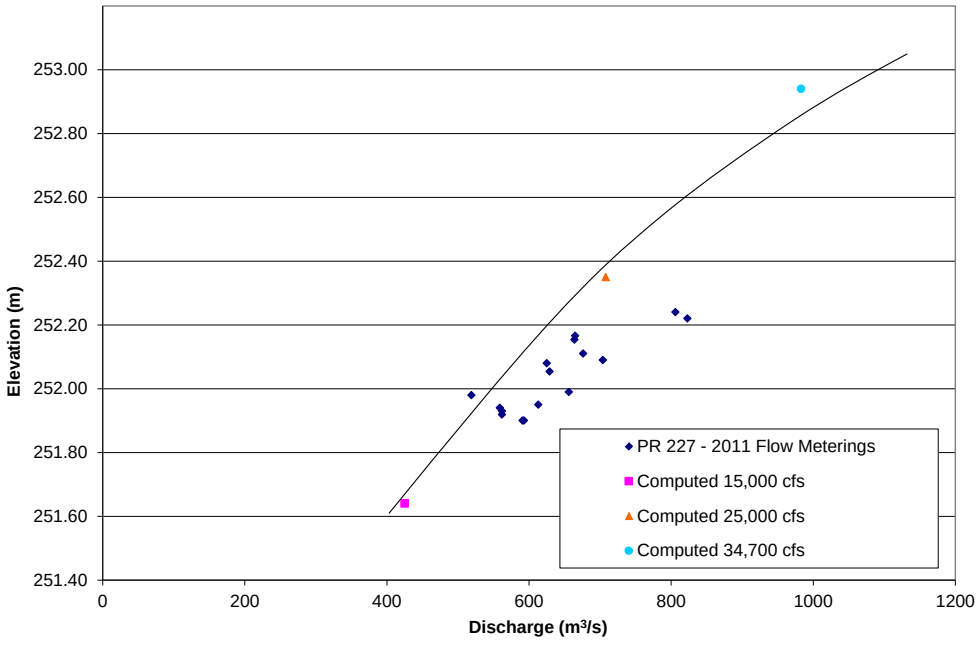
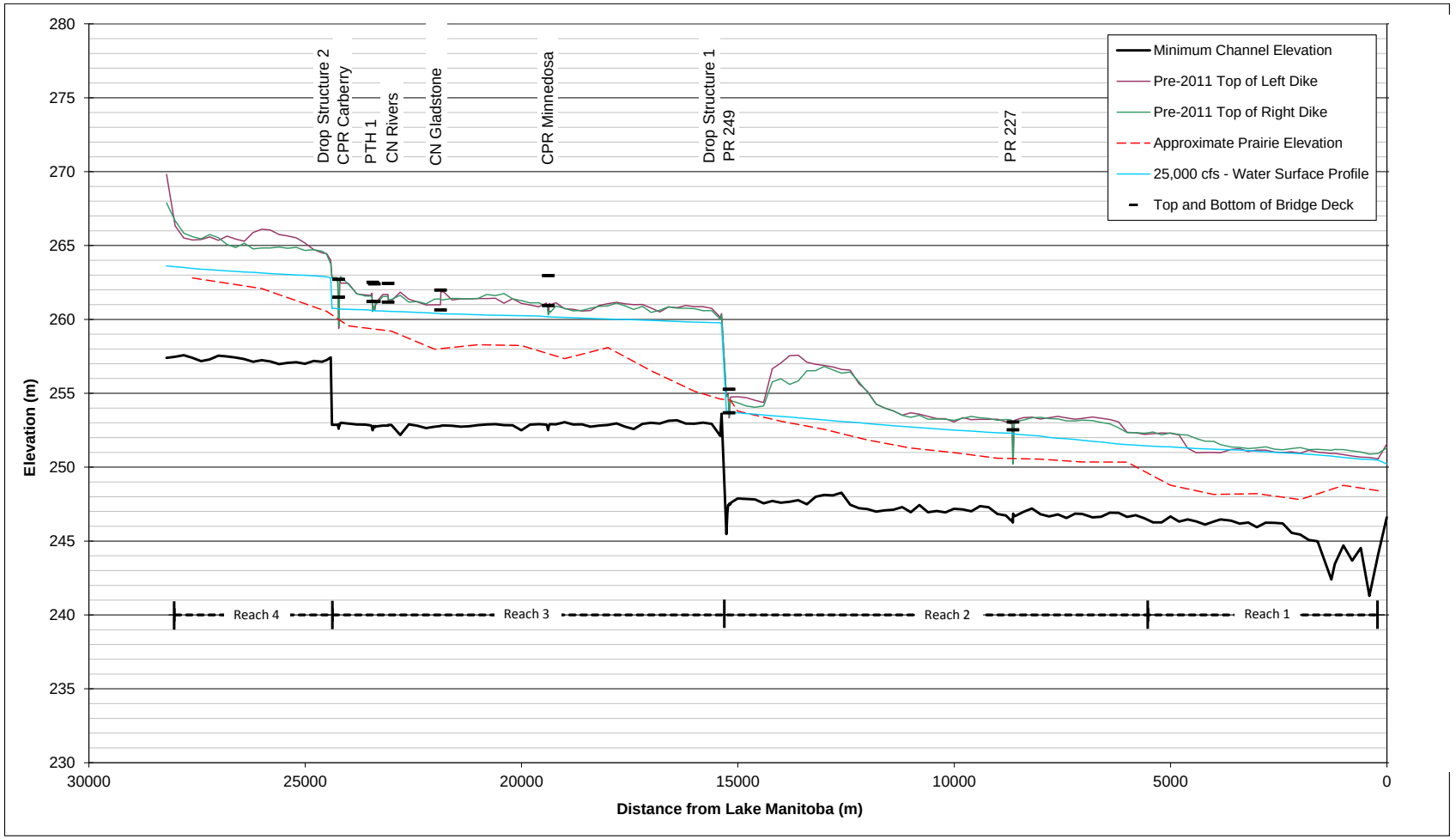


Figure 8.46
PR 227 Rating Curve – Surveyed vs Computed Water Levels



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

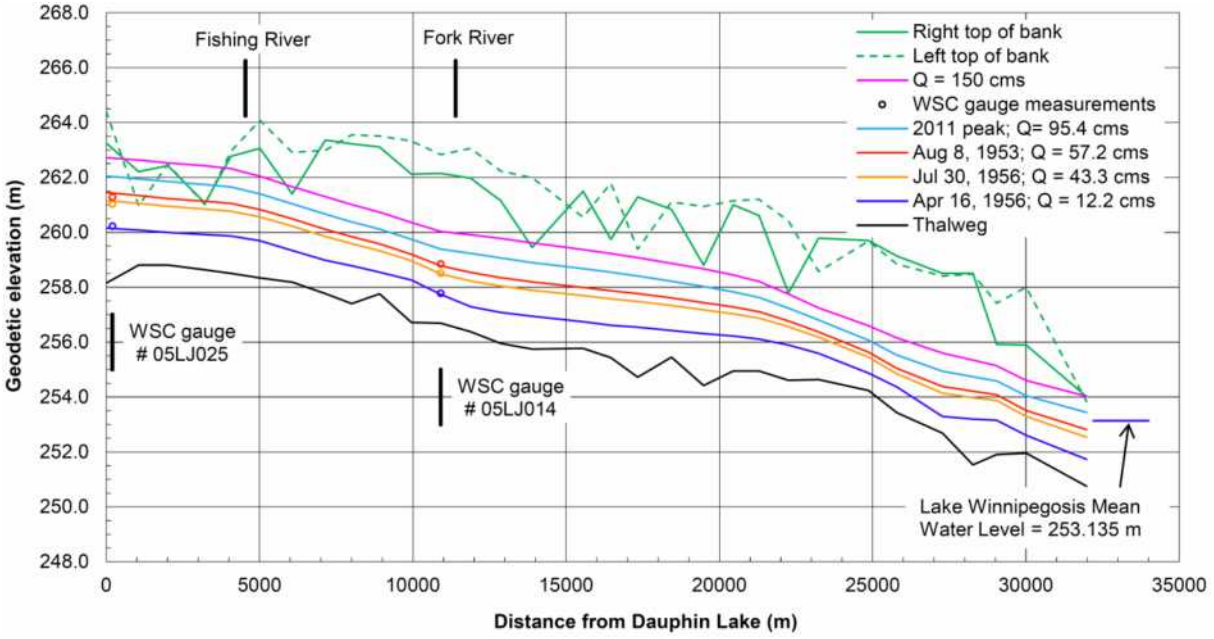
Figure 8.47
Portage Diversion Profile for Existing Conditions



8.3.5 Mossy River

The Mossy River is about 32 km (51 mi) long and conveys water from Dauphin Lake to Lake Winnipegosis. The river bed slope of the upper 25 km (25 mi) of channel is about 0.2 m/km (1.1 ft/mi) while the lower 7 km (11 mi) of the river has a bed slope of 0.40 m/km (2.1 ft/mi) (Figure 8.48). The hydraulic characteristics of the upper 10 to 15 km (16 to 24 mi) of channel immediately downstream of the lake outlet control the outflows from the lake during high lake levels. That is irrespective of the stoplog configuration of the outlet structure at the time. The average width of the channel and mean depth at bank full conditions are 80 m (260 ft) and 2.7 m, (8.9 ft), respectively, in the 10 km (16 mi) long reach below the outlet structure.

Figure 8.48
HEC-RAS Model Calibration Profiles of the Mossy River



Note: Elevations can be converted to feet by dividing metric values by 0.3048.

Flows on the Mossy River have been reported at two gauging locations over the period of record. From 1948 to 1956, the Gauge 05LJ014 was located about 11 km (17.7 mi) downstream of the lake outlet – just upstream of the confluence with the Fork River. Conventional flow measurements were made and rating curves were constructed on a routine basis at this location, although the gauge datum was never referenced to a geodetic datum. From 1957 to the present, the gauge (renumbered to 05LJ025) was located just below the outlet structure.

The largest flow on record was 95.4 m^3/s (3,370 cfs) and occurred in 2011. By comparison, the mean peak annual flow over the period of record is 30.6 m^3/s (1,080 cfs).

The Fishing and Fork Rivers enter the Mossy River from the west 4.3 km (2 mi) and 11.1 km (17.9 mi) downstream of Dauphin Lake, respectively. Peak inflows to the Mossy River from these two tributaries are not coincidental with those on the Mossy River due to the lag effects provided by storage in Dauphin Lake. Therefore, the flows from these two tributaries are not germane to the role that the Mossy River plays in causing high water levels on Dauphin Lake. It is interesting to note also that the flows in the Fork and Fishing Rivers can be occasionally so large as to cause a temporary backflow through the Mossy River Control Structure into Dauphin Lake. Although an interesting phenomenon, this has no influence on the high flood levels on Dauphin Lake.

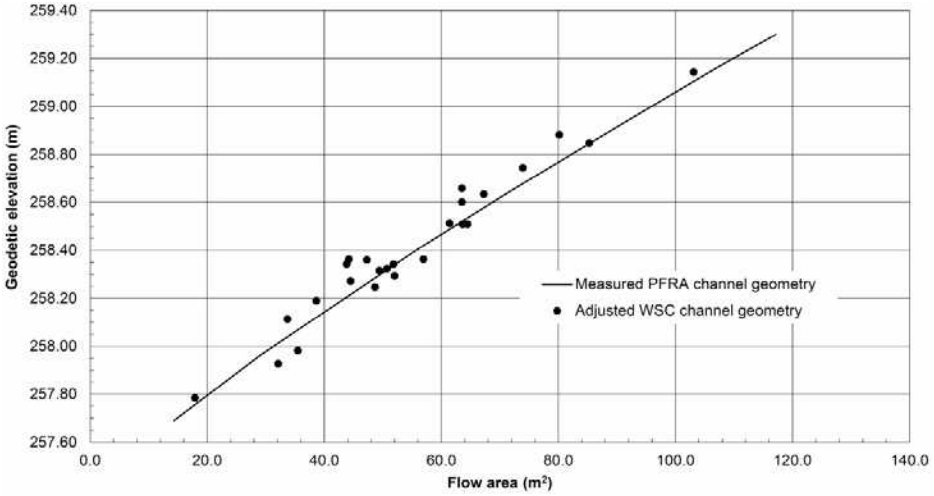
There appear to be no recent bathymetric surveys of the Mossy River. The last systematic survey is believed to have been conducted in 1949 by the Prairie Farm Rehabilitation Administration (PFRA) when 31 cross sections were established at more or less equal intervals along the river between the two lakes. It is assumed that those river cross sections are still reasonably representative of the current channel geometry, and a HEC-RAS model was developed on that basis. The cross sections were digitized from the hard copy charts provided by the Prairie Farm Rehabilitation Administration / Agriculture and Agri-Food Canada (PFRA and AAFC, 2008). The cross section locations were transferred to GoogleEarth from the 1949 location maps. Model boundary conditions were based on water levels on Lake Winnipegosis measured at WSC Gauge 05LH001 and flows provided at WSC Gauge 05LJ025 at the outlet of Dauphin Lake.

Since the water levels at Gauge 05LJ014 had not previously been corrected to geodetic elevations, it was necessary to establish a relationship between:

- The channel cross sectional area (referenced to a geodetic datum) of the PFRA cross sections close to the WSC metering site.
- The local cross sectional area geometry as measured by WSC during numerous discharge measurements and for which the water level was referenced to the gauge datum.

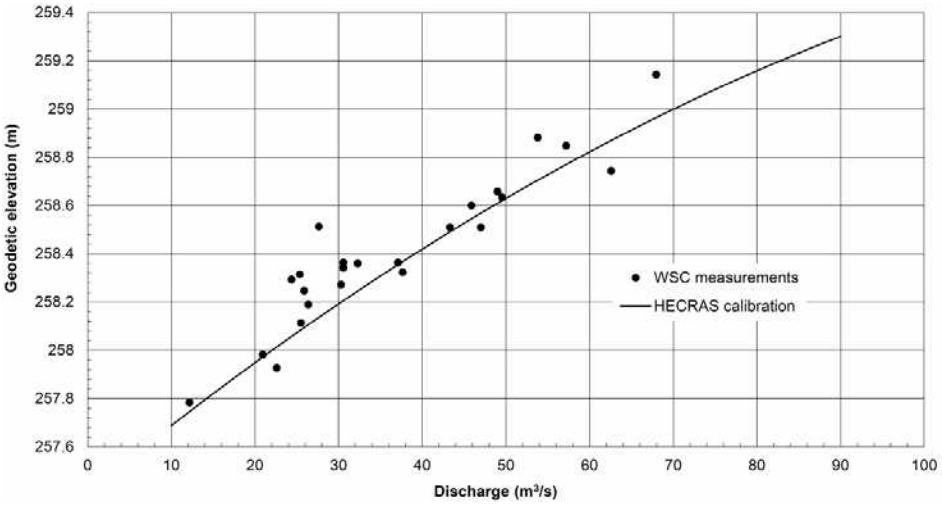
Figure 8.49 shows the comparison between the measured channel geometry of the PFRA cross sections and the WSC cross sectional area adjusted to the appropriate datum. Gauge zero at Gauge 05LJ014 was estimated to be El 229.8 m (753.9 ft).

Figure 8.49
Relationship between Flow Area and Water Level at WSC Gauge 05LJ014



The corresponding rating curve at the gauge adjusted to geodetic datum is shown in Figure 8.50.

Figure 8.50
Rating Curve for WSC Gauge 05LJ014

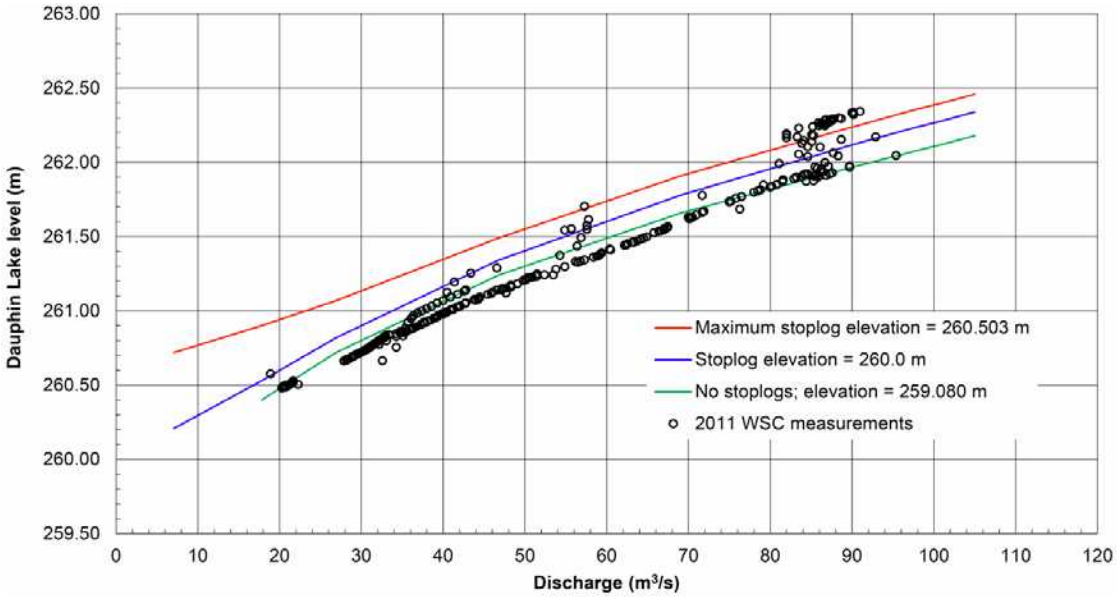


Note: Elevations can be converted to feet by dividing metric values by 0.3048.

The Manning's roughness coefficient of the modeled riverbed was estimated from calibration against measured water levels at the two WSC gauges. The selected roughness coefficient of 0.034 would be applicable over a wide range of high flows. Several simulated profiles are shown in Figure 8.48. The discharge capacity of the Mossy River channel is estimated from the HEC-RAS analysis to be approximately 150 m^3/s (5,300 cfs) with what appears to be only a minor amount of overbank flooding upstream of the Fishing River confluence. Minor diking might increase the discharge capacity further, but this would need to be confirmed with more detailed surveys.

The HEC-RAS model results were also used to generate a stage-discharge relationship downstream of the Mossy Dam that would be independent of the stoplog configuration and reflect only the channel conveyance characteristics (Figure 8.51). That figure also shows the stage-discharge relationships for two scenarios of stoplog removal. Superimposed on the three curves are individual discharge-elevation points that were reported by WSC, but without corresponding information on the stoplog settings. When the stoplogs are removed from the dam, the channel rating curve downstream of the structure reconciles reasonably well with the WSC data for discharges above 20 m^3/s (700 cfs).

Figure 8.51
Rating Curve for the Dauphin Lake Outlet for a Range of Stoplog Sill Elevations



Notes: Water levels can be converted to feet by dividing metric values by 0.3048.
Flows can be converted to cfs by multiplying metric values by 35.3.

8.4 WIND SETUP MODEL OF LAKE MANITOBA

The Monte Carlo approach that was adopted and is described in subsequent sections of this report generated thousands of wind setup events on Lake Manitoba at different locations around the lake. The magnitude of rise of the Lake Manitoba water levels for each event was determined based on the wind setup analysis described below.

8.4.1 Estimation of Wind Setup

A wind setup computer program developed by the Province of Manitoba was used for the analysis. The methodology used in the program follows the procedures in the USACE Shore Protection Manual, 1977 edition (USACE, 1977). The program is a one-dimensional model, which computes wind setup along a user-defined axis at different time increments associated with a given wind speed. The model utilized a finite difference approach of the differential equations of wind setup rather than using the explicit solution (e.g., the Zuider Zee formula). Specifically, the program solves Equations 3-103, 3-104, and 3-105 in the Shore Protection Manual (USACE, 1977). Wind data was provided by the Province of Manitoba.

Wind setup around Lake Manitoba is variable as determined by the variation in wind speed by direction and the variability of the shoreline due to the irregular shape of the lake. To account for this variability, wind setup was computed at 32 locations around the lake. The variability of the wind speed by direction was determined from an annual frequency chart of wind speed data in polar coordinates (wind roses) developed for the South Basin of Lake Manitoba by Custom Climate Services and provided by Manitoba Infrastructure and Transportation. Due to the relatively large size of the lake, it was estimated that a minimum duration of six hours would be required to fully set up the water level. The same chart was also applied to the North Basin. It should be noted that using a wind rose based on seasonal data could yield slightly different results than one based on annual data. However, this difference would likely be negligible and was not considered at this stage of study.

Due to the constriction in the width of the lake at the Narrows, the North and South pools of Lake Manitoba were assumed to be independent and separate wind setup analyses were conducted for each pool. Major and minor axes directions were defined for each pool. The major

axis was defined along the longest length of the lake with the minor axis defined perpendicular to the major axis. Lake cross sections were defined for each pool perpendicular to the axis direction. Winds were applied to the axis if the angle between the directions of the wind and the axis was less than 90 degrees (clockwise and counter clockwise). To compute setups for the four cardinal and four ordinal directions, two axes were used for each pool.

For the north pool, the two axes were north-south and west-east, which consisted of four cardinal directions, namely north, south, west and east. For the south pool, the two axes were the cardinal directions of NW-SE and NE-SW. Cross sections were defined and were evenly placed orthogonally to the axes. Figures 8.52 and 8.53 illustrate the cross section locations for the south and north pools, respectively.

Figure 8.52
Cross Sections on South Pool

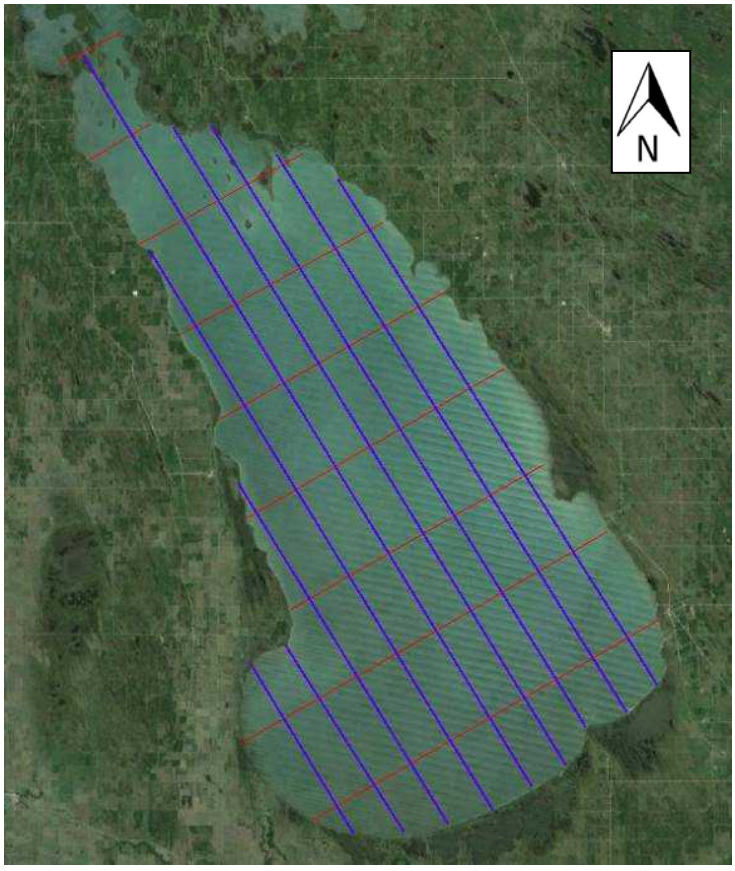
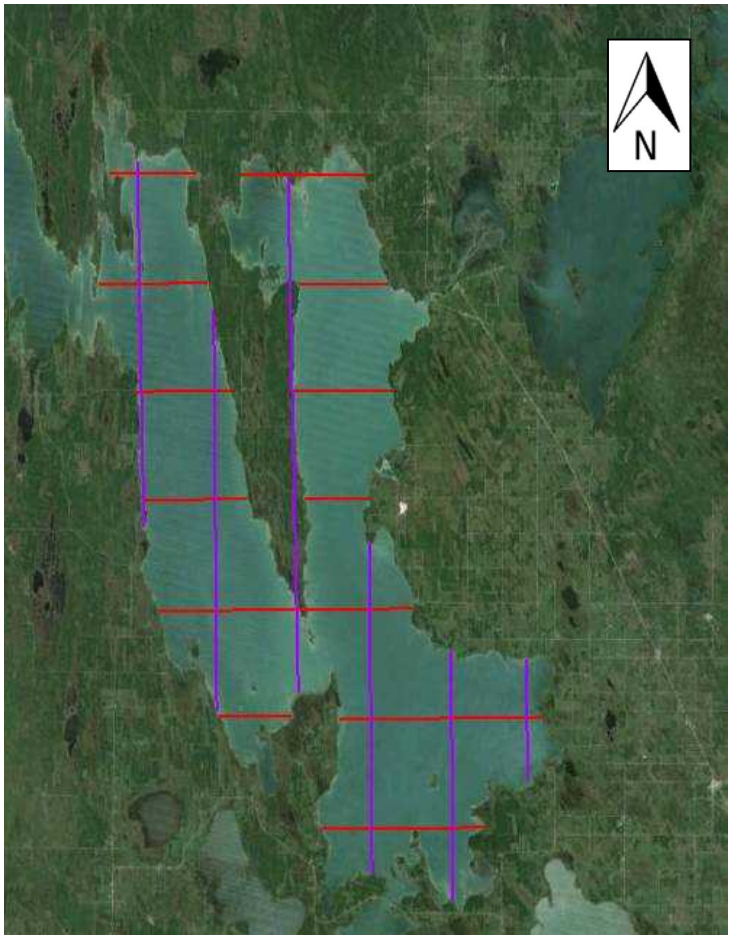


Figure 8.53
Cross Sections on North Pool



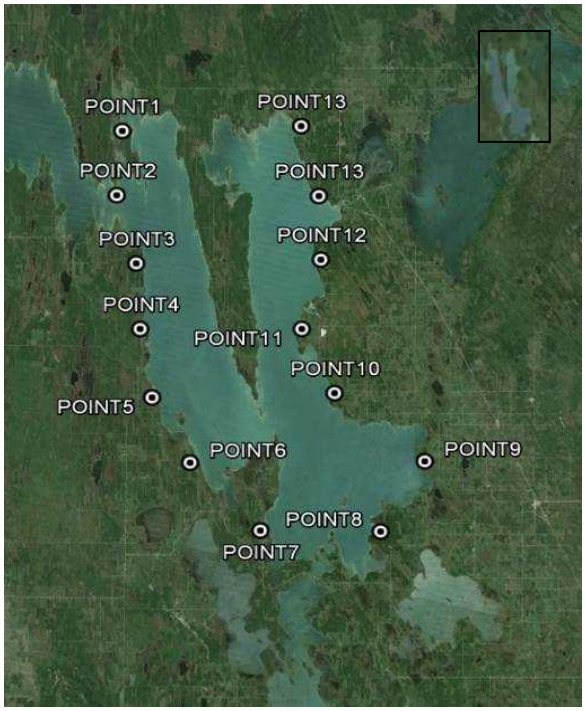
Rectangular cross sections were assumed with the width and depth obtained from the hydrographic maps published by the Canadian Hydrographic Service. Other inputs to the model include: number of cross sections, cross section spacing, computational time step, magnitude of the wind speed for the return period, the still water level and the angle between the wind direction and the axis.

The computed setup at each cross section was assumed to apply equally at all locations along the section. The setups at the opposite shore were, therefore, computed equally for the wind direction that was applied. The maximum setup for the chosen return period at each point along the shoreline was similarly computed for all wind directions, and the maximum value was selected for the setup.

Figures 8.54 and 8.55 show points along the lake shore where the setups were computed.

Figure 8.54
Computation Points on South Pool

Figure 8.55
Computation Points on North Pool



Wind setup typically varies inversely proportionally to the depth of the lake. However, results of the numerical model showed that the variation in possible Lake Manitoba depths did not appreciably affect the computed setup. The wind setup was shown to be much more sensitive to the wind speed and lake geometry rather than to water depth. It was, therefore, assumed for this study that the maximum wind setup was independent of the still water level (SWL). A constant SWL of 247 m (810.2 ft) was selected for all setup computations. For an assumed constant SWL of 249.0 m (816.9 ft) the wind setup was determined to decrease by approximately 0.10 to 0.15 m (0.3 to 0.5 ft), depending on the location around the lake during a wind event with an average 6h wind speed of 75 km/h (47 mi/h). This decrease in water level would be even less for a more frequent wind event with lower wind speeds. On this basis, it was determined that using the SWL of 247 m (810.2 ft) to determine wind setup was a conservative assumption.

8.4.2 Results

At each chosen point (see Figures 8.54 and 8.55), wind setups under all possible combinations of wind speeds and angles between axis and wind were computed. The maximum among the combinations was taken as the maximum wind setup at a given location. Each maximum wind setup is associated with a return period that is the same as the return period of the wind speed, thus the wind setup is fully dependent on wind speed. Figures 8.56 to 8.59 summarize the results of modeled wind setup at each point along the west shore of north pool, the east shore of north pool, the west shore of South Pool and the east shore of South Pool, respectively.

Figure 8.56
Wind Setup on West Shore of South Pool

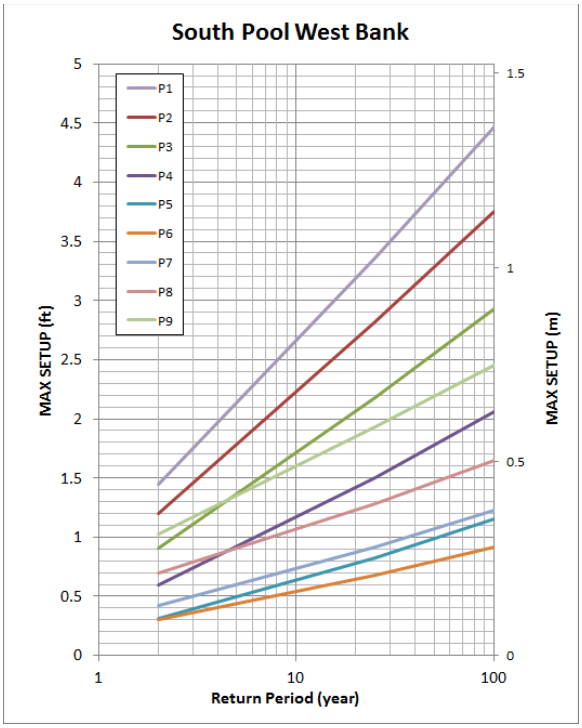


Figure 8.57
Wind Setup on East Shore of South Pool

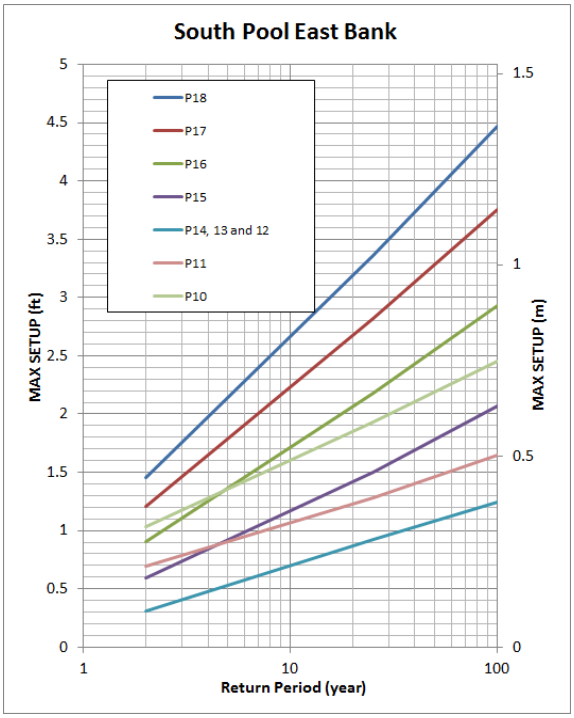


Figure 8.58
Wind Setup on East Shore of North Pool

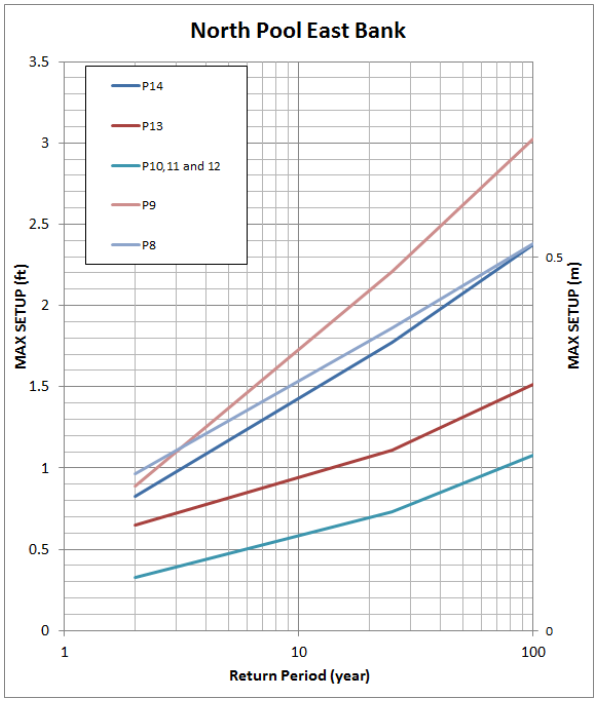
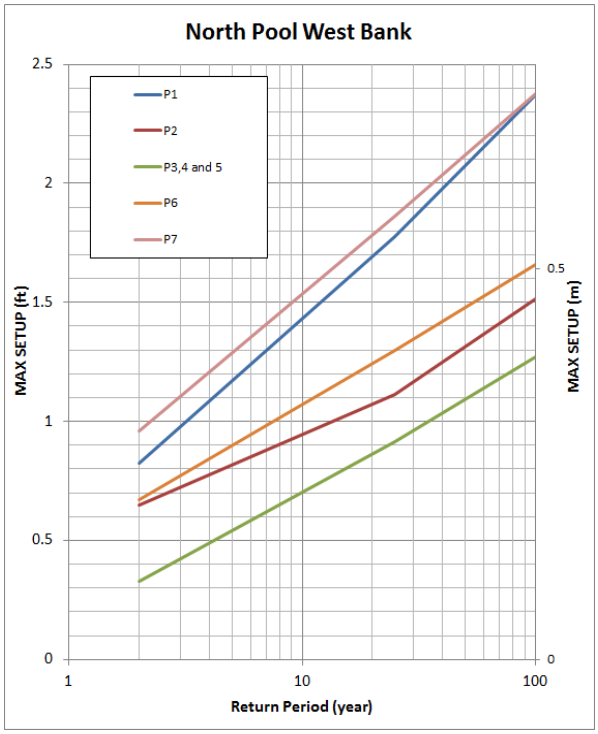


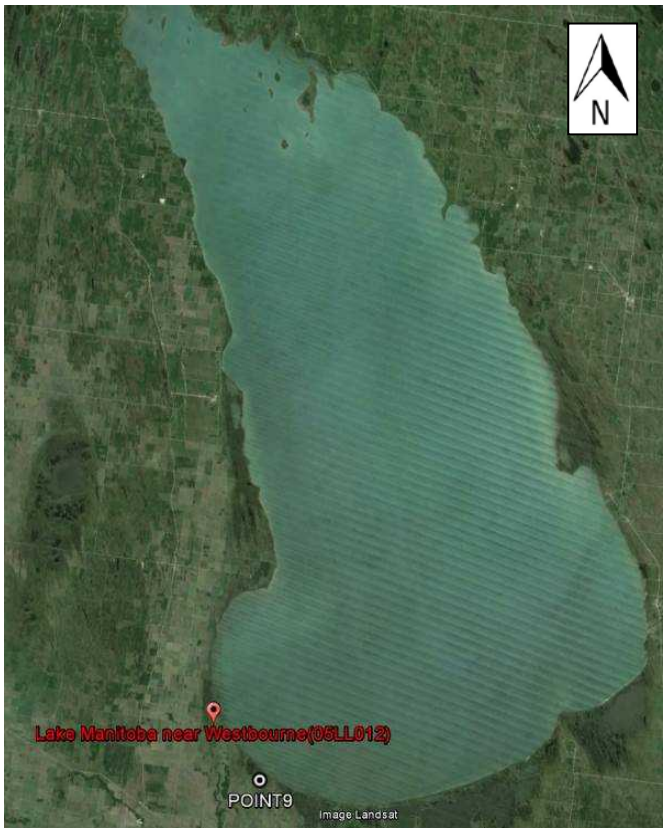
Figure 8.59
Wind Setup on West Shore of North Pool



The results of the model simulations were compared to the recorded setup data on Lake Manitoba at the WSC stations at Westbourne and Steep Rock. The location of the two WSC stations and the closest points of the 1-D model are shown on Figures 8.60 and 8.61.

Figure 8.60
Location of Steep Rock and Point 11 of 1-D Model

Figure 8.61
Location of Westbourne and Point 9 in 1-D Model



The recorded wind setup was estimated at the two stations by subtracting the recorded average daily lake levels from the normalized lake level. This method of calculating wind setup is an approximation as it uses average daily levels instead of instantaneous levels. A frequency analysis was then computed on the recorded wind setup. Table 8.7 summarizes the results of the frequency analysis and the computed wind setup using the 1-D model.

Table 8.7
Comparison of Modelled and Recorded
Wind Setup at Steep Rock and Westbourne

North Pool Wind Setup (m)			
	2 year	25 year	100 year
Steep Rock	0.10	0.17	0.21
1-D model(Point 11)	0.08	0.22	0.32
South Pool Wind Setup (m)			
Westbourne	0.33	0.64	0.81
1-D model(Point 9)	0.33	0.59	0.75

Note: Water levels can be converted to feet by dividing metric values by 0.3048.

As shown on Table 8.7, the computed wind setup values compared reasonably well to the wind setup calculated from recorded water levels, considering that both methods are only approximations of what the actual wind setup could be under a series of assumed conditions. In addition, the results of the frequency analysis showed that the computed wind setup values fitted a lognormal distribution. A one sample KS test used to test this assumption passed with high p values equal to 0.91 and 0.88, respectively.

8.4.3 Wave Uprush

The magnitude of wave uprush on a structure depends on numerous variables such as shoreline conditions, distance from the shoreline, steepness of slope, depth of water and slope roughness, which are all unique to the shoreline location. In addition, this data is not available for every single structure at risk of flooding around Lake Manitoba. Simplistic assumptions and approximations were, therefore, required for the estimation of wave uprush to be used for the combined wind and wave effects water level for use in the damage assessment in this study.

A single value for the wave uprush was used to represent the wave uprush around the lake, regardless of the location. A number of wave uprush conditions were determined for different wave conditions on Lake Manitoba. The wave characteristics were determined based on methods outlined in the USACE 1984 Shore Protection Manual (USACE, 1984). Wave uprush was determined based on the USACE momentum flux equation. The results showed that, on average, the magnitude of wave uprush was approximately equal to the magnitude of wind

setup (i.e., 0.5 m (1.6 ft) of setup translated also to 0.5 m (1.6 ft) of wave uprush). It was determined that the average contribution of wind setup to the combined water level (all events included) ranged between 0.6 m to 0.75 m (2 to 2.5 ft). A conservative wave uprush value of 0.7 m (2.25 ft) was selected. The application of wind setup in the Lake Manitoba damage model is discussed in in Section 8.6.2.

For comparison purposes, the computed wave uprush values were compared with wave uprush values determined by the Province of Manitoba for the calculation of the Flood Protection Level on Lake Manitoba as outlined in the February 16, 2012 memorandum entitled, "Flood Protection Level Determination – Lake Manitoba South Pool" (MC&WS, 2012a) and in the February 21, 2012 memorandum entitled "Flood Protection Level Determination - Lake Manitoba North Pool" (MC&WS, 2012b). The average wave uprush was determined to be 0.65 m (2.1 ft) in the memorandum for the South Pool. These results concur with the 0.7 m (2.3 ft) wave uprush value selected for this study. Furthermore, the results of the Province of Manitoba memorandum for the North Pool (MC&WS, 2012b) showed that the average wave uprush on the North Pool on Lake Manitoba was less than on the South Pool. The assumption of 0.7 m (2.3 ft) for the wind uprush for both north and south pools is considered to be conservative.

8.5 SYNTHETIC GENERATION OF FLOOD EVENTS

8.5.1 Upper Assiniboine River and Lake Manitoba Local Inflows

One of the most respected and experienced authorities in the world on analysis and planning of flood protection systems is the U.S. Army Corps of Engineers (USACE). It has become standard policy of the USACE to examine the effects of risk and uncertainty in evaluation of the performance of potential flood mitigation works. The usual approach by the USACE is to apply the HEC-FDA ("Hydrologic Engineering Center – Flood Damage Analysis") software package. That package incorporates the concept of uncertainty into the estimation of hydraulic performance and the "expected annual damages" (EAD) that form the cornerstone of the economic analysis.

The FDA software uses a "Monte Carlo" procedure to generate a large representative dataset of river flows that lead to the estimation of the EAD. This is a technique wherein large numbers of

synthetic years of flood events are generated using the statistical characteristics of flood magnitude, and the relationship between flood flow and water levels, and ultimately the impact on flood damages. Thousands of years of flood events, water levels, and potential flood damages are typically generated by the numerical model so as to develop an average that is truly representative of the diversity of river conditions that could be expected to occur. This process develops a realistic representation of the frequency of flood damages for large floods that improves upon the limitations of estimating directly from the observed record alone.

The technique incorporated in FDA was used in the studies and planning of the expansion of the Red River Floodway in 1999-2000 by KGS Group (2000a). It was viewed as a rigorous, comprehensive technique that would be well suited and justified for use in the evaluation of flood protection options for Lake Manitoba as well. A difficulty in applying this technique, however, is that the flooding of Lake Manitoba is caused by two separate sources (Assiniboine River via transfers through the Portage Diversion, and the local drainage basin that supplies inflow to Lake Manitoba). This issue of dual sources of flood waters does not lend itself well to the direct application of the standard FDA software.

The procedure adopted for the study reported herein was as follows. A Monte Carlo analysis was first used to generate a large number of flood volumes of the Assiniboine River and the local Lake Manitoba inflow. This formed the stochastic dataset of flood events in each basin. This generation of random flood volumes for the two basins was based on the statistical distribution and correlation of the observed volumes in the historical record.

The statistical distributions of observed Assiniboine River and local Lake Manitoba inflow volumes discussed in Section 8.3.1 of the report were selected as lognormal and Gumbel distributions, respectively, based on a visual comparison of the “goodness of fit” of the theoretical distribution to the actual recorded data. Comparisons of the theoretical distributions with the empirical distributions of the observed volumes are shown in Figures 8.62 and 8.63.

Figure 8.62
Statistical Distribution of Assiniboine River Volumes

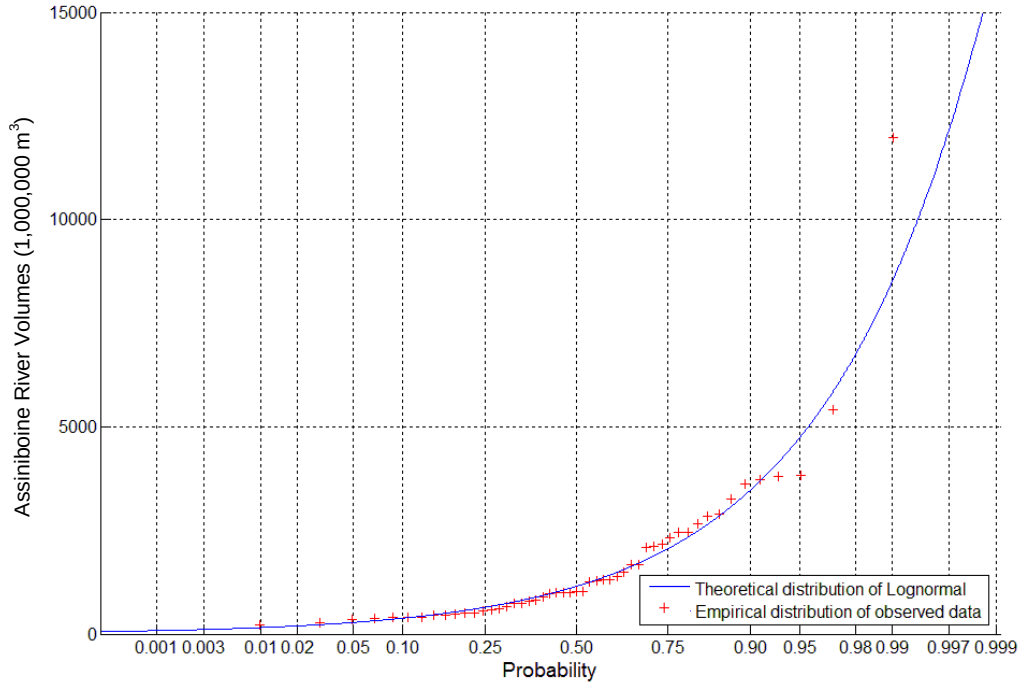
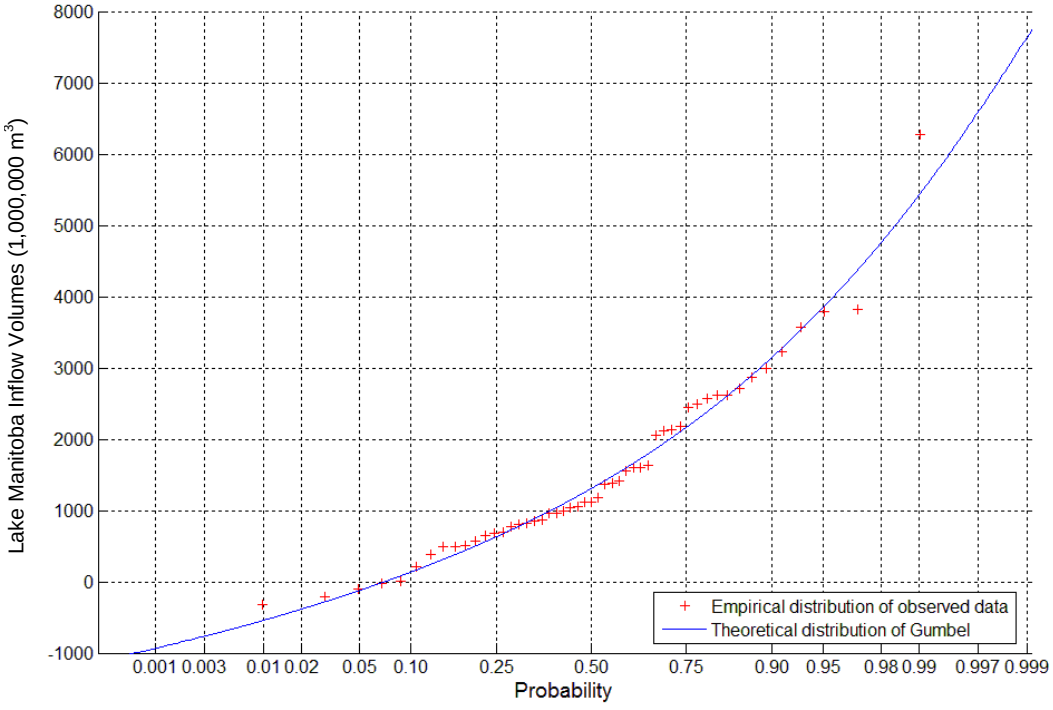


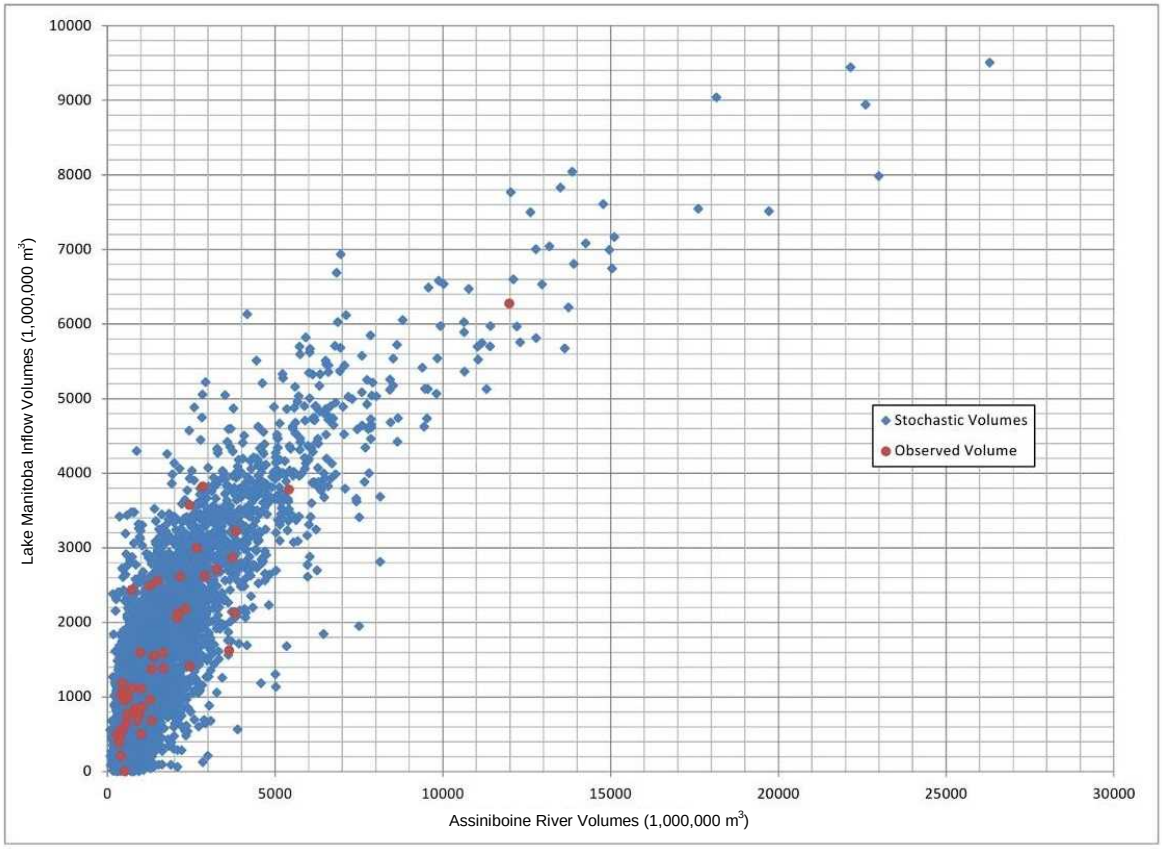
Figure 8.63
Statistical Distribution of Local Lake Manitoba Inflow Volumes



The generation of the 5,000 synthetic combined flood volumes was done using the MATLABTM software.

A comparison of volumes generated from the Monte Carlo model and the observed historic volumes in the two basins is shown on Figure 8.64. The figure shows that the observed and stochastic volumes follow a similar trend, as depicted by the polynomial fit of the two different data series.

Figure 8.64
Comparison between Stochastic Volumes and Observed Volumes

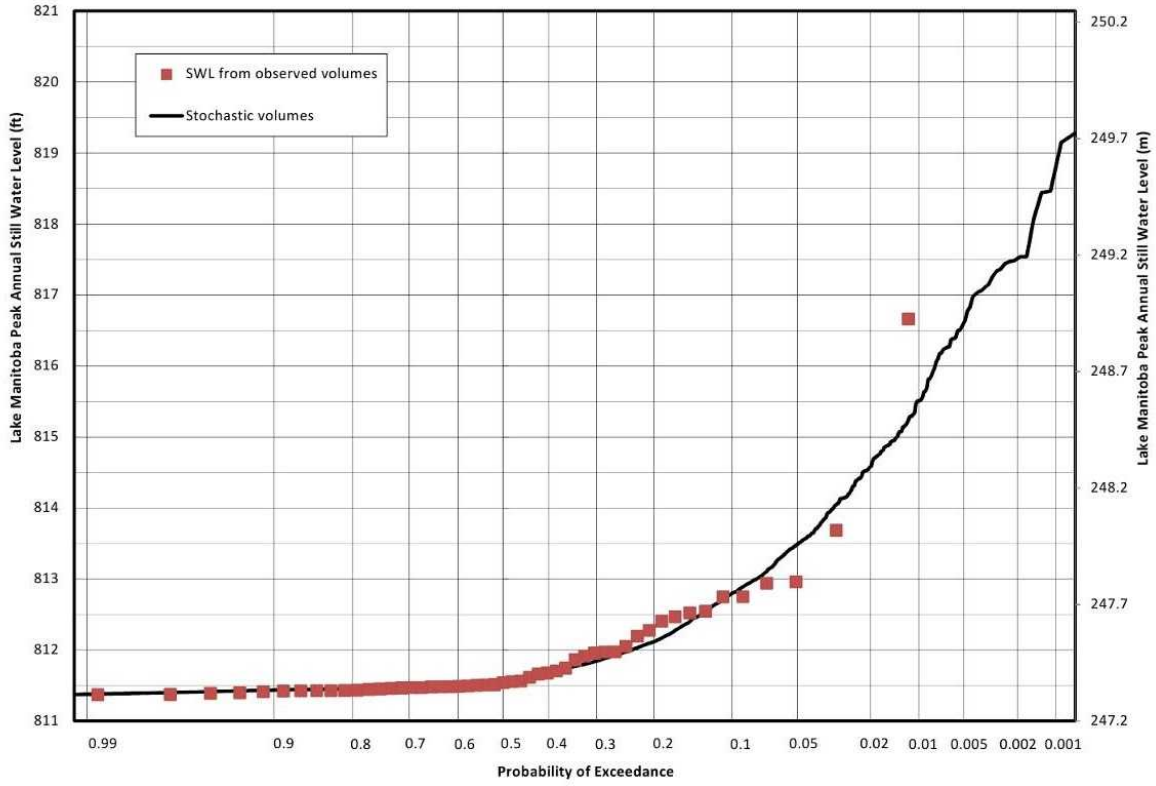


For each synthetically generated flood event, the peak Lake Manitoba water level was determined based on the results of the routing model using the relationship between Lake Manitoba water levels and volumes of the Assiniboine River and of local Lake Manitoba inflows shown in Figure 8.14 (Section 8.2.2.3). A frequency curve of the computed peak water level was

then generated to obtain the probability of exceedance of still water levels (SWL) on Lake Manitoba.

The frequency curves of the stochastic SWLs compared to the SWLs interpolated from observed volumes are shown on Figure 8.65.

Figure 8.65
Computed Frequency Curve of Still Water Levels on Lake Manitoba Compared to Levels from Observed Values



In addition, for comparative purposes, and to examine how the synthetic water levels compared with other assumptions, other frequency curves were developed for this study. These represent the SWL of Lake Manitoba for the following separate hypothetical scenarios:

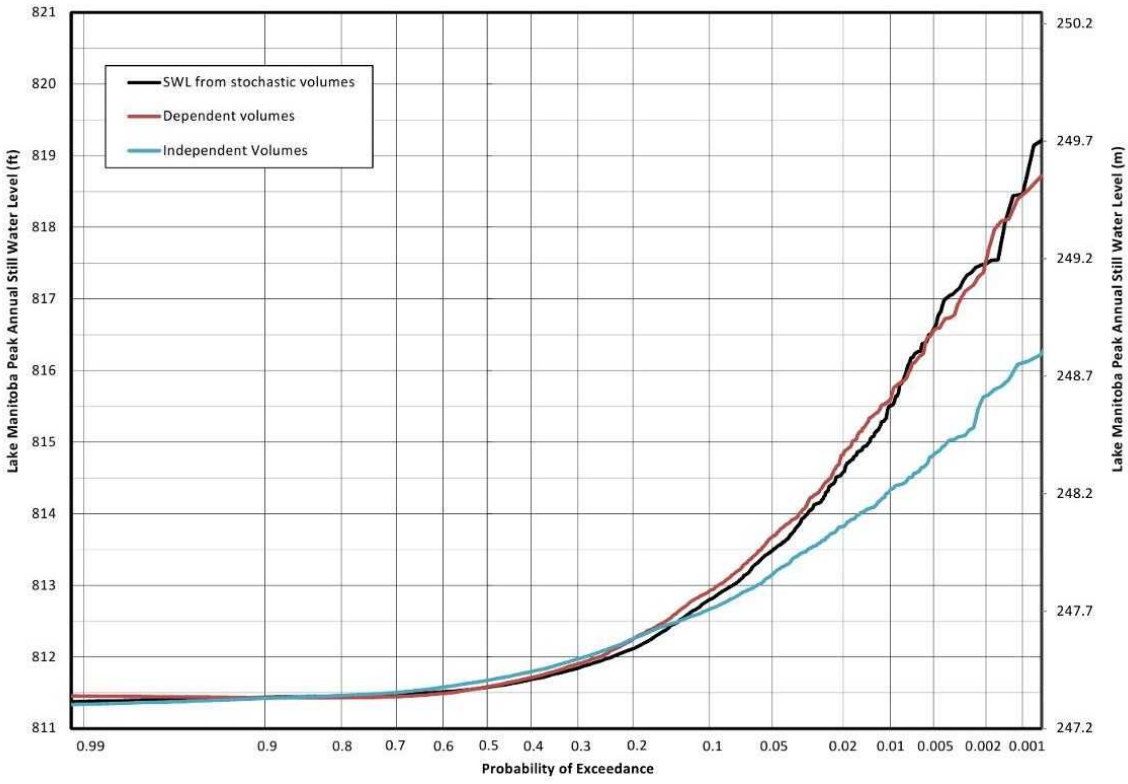
- Lake Manitoba inflow volumes and Assiniboine River flood volumes exactly as recorded from 1961 to 2011. Only 51 data points were available.
- Lake Manitoba inflow volumes and Assiniboine River flood volumes were assumed to be fully statistically dependent (e.g., an event with a Lake Manitoba flood volume with an annual probability of being equal or greater than X% will always combine with an

Assiniboine River Flood Volume with an annual probability of being equal or greater than X%).

- Lake Manitoba inflow volumes and Assiniboine River volumes were fully statistically independent (i.e., no statistical correlation whatsoever between flood volumes in each basin exists).

The frequency curves of the stochastic SWLs compared to the hypothetical SWLs under conditions of statistical dependency and statistical independency are shown for comparative purposes only on Figure 8.66.

Figure 8.66
Computed Frequency Curve of Still Water Levels for Lake Manitoba Compared to Conditions of Statistical Dependency and Independency



8.5.2 Wind Setup on Lake Manitoba

A Monte Carlo analysis was also used to generate wind setup events that coincide with the 5000 synthetically generated flood events that were described in Section 8.5.1. Stochastic simulation of wind setup with a Monte Carlo approach requires knowledge of the distribution of

the wind setup data. A lognormal distribution was assumed based on the results of the Wind Setup Analysis discussed in Section 8.4.2. Parameters of lognormal distribution were then calculated for the previously computed wind setup at each of the 32 point locations (Figures 8.54 and 8.55). These parameters were used to populate stochastic wind setups at each location.

The “Combined Water Level” (CWL) was then calculated at each of the 32 point locations for the synthetically generated flood events. As discussed in Section 8.4.1, changes in SWL do not cause significant changes in wind setup. The SWL and wind setup were, therefore, assumed to be statistically independent events. This assumption allowed the generation of SWL and wind setup with independent random generation techniques, and the summation of the generated values to determine the CWL for a particular synthetic event.

Frequency plots were then developed based on stochastic CWL at the north and south pools, respectively. Representative frequency curves at three point locations on the north and south pools are shown in Figures 8.67 and 8.68. The CWL frequency curves show that the CWL on Lake Manitoba varies around the lake due to the varying contribution of the wind setup for a given return period.

Figure 8.67
Frequency Curve of CWL at North Pool

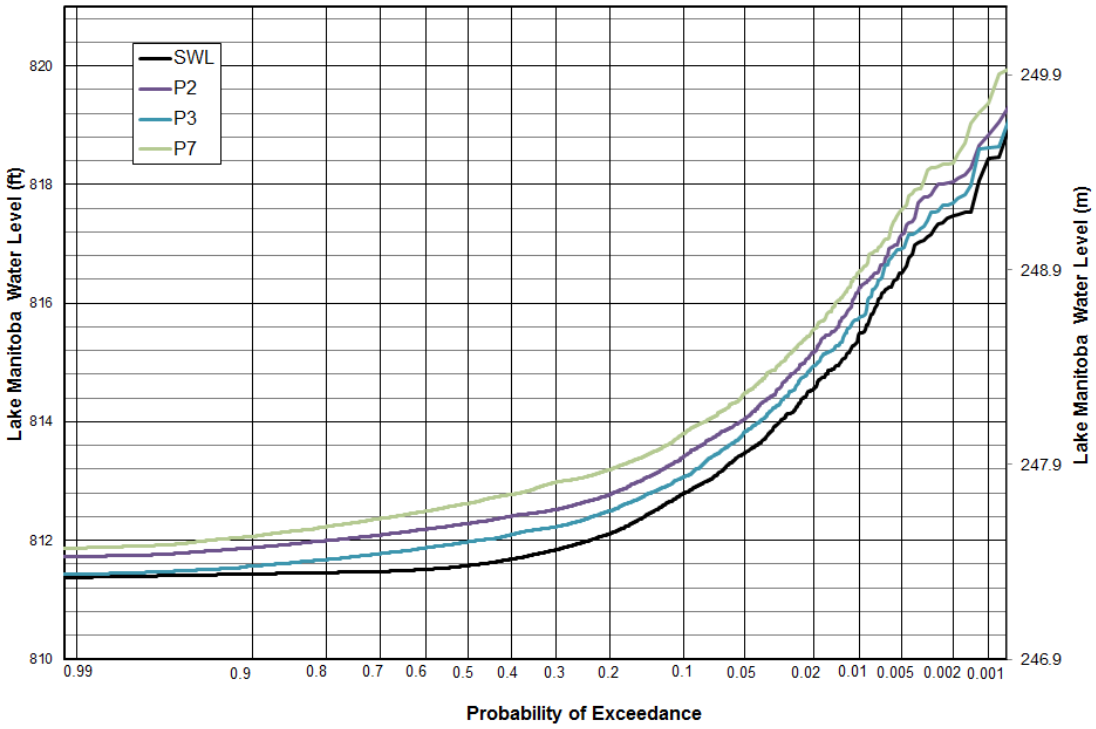
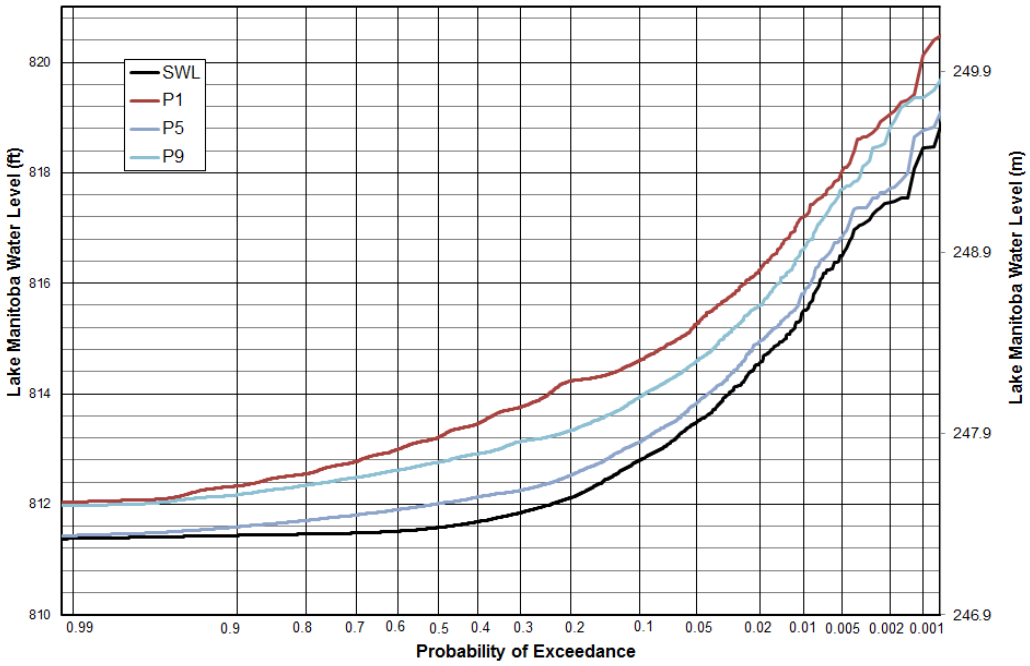


Figure 8.68
Frequency Curve of CWL at South Pool



8.5.3 Assiniboine River and Red River

A comprehensive review of the flood characteristics of the Red and Assiniboine Rivers was undertaken as a basis for developing the best strategy for generating synthetic flood events that would be statistically representative of the two rivers. Section 5.3.2 of the report contains the analyses undertaken and the conclusions developed. In summary, the results were as follows:

- There appeared to be no significant statistical correlation of flood peaks in these two rivers for above normal events. There was roughly an equal chance that the flood flow in the Assiniboine River would be below average when the Red River was experiencing an abnormally large flow, and that it would be above average for the same flood in the Red River. In statistical terms, for above average flood events, the flood flows in the two rivers are considered to be statistically independent.
- There appeared to be some weak correlation between the flood events in the two rivers for moderate to low flood magnitudes. Synthetic generation of flood events in this range of magnitude should, therefore, consider that there was some weak statistical dependency.

8.5.3.1 Technique for Synthetic Generation of Stochastic Flood Flows

The so called ‘Monte Carlo’ procedure was used to simulate pairs of the stochastic flood flows in the Assiniboine and Red Rivers. Simulations were based on the characteristics of existing flow records including the statistical distribution of each river and the statistical correlations of flood events in both rivers.

Ideally this would be based on state of nature hydrographs for both rivers, with the appropriate consideration of the timing of the flood peak in Winnipeg. In other words, the fact that the annual peak flows of the two rivers do not normally coincide in time should ideally be recognized. However, accurate recognition of this fact and the effects of the mitigation schemes on the Assiniboine River throughout the flood hydrograph for each year would require a level of engineering effort that could not be supported within the budget of the current study. Consequently, a conservative approach was adopted that considered the peaks of the two rivers would always coincide in time. This assumption would clearly set an upper limit on the potential effects of the Assiniboine River works on the stakeholders in Winnipeg.

The analysis was further complicated by the fact that the state of nature flow data for the Red River supplied by MIT for this study included flood peaks that occurred in some isolated years prior to the main record of consecutive flood peaks that began in 1892. Proper statistical recognition of these isolated floods would require a level of sophistication and engineering effort that could not be justified in the context of this conceptual level of study. Furthermore, although the isolated floods in the 1800s were quoted as specific best estimates of the annual peaks in those isolated years, KGS Group's understanding was that these flood estimates were by no means accurate. For example, the flood peak of the Red River in 1826 has been quoted repeatedly in various references to be 6,370 m^3/s (225,000 cfs), based on analyses done during the Royal Commission on Floods in Winnipeg in 1958 (RC, 1958). However, KGS Group's review of that data showed that this flood peak was simply an average of two widely divergent calculations, and the true flood peak in 1826 could not be determined. KGS Group speculated that it could lie within a range from 5,100 to 8,500 m^3/s (180,000 to 300,000 cfs) (Winnipeg Flood Protection and the Fleming Conundrum, Carson et al., 2002). Given the inaccuracies with the flood peaks of these isolated years, and the engineering complexities of actually incorporating them properly into the analysis, an approach was taken to approximate the statistical parameters needed for the stochastic analysis.

This approximation was achieved by first defining a set of estimated flood peaks using MIT's estimated "state of nature" flows for the Red River upstream of the Forks, combined with WSC records for the Assiniboine River at Holland. WSC's records were from the period of 1961 to 2011 and were regulated flows; MIT's records were from the period of 1892 to 2011 and were not regulated (regulation effects were previously extracted by MIT).

As discussed in Section 5.3.2, there appeared to be a weak correlation of flood peaks in lower to moderate flood events in the two rivers. On the other hand, there appeared to be no significant correlation of flood peaks in the two rivers for above normal events. Breakpoints in peak flows were selected that would separate the low floods wherein some weak correlation occurred, from moderate to large floods wherein no correlation was evident. These breakpoints in the Red and Assiniboine rivers' peak flows were selected to be 1,750 m^3/s (61,800 cfs) and 600 m^3/s (21,200 cfs), respectively. If the flow on the Red River is below 1,750 m^3/s (61,800 cfs), it is more likely to occur simultaneously with a flow in the Assiniboine River that is less than 550 m^3/s (19,400 cfs). On the other hand, if the flow on the Red River is larger than

1,750 m^3/s (61,800 cfs), it has an equal chance of occurring with flows of any magnitude in the Assiniboine River. Above 1,750 m^3/s (61,800 cfs), the flood flows on the two rivers were considered statistically independent.

A model describing the relationship of the flood peaks was built in MATLAB, as a basis for simulating stochastic peaks. The key parameters of this model were based on the statistical distribution of both rivers and the correlation of their low flows (as defined by the limits described above). The correlation coefficient of the observed low flows of the Red and Assiniboine Rivers was 0.42, indicating a moderate degree of correlation between the two. The MATLAB model used the Copula method available within MATLAB to generate stochastic events (Red and Assiniboine peak flows) that were reflective of this moderate degree of correlation.

The statistical distribution of the flood peaks in the Assiniboine River were best represented by a Log-Pearson Type lll distribution, based on a visual examination of the curve fit to the observed dataset. The graphical displays of the theoretical probability distribution and that of the observed and stochastic data for the Assiniboine River are shown in Figure 8.69.

The statistical distribution of the flood peaks in the Red River were best represented by a Log-Pearson Type III distribution, based on a visual examination of the curve fit to the observed dataset. The graphical displays of the theoretical probability distribution and that of the observed and stochastic data for the Red River above the Forks are shown in Figure 8.70.

The MATLAB model used the Copula method to generate stochastic events (Red and Assiniboine peak flows) that were reflective of not only this moderate degree of correlation but also the marginal distribution of each original dataset. The MATLAB model was used first to simulate the peak flows in each river, as if they were correlated based on the relatively weak relationship described above. The simulated peak flows that turned out to be above the pre-selected break points between low and high flows were then simulated again. However, the second simulation used the same statistical distributions but different random probabilities for each river. This step eliminated the correlation of the high peaks while still ensuring that their probability distributions were representative of each river.

Figure 8.69
Probability Distribution for Assiniboine River Flows

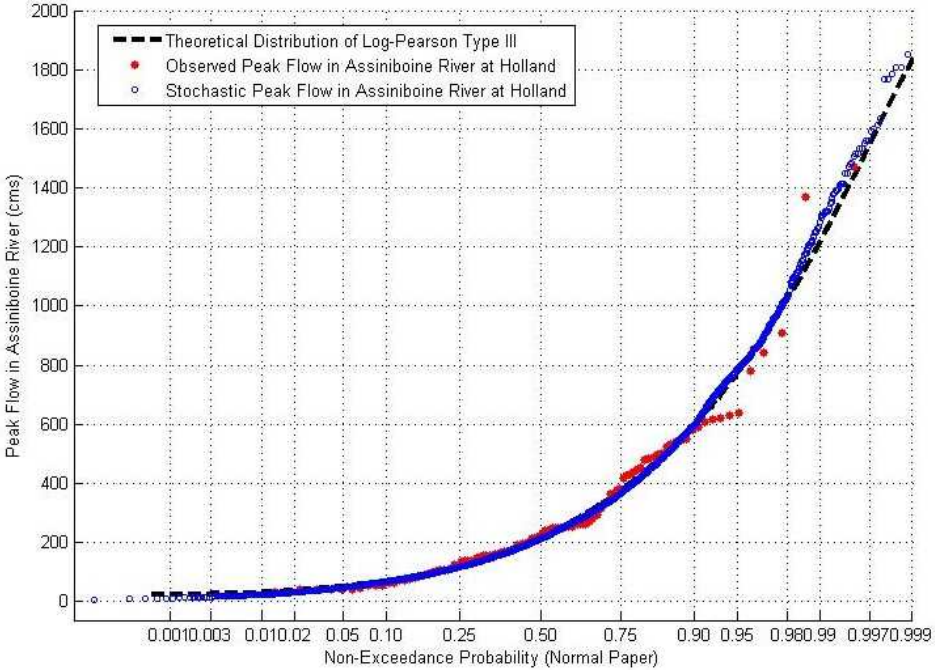
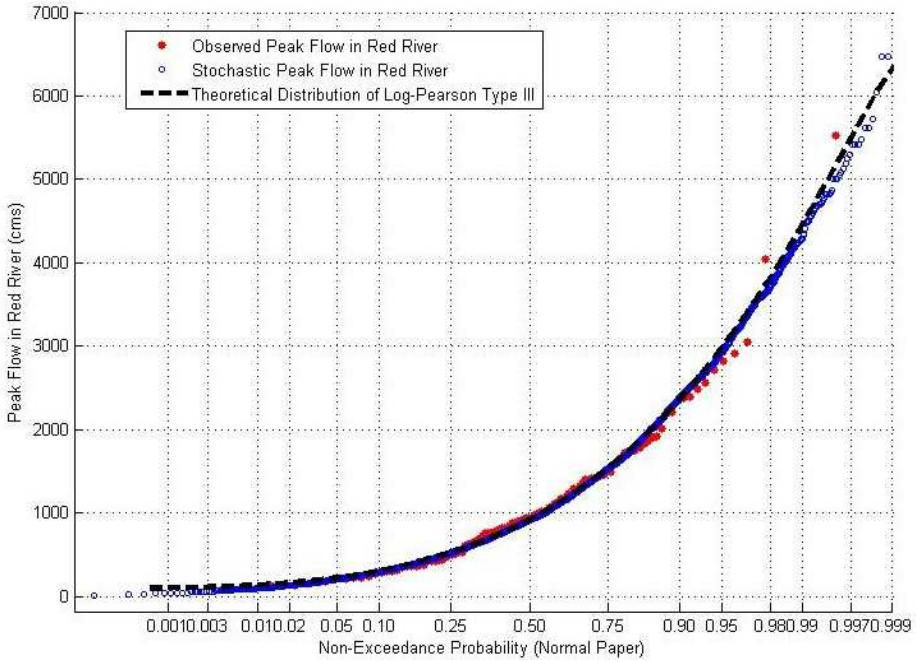


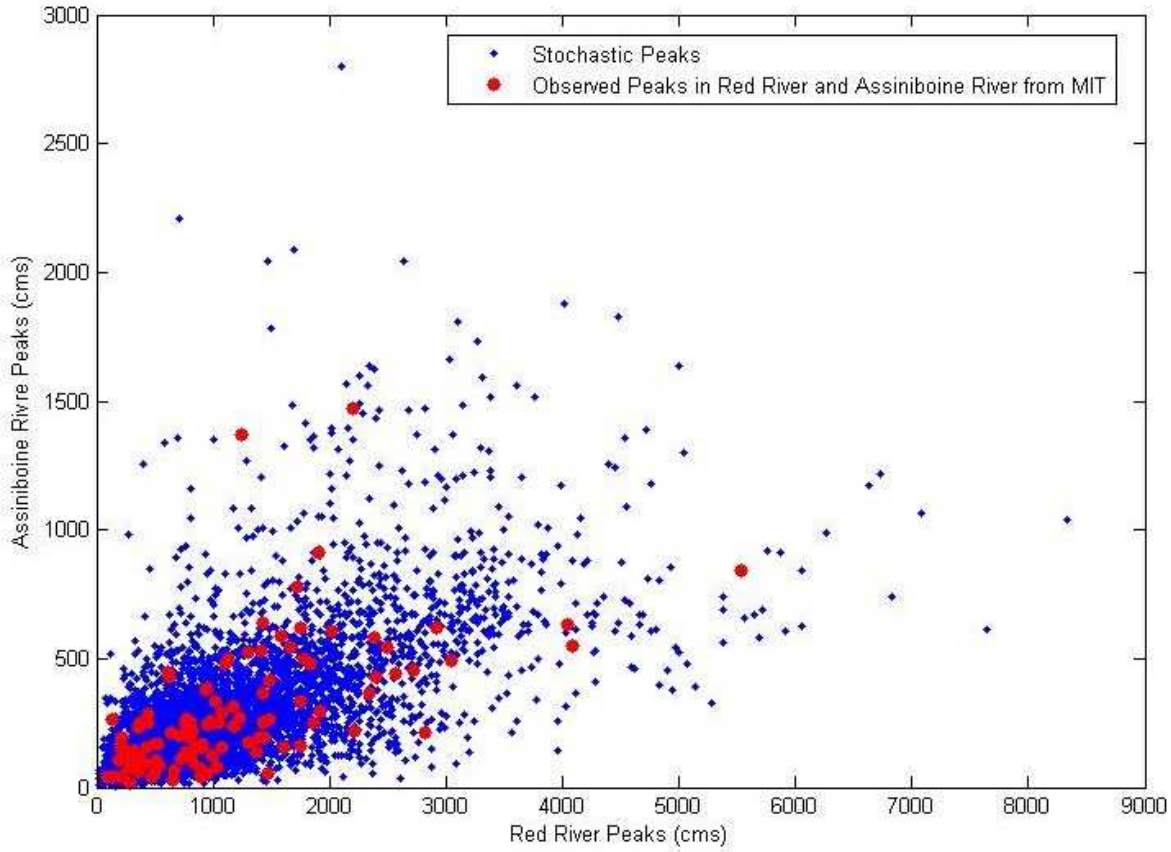
Figure 8.70
Probability Distribution for Red River Flows above the Forks



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 8.71 compares the observed flows of the two rivers with the simulated synthetic flows.

Figure 8.71
Scatter Plot of Stochastic Peak Flows vs.
Observed Peak Flows for the Red and Assiniboine Rivers



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

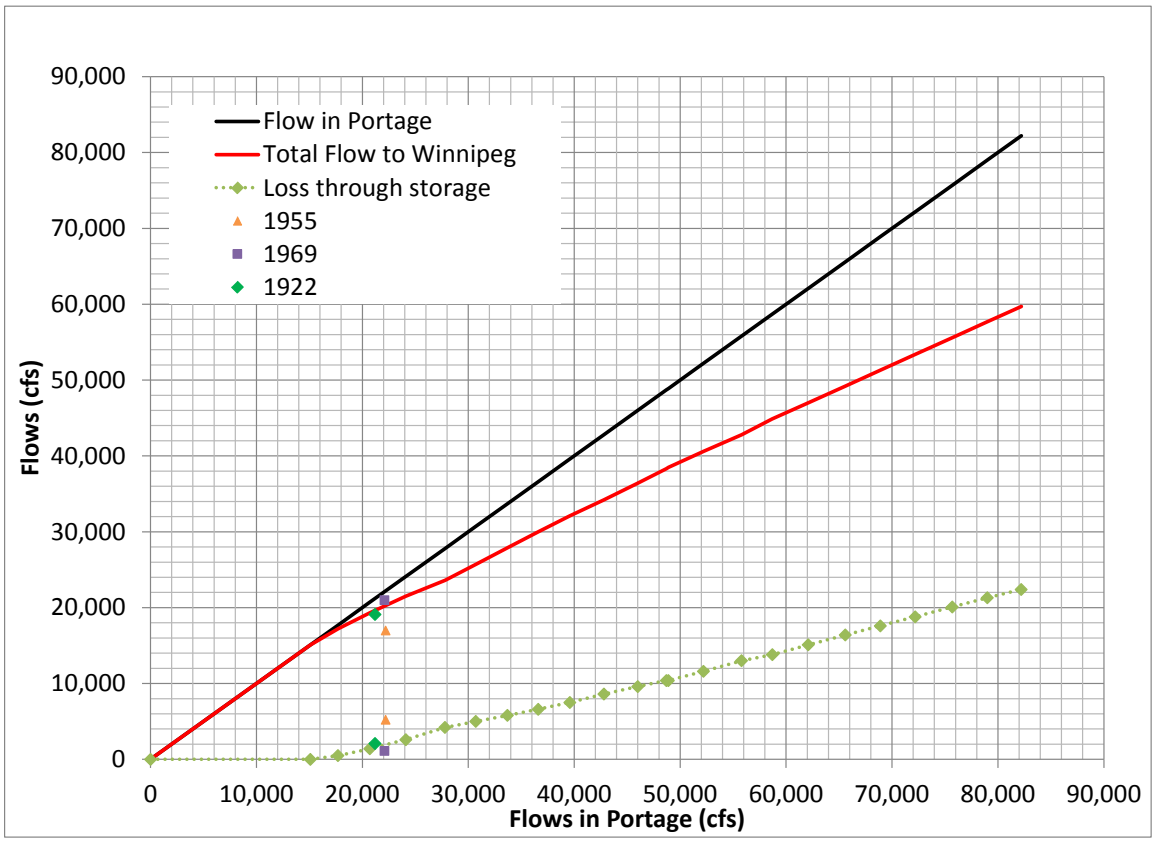
In the modelling for this analysis, simulated Assiniboine River flows at Holland were routed through the Portage Diversion and Lower Assiniboine River using a relationship of peak flows of the Assiniboine River at Holland and the best estimate of the modes of operation of the Portage Diversion. The assumptions that constitute the best estimate of the modes of operation were listed in Section 5.2.4. They were based on knowledge of the current operation guidelines and experience with the operation of the Portage Diversion in the past. The key assumptions are summarized below:

- The Portage Diversion is not operated when flows in the Assiniboine River upstream of Portage la Prairie are less than 283 m^3/s (10,000 cfs).

- A minimum of 75% of flows that exceed 283 m^3/s (10,000 cfs) upstream of Portage la Prairie are diverted into the Portage Diversion.
- The Portage Diversion is operated up to its maximum capacity of 708 m^3/s (25,000 cfs) to maintain flows below 510 m^3/s (18,000 cfs) in the Assiniboine River downstream of Portage la Prairie.
- Flows in the Portage Diversion cannot exceed its design capacity of 708 m^3/s (25,000 cfs).

On the Lower Assiniboine River, when flows exceed the capacity of the Provincial dikes that are located between Portage la Prairie and approximately Baie St. Paul, widespread flooding occurs along the area. Most of the overflows make their way back to Winnipeg, either by returning into the Assiniboine River channel or by traveling south-easterly into the La Salle River system. However, a portion of the flows are lost through storage; and, therefore, the peak flows from the Assiniboine River that enter the City of Winnipeg are attenuated. This situation was incorporated into the model based on a relationship between peak Assiniboine River flows at Portage and overflows lost through storage. The relationship was devised based on the "Average Assiniboine River Contribution to Red River Peaks" prepared by Bill Cook of the Manitoba Water Resources Branch in 1962 (Cook, 1962), and checked with WSC data from the 1922, 1955 and 1969 floods. The relationship is summarized on Figure 8.72.

Figure 8.72
Relationship of Assiniboine River Flows at Portage and Overflows Lost through Storage



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

8.6 FLOOD DAMAGE MODELING

8.6.1 Lower Assiniboine River

Where the Assiniboine River flows out of the Manitoba Escarpment west of Portage la Prairie onto the bed of glacial Lake Agassiz an alluvial fan has developed. Over the past century, dikes have been constructed along the banks of the river; but the river has considerably less capacity from Portage la Prairie to Headingley (Lower Assiniboine River) than it has west of Portage. Based on a survey of the existing dike top elevations and water surface profiles developed for various discharges, the channel should be able to pass approximately 650 m^3/s (23,000 cfs) without overtopping the existing dikes. However, the experience during the 2011 Flood demonstrated that some sections of the dikes were not structurally able to withstand extended

periods of high flow and would likely fail. Substantial damages would occur on the Lower Assiniboine River due to overtopping and failure of the existing dikes.

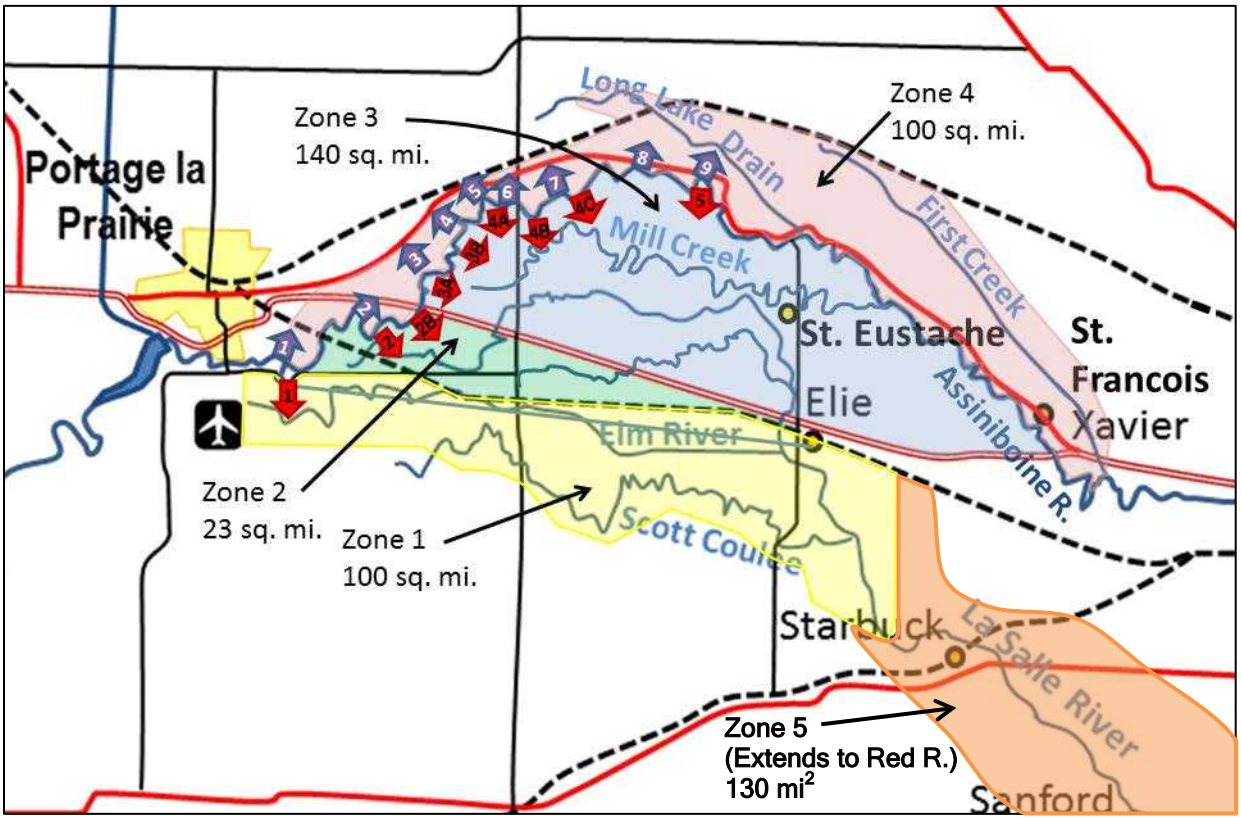
This section explains the method and results of the analysis to determine the frequency damage relationship for the Lower Assiniboine River from failure or overtopping of the existing dikes that extend from Portage la Prairie to approximately St. Francis Xavier. The results of the analysis were used to determine the benefits on the Lower Assiniboine River of the flood mitigation alternatives being investigated for the current study.

8.6.1.1 Dike Breach Scenarios

Improvement schemes for the existing diking system of the Lower Assiniboine River were considered for this study and are documented in Section 10.1. To develop the improvement schemes, it was necessary to assess the current condition and capacity of the existing diking system. Results of the analysis, which included an estimate of the probability of failure of different dike sections for a range of flow conditions, have been documented in Section 10.1. It was determined that a significant risk of failure from piping, seepage or slope failure existed in several locations, with water levels well below the top of the dikes. In fact, the current reliable flow capacity of the dikes has been estimated to be 425 m^3/s (15,000 cfs). At flows of 510 m^3/s (18,000 cfs), the dikes have on average an approximate 20% probability of failure.

To determine what potential damages could arise from a dike breach along the Lower Assiniboine River, it was first necessary to understand the potential hydraulic conditions that would form a potential dike breach. Although detailed surveys and data on the dikes are available, there is still considerable uncertainty of the locations and the failure method that would cause a breach to occur. Therefore, dike breach scenarios were developed for the wide range of conditions that could occur. Based on the results of the geotechnical and hydraulic analyses documented in Section 10.1 and topographic data from the 2011 LiDAR surveys, nine potential breach locations were identified on each side of the river. These breach locations are shown in Figure 8.73.

Figure 8.73
Breach Locations and Areas of Potential Overflows



The following two breach scenarios were analyzed at each of the locations identified in Figure 8.73:

1. Overtopping – The breach width at overtopping locations would equal five times the upstream water depth or head. This ratio was based on what has been widely used for Dam Breach Studies in North America (US FERC, 1993).

2. Structural Failure – Breaches that occurred due to piping, seepage, slope failure, etc. These breaches are more severe than those caused due to overtopping. The breach width is determined from the Froehlich Equation. The Froehlich Equation is a relationship that was developed based on approximately 60 historic dam breaches and accounts for several factors including: breach height and overflow volume, as well as the type of failure (Wahl (USBR), 1998).

The potential width of the breaches varied considerably, depending on the failure method and the location due to the different heads. For example at 708 m^3/s (25,000 cfs), the potential breach width due to overtopping varied between approximately 4 m to 16 m (13 ft to 53 ft), while the width from structural failure varied between 105 m and 210 m (345 ft and 690 ft). Evidently,

the overflow resulting from a structural failure proved to be much larger than that of overtopping. This is further discussed in Section 8.6.1.3.

8.6.1.2 Zones of Overflow

East of Portage la Prairie the Assiniboine is a perched river. When a dike fails the overflow moves away from the river and continues cross country into adjacent waterways. Based on a review of LiDAR data, potential breach locations, as well as existing drainage patterns in the area, five overflow zones have been identified on Figure 8.73:

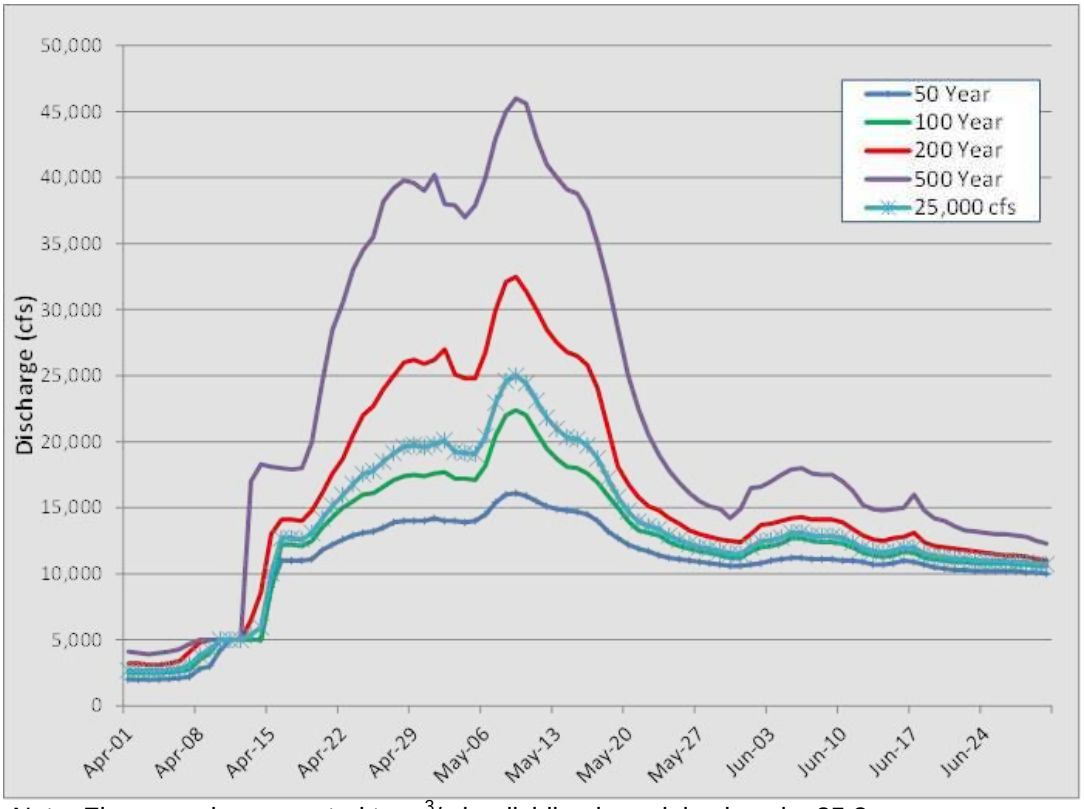
- Zone 1 (25,900 ha or 100 mi^2) overflows into the Elm River and Scott Coulee, eventually flowing into the La Salle River south of Elie.
- Zone 2 (5,960 ha or 23 mi^2) flows into an area bounded by the Trans-Canada Highway on the north and the CNR tracks on the south. These overflows enter the upper La Salle River north-west of Elie.
- Zone 3 (36,260 ha or 140 mi^2) is bounded by the Assiniboine River on the north and the Trans-Canada Highway on the south. Some of this overflow would join the La Salle River north-west of Elie, and some would re-enter the Assiniboine River east of St. Eustache via Mill Creek.
- Zone 4 (25,900 ha or 100 mi^2) is the area north of the Assiniboine River. West of Baie St. Paul the overflows travel north-easterly parallel to the river towards Long Lake Drain. Normally, Long Lake Drain flows into the Assiniboine River near the Baie St. Paul Bridge (PR 248). However, during periods of high flow on the Assiniboine River water levels on Long Lake Drain are backed up, which results in the overflows from the north continuing easterly eventually flowing into Scott's Drain. Similar to Long Lake Drain, the capacity of this drain is also reduced due to backwater from the high Assiniboine River flows, so the overflows would continue south-easterly into First Creek and Second Creek. These two drains join the Assiniboine River just east of Lido Plage.
- Zone 5 (33,670 ha or 130 mi^2) represents the portion of the overflows, which travel towards and into the La Salle River system. Zone 1, 2 and 3 all contribute to the La Salle River along with runoff from the La Salle River watershed. The catchment includes an area north of Starbuck to Dacotah and extends south beyond Sanford and east to the Town of La Salle before connecting to the Red River.

8.6.1.3 Overflow Analysis

Potential breaches were assessed for each of the 18 identified sites for a river flow of 708 m^3/s (25,000 cfs) downstream of the Portage Diversion as discussed below. The 708 m^3/s (25,000 cfs) flow was selected as it was approximately the midpoint between the existing

reliable dike capacity of 425 m^3/s (15,000 cfs) and the 1:500 year flow on the Lower Assiniboine River of 1,303 m^3/s (46,000 cfs). A flood hydrograph for the 708 m^3/s (25,000 cfs) flow condition was determined by interpolating from the design hydrographs for the Lower Assiniboine River as shown in Figure 8.74.

Figure 8.74
Lower Assiniboine River Hydrographs



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3

These hydrographs were developed based on the design hydrograph developed at Holland and documented in Section 5.4, which were routed through the Portage Diversion (PD) and Lower Assiniboine River using a relationship of peak flows of the Assiniboine River at Holland and the best estimate of the modes of operation of the PD. The assumptions that constitute the best estimate of the modes of operation are listed in Section 5.2.4. They were based on knowledge of the current operation guidelines and experience with the operation of the PD in the past. The key assumptions are summarized below:

- The PD starts operation when flow in the Upper Assiniboine River exceeds 283 m^3/s (10,000 cfs).
- A minimum of 75% of flows that exceed 283 m^3/s (10,000 cfs) in the Upper Assiniboine River are diverted into the PD.
- The PD is operated up to its maximum capacity of 708 m^3/s (25,000 cfs) to maintain flows below 510 m^3/s (18,000 cfs) in the Lower Assiniboine River downstream of Portage la Prairie. Flows in the PD are not allowed to exceed this design capacity of 708 m^3/s (25,000 cfs).

In 2011, flows in the PD were allowed to exceed its design capacity. However, this was done on an emergency basis only and, therefore, does not constitute the base condition on the Lower Assiniboine River.

A breach was simulated for the 708 m^3/s (25,000 cfs) discharge at each site; and, subsequently, the elevation difference between the river level and the prairie level was determined. Results of a backwater analysis, documented in Section 8.3 were used to determine the river level at that location for each day based on the simulated river flow. When the computed river level was higher than the prairie level, the breach overflow was computed using the weir formula. Overflows were calculated for both breach scenarios as discussed in Section 8.6.1.1.

When computing the overflows, it was assumed that the breach formed as soon as river levels reached the prairie level. In reality, the failure is not likely to occur until river levels are above prairie level and the pressure causes the dike to fail (structural failure), or until the water exceeds the top of the dikes (failure from overtopping). The result would be that the actual failure would occur a few days later than simulated. Because there was no accepted method for predicting when the dike would fail, the procedure assumed failure on the first day. This assumption had little effect on the results of the analysis for a breach caused by structural failure because, in most cases, the entire zone downstream of the breach would be flooded during the first half of the flood period. If flooding was delayed for a few days, the entire zone would still flood. Additionally, past experience has shown that the formation of ice jams in the Assiniboine River, which generally occur well before the peak of the flood, cause local increases in the water levels and increase the risk of a dike breach along the river. By simulating a breach to occur on the first day, the model inherently accounts for the risks associated with ice in the river.

In the analysis, the local drainage capacity estimated using judgement based on knowledge of the area was subtracted from the daily overflows to compute the daily volume that would go into storage. This overflow volume was accumulated day by day until the end of the flood. It was assumed that the excess flows would flood sections of land between the mile roads until the roads were overtopped. By this procedure, the flooding would proceed across the flood plain until the entire adjacent zone was flooded. Once the entire zone was flooded, the additional flows would enter a receiving stream.

LiDAR data was used where possible to obtain topographic information, as well as profiles of major roadways within the flood zones. However, a rigorous analysis of the progression of the flooding was not possible because the LiDAR data did not extend over the downstream portion of the study area. Additionally, data on the discharge capacities of all culverts and bridges in the flood region could not be compiled and analysed at this conceptual phase of analysis. Therefore, simplified assumptions were used. It was estimated and assumed that each land section would flood to an average depth of 0.9 m (3 ft) for the 708 m^3/s (25,000 cfs) breach scenario. This was based on the observation that mile roads were typically 0.6 to 0.9 m (2 to 3 ft) above the prairie and about 0.3 m (0.5 ft) of additional water level would be required for sufficient volumes to flow over the roadway. Figures 8.75 and 8.76 show the computed overflows for the first breach site on the south dike of the river.

Figure 8.75
Computed Overflows from Site 1

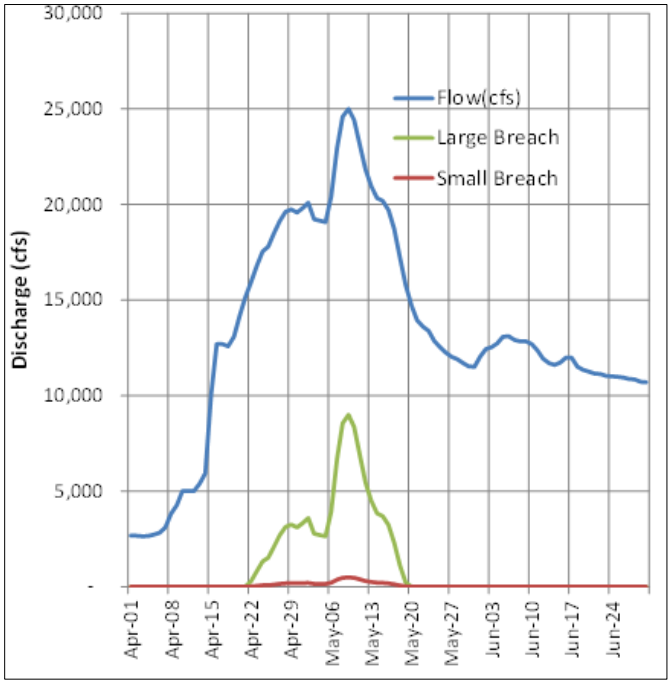
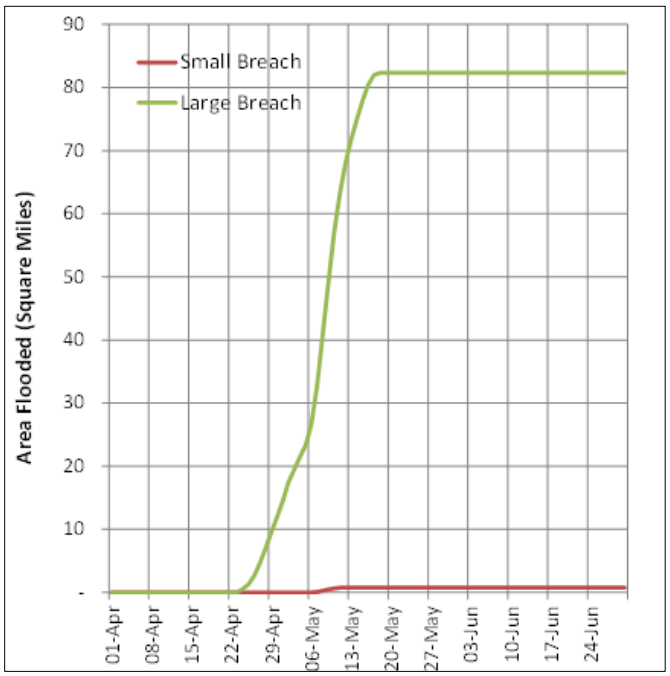


Figure 8.76
Computed Flooding from Site 1



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3

The results shown on Figures 8.75 and 8.76 are located at the Hoop and Holler site. On the figures, "Large Breach" represents the breach scenario due to structural failure of the dikes while "Small Breach" represents the breach scenario due to overtopping. The simulated overflows commenced on April 21 when the river levels began to exceed the adjacent prairie elevation at the location of the breach. The corresponding flow on this day was 425 m^3/s (15,000 cfs). Overflows then continued until May 19 when the river levels returned to an elevation below the adjacent prairie. Under the breach scenario due to overtopping, only 0.7 mi sections (185 ha or 460 ac) would be flooded. This flooding would be similar to the 2011 flooding below Hoop and Holler. However, under the breach scenario due to structural failure, the overflow would flood 82 mi^2 (21,200 ha or 52,500 ac). This would inundate most of Zone 1. These flows would eventually continue down the La Salle River.

A similar analysis was carried out for each of the 18 breach sites. The results are summarized in Table 8.8. On the table, the maximum overflow at each site is shown. This value was determined based on the hydraulic conditions of each individual breach. Since the Assiniboine River can convey 283 m^3/s (10,000 cfs) before the water level exceeds the adjacent prairie elevation, the maximum overflows for the 708 m^3/s (25,000 cfs) scenario did not exceed 425 m^3/s (15,000 cfs).

Table 8.8
Flooded Area for 708 m^3/s (25,000 cfs) Breach Scenarios

Breach Location	Adjacent Zone	Overtopping Scenario			Structural Failure Scenario		
		Max Overflow (cfs)	Volume of Flood (ac ft)	Area Flooded (mi^2)	Max Overflow (cfs)	Volume of Flood (ac ft)	Area Flooded (mi^2)
1	Zone 1	500	1,400	0.7	9,000	158,000	82
1N	Zone 4	900	23,600	12.3	13,000	375,500	100
2N	Zone 4	200	1,200	0.6	4,100	52,500	27.4
2	Zone 2	200	1.3	0.7	5,400	76,600	23
2B	Zone 2	600	12,900	6.7	11,000	287,000	23
3A	Zone 3	200	1,200	0.6	4,700	58,700	30.6
3N	Zone 4	1000	24,900	13	12,000	375,000	100
3B	Zone 3	1100	31,600	16.4	13,000	418,000	140
4N	Zone 4	3900	215,000	100	15,000	913,000	100

Breach Location	Adjacent Zone	Overtopping Scenario			Structural Failure Scenario		
		Max Overflow (cfs)	Volume of Flood (ac ft)	Area Flooded (mi^2)	Max Overflow (cfs)	Volume of Flood (ac ft)	Area Flooded (mi^2)
4A	Zone 3	2900	148,000	77	15,000	796,000	140
5N	Zone 4	5600	367,000	100	15,000	1,034,000	100
6N	Zone 4	2400	79,100	41.2	15,000	676,000	100
4B	Zone 3	4200	186,000	97	15,000	861,000	140
7N	Zone 4	2600	89,700	46.7	15,000	689,000	100
4C	Zone 3	4900	245,000	128	15,000	907,000	140
8N	Zone 4	2100	59,100	30.8	15,000	591,000	100
5	Zone 3	2900	120,000	62.3	15,000	120,000	62
9N	Zone 4	2700	101,000	52.5	15,000	693,000	100

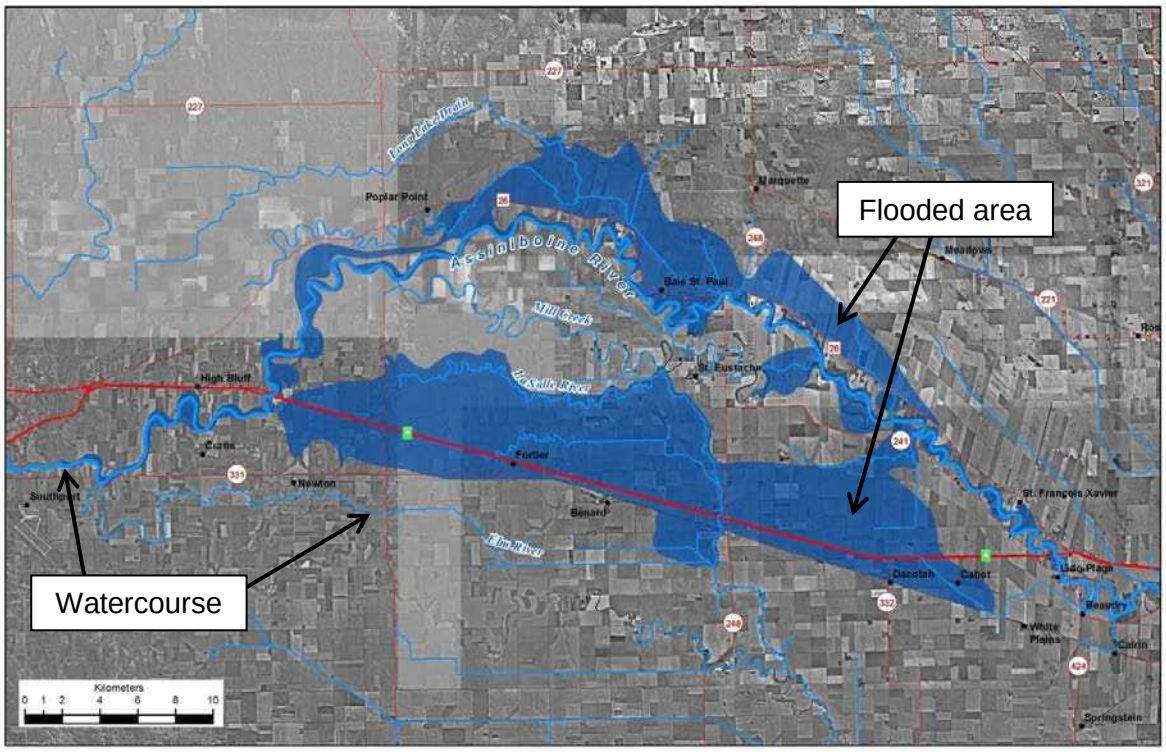
Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

Results presented in Table 8.8 are based on an analysis of each breach location individually. In reality, when a breach occurs, the flows in the river downstream from the breach are reduced. For example, at the first north dike breach point (1N), the peak breach flow for the structural failure scenario is 368 m^3/s (13,000 cfs). With 708 m^3/s (25,000 cfs) in the river and a breach discharge of 368 m^3/s 13,000 cfs, there would only be 340 m^3/s (12,000 cfs) in the lower reaches of the river. This would result in some reduction in flood flows and reduction in risks of flooding further downstream. For example, high levels on the Assiniboine River restrict flows along the Long Lake Drain resulting in considerable flooding north of the river as was observed in 2011. With an upstream breach, the river flows would be lower and Long Lake Drain would be able to convey floodwaters into the Assiniboine River. Furthermore, potential overflows from other breaches located downstream would be less due to the lower river flows. Analysing each breach individually, therefore, provided a conservative estimate of the potential overflows that could occur. The results presented in Table 8.8 formed the basis of the damage estimates discussed in Section 8.6.1.4 of this document.

Comparison to 1955 Flood

The 1955 flood was used to compare the model results to a historic event. In 1955 the Assiniboine River dikes experienced several breaches, which caused substantial flooding. The estimate of flooded area for this event is shown in Figure 8.77 (Manitoba, 2013a).

Figure 8.77
Map of 1955 Assiniboine River Flood Area
(Manitoba, 2013a)



According to the 1955 flood report (Morris, 1955), an area of approximately 51,800 ha (200 mi^2) was flooded during this event, which had a recorded peak flow on the Assiniboine River at Portage la Prairie of approximately 637 m^3/s (22,500 cfs). Based on the 1955 flooded area map and report, the location of the breach on the South Dike was identified to be in the vicinity of Site 2B in Figure 8.73 while the breach on the North Dike was near Site 5. For the current study, assuming a breach occurring at both of these sites and using the 1955 Assiniboine River hydrograph recorded by Water Survey of Canada near Portage la Prairie, the simulated flooded area for the breach overflow analysis was about 45,300 ha (175 mi^2). The area produced from the analysis is considered to be representative of the magnitude of the actual 1955 event

considering the uncertainties with the exact size, location and timing of the breaches. It was, therefore, concluded that the method used to estimate the potential extent of overflows due to a dike breach would generate results that were representative of the actual overflow conditions that could occur along the Lower Assiniboine River. As such, the flooded area was used to estimate the damages that would occur in the event of a breach of the Lower Assiniboine River dikes.

North Dike Overflows – Zone 4

All of the north dike overflows would flow parallel to the river into Zone 4. Under the overtopping scenario, only breach sites 4 and 5 would flood the full zone. The excess overflows would initially increase the flooding depth, but eventually the flows would re-enter the Assiniboine River west of Headingley via First Creek (shown on Figure 8.73) and Second Creek (located slightly further downstream).

Under the structural failure scenario, most of the north dike breaches would completely flood Zone 4. Based on the results of the analysis, it was estimated that there would still be more than 283 m^3/s (10,000 cfs) spilling into Zone 4 from these breaches after the area was completely flooded. In fact, it was determined that in most cases the flood zone was fully inundated before the peak flow on the river occurred. This was also the case for two of the overtopping scenarios. This "excess" overflow would temporarily add to local flooding, but much of it would flow back into the Assiniboine River further downstream. This would have the effect of returning the Assiniboine River flows back to the 566 m^3/s (20,000 cfs) range.

Based on the results of the analysis, a flooded area of 25,900 ha (100 mi^2) at a flow of 708 m^3/s (25,000 cfs) was used to represent Zone 4 in the damage model discussed in Section 8.6.1.4, for both the structural failure and overtopping scenarios.

South Dike Overflows – Zones 1, 2 and 3

The only breach location that would flood into Zone 1 was breach site 1. Overflows from this breach were described in the example provided in Section 8.6.1.3 (Hoop and Holler location).

Overflows from a breach at sites 2 and 2B would flow into Zone 2. These overflows would likely remain north of the CNR railway, and would flow into Zone 3 via the La Salle and West La Salle Rivers.

Overflows from breaches at sites 3A to 5 would flow into Zone 3 into the La Salle River and Mill Creek. This zone is quite flat, but prairie level at Mill Creek's confluence with the Assiniboine River is at about 242 m (794.0 ft). The water surface profiles show that the Assiniboine can convey about 625 m^3/s (22,000 cfs) at this level. However, with an upstream breach, the Assiniboine River flows could be as low as 283 m^3/s (10,000 cfs). Therefore, the capacity of Mill Creek should not be restricted by Assiniboine River levels. Once the flooding has worked its way down Mill Creek, it would flow back into the Assiniboine River, resulting in increased flows downstream through St. Francois Xavier, Headingley, and Winnipeg.

In addition, there was a Flood Damage Reduction Study carried out for the Village of Elie, Manitoba by Manitoba Water Resources in 1984 (Manitoba, 1984a). Based on this study, the 100-year flood level on the La Salle River just north of the Trans-Canada Highway is 240.8 m (790.0 ft). The corresponding flow through the highway is 53.8 m^3/s (1,900 cfs). It is unlikely that the culverts through the highway and the railway bridge could convey much more than the 100-year flood level on the La Salle River, since they were designed to convey the local flow in the La Salle River rather than larger overflows from the Assiniboine River. This flow restriction would have the effect of limiting the flow to the La Salle River from any Assiniboine River overflows. Most of the flood flows would be directed east and re-enter the Assiniboine River via Mill Creek or the Barickman Coulee (located about 5 km (3.1 mi) south-east of Mill Creek) as was observed in the 1955 Flood and further discussed in the section on Overflows to La Salle River – Zone 5 below.

Based on the results of the analysis at a flow of 708 m^3/s (25,000 cfs), flooded areas of 180 ha (0.7 mi^2) for the overtopping scenario and of 21,240 ha (82 mi^2) for the structural failure scenario were used to represent Zone 1 in the damage model discussed in Section 8.6.1.4. In Zone 2, a flooded area of 1,920 ha (7.4 mi^2) was used for the overtopping scenario and 5,960 ha (23 mi^2) for the structural failure scenario. In Zone 3, a flooded area of 36,260 ha (140 mi^2) was used for both scenarios.

Overflows to La Salle River – Zone 5

Flooding in Zone 5 was assumed to be the result of a breach of the south dike early during the flood. Excess overflows from Zones 1, 2, and 3 would contribute to the discharge carried by the Elm River and Scott Coulee prior to entering the La Salle River.

Experience from the 1955 flood shows that even though a breach on the south dikes allowed as much as approximately 170 m^3/s (6,000 cfs) to escape from the Assiniboine River, peak flows in the La Salle River due to overflows of the Assiniboine River only reached approximately 62 m^3/s (2,200 cfs) in Starbuck (Morris, 1955). As discussed previously, this is likely attributed to the CNR rail line and the Trans-Canada Highway, which impeded overflows from entering Zone 1 except at the La Salle River, where the existing capacity of the culverts, bridge and crossing at Elie were relatively low.

The majority of overflows to the La Salle River were, therefore, due to a breach of the south dikes at a location near Site 1. Under the overtopping scenario, flows through Site 1 would have negligible effects to the flood peaks on the La Salle River; and the flooded area in Zone 5 for this scenario would be 0. However, under the structural failure scenario, up to 255 m^3/s (9,000 cfs) would flow into the Elm River channel from a breach of the dikes. The peak breach flow of 255 m^3/s (9,000 cfs) would be attenuated somewhat by the flood storage on the La Salle River, but flows estimated in the order of 113 m^3/s to 142 m^3/s (4,000 cfs to 5,000 cfs) could be added to the La Salle River.

Local spring runoff in the La Salle River for the 1:100 year flows listed in the 1984 La Salle River Backwater Study (Manitoba, 1984b) is:

- Elie 55 m^3/s (1950 cfs).
- Starbuck 159 m^3/s (5600 cfs).
- Sanford 168 m^3/s (5900 cfs).
- La Salle 224 m^3/s (7900 cfs).

Based on this information, a breach due to structural failure near Site 1 would likely result in flows in excess of the 100-year flood at Elie, Starbuck and Sanford, resulting in considerable

risk that flood damages would occur in these towns. Also the La Salle Backwater Study (Manitoba, 1984b) indicated that agricultural flooding started at about 30 m^3/s to 40 m^3/s (1000 cfs to 1500 cfs) between Starbuck and Elie. Therefore, the overflows would result in some agricultural flooding along the La Salle River within this reach of the river. Assuming that the La Salle River was already at bank full capacity and also that half the volume of water that contributed to flooding in Zone 1 would flood Zone 5, it was estimated that overflows into Zone 5 would flood an area of approximately 10,620 ha (41 mi^2).

Overflows for Larger Floods

For the current study terms of reference, a flow of 1,303 m^3/s (46,000 cfs) on the Lower Assiniboine River was selected as it represented the 500-year event, assuming that the Portage Diversion was diverting 708 m^3/s (25,000 cfs).

At a flow of 1,303 m^3/s (46,000 cfs) on the Lower Assiniboine River, most of the existing diking system would be overtopped and significant breaches due to the structural failure of the dikes would occur. Both structural and overtopping failure scenarios were assumed to yield the same extent of flooding.

Since most of Zones 1 to 4 were shown to be flooded at a flow of 708 m^3/s (25,000 cfs), it was, therefore, assumed that the entire area of Zones 1 to 4 would be flooded at a flow of 1,303 m^3/s (46,000 cfs).

For Zone 5, detailed hydraulic analyses that were beyond the scope of the current study would be necessary to determine the flooded extent along the La Salle River. Instead, a “low” and “high” estimate was determined for this study to quantify the approximate minimum and maximum limits of the potential area of overflow. For the “low” estimate, it was assumed that the flooded area would be at a minimum equal to that of the 708 m^3/s (25,000 cfs) scenario (10,620 ha or 41 mi^2). For the “high” estimate, it was assumed that the entire zone would be flooded (33,670 ha or 130 mi^2).

Specific details of other assumptions that formed the “low” and “high” damage estimates at the flow of 1,303 m^3/s (46,000 cfs) have been described in Section 8.6.1.4 of this document.

8.6.1.4 Estimation of Flood Damages

A list of flood damages considered for this study is outlined in Section 7.0, which outlines the basis for economic evaluations of options for flood mitigation. Using the results of the dike breach analysis discussed in Section 8.6.1.1, an integrated Excel and GIS based model was developed to estimate the potential damages due to flooding of the Lower Assiniboine River. Flood damages were grouped into the following six categories in the flood damage model:

1. Agricultural Damages.
2. Damages to Buildings and Structures.
3. Damages to Public Infrastructure and Utilities.
4. Flood Response and Fighting Costs.
5. Disruption to Transportation.
6. Business Losses, Stress and Anxiety and Other Damages.

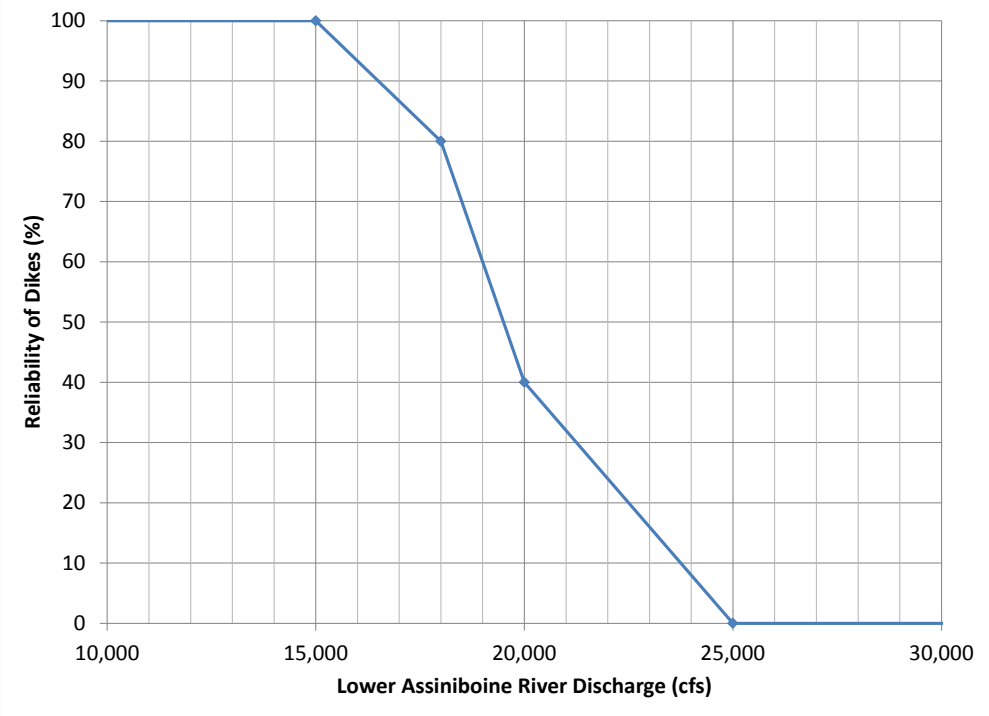
Flood damages were computed for each of the five zones described in Section 8.6.1.2 for Assiniboine River flows of 709 m^3/s and 1,303 m^3/s (25,000 cfs and 46,000 cfs). At the flow of 709 m^3/s (25,000 cfs), damages were computed for structural failure and overtopping breach scenarios. At the flow of 1,303 m^3/s (46,000 cfs), a "low" and "high" damage estimate was computed to represent the approximate range in uncertainty of the estimate. The "low" value corresponded to a reasonable estimate of the minimum level of damages that could be expected while the "high" value corresponded to a conservative estimate of the maximum level of damages that could be expected.

As outlined in Section 10.1.5, the base condition for this study assumes that the existing dikes along the Assiniboine River are reliable at a flow of 425 m^3/s (15,000 cfs) or less. However, without dikes or with dikes that are completely unreliable, it was assumed that flood damages would start to occur at a flow of approximately 283 m^3/s (10,000 cfs). This flow is equivalent to the point where the Assiniboine River water surface elevation approximately equals the natural bank full conditions (Hatch, 2011a). Damages at flows below 708 m^3/s (25,000 cfs) were determined by interpolating from damages computed at and 708 m^3/s (25,000 cfs) and zero damages at 283 m^3/s (10,000 cfs).

For the current dike system in place, and assuming the dike system is 100% reliable, the point of incipient flooding where overtopping of the dikes would start to occur was assumed to be 651 m^3/s (23,000 cfs) and is based on a survey of the existing dike top elevations and water surface profiles developed for various discharges of the channel.

The adopted damage curve for this study was assumed to fall between the "low" and "high" computed damage values, as well as between the damage values computed for the structural failure scenario (unreliable dikes) and the overtopping scenario (reliable dikes). As mentioned above, the base condition for this study assumed that the existing dikes along the Assiniboine River were reliable at a flow of 425 m^3/s (15,000 cfs) or less. Therefore, the final discharge-damage curve adopted for this study had a point of incipient flooding of 425 m^3/s (15,000 cfs). Furthermore, based on the results of the geotechnical analysis outlined in Section 10.1.5, it was assumed that at 509 m^3/s (18,000 cfs), the existing diking system was on average 80% reliable, and at 566 m^3/s (20,000 cfs), it was on average 40% reliable. At 708 m^3/s (25,000 cfs), it was assumed that the existing diking system was no longer reliable and that a structural failure of the dikes would occur. A plot showing the curve that represents the current reliability of the dikes is provided on Figure 8.78.

Figure 8.78
Reliability of Lower Assiniboine Dikes as a Function of Discharge



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

The sections that follow describe how the damages in the model were computed.

8.6.1.5 Data on 2011 Flood Damages and Mitigation

Data on 2011 Flood damages and mitigation was used to develop a discharge-damage relationship along the Lower Assiniboine River, as well as to compare the 2011 Flood to the results of the damage model.

The 2011 Flood damage data and post-flood mitigation initiatives were obtained from various provincial organizations for the entire Province of Manitoba, including the Emergency Measures Organization (EMO), Manitoba Agricultural Services Corporation (MASC) and Manitoba Agriculture, Food and Rural Development (MAFRD). Each organization was responsible for separate types of programs, which provided financial assistance or compensation for damages related to the 2011 Flood, or for post-flood mitigation initiatives. A summary of the data obtained for the study is provided in Appendix E.

Where possible, a GIS analysis was conducted to isolate damages due to flooding on the Lower Assiniboine River from the Province-wide totals and to subdivide the data into the following five categories:

1. Agricultural Damages.
2. Damages to Buildings and Structures.
3. Damages to Public Infrastructure and Utilities.
4. Flood Response and Fighting Costs.
5. Business Losses.

Since transportation data was not available in 2011, a sub-category for this type of data was not included in the analysis of the 2011 Flood damages.

In instances where insufficient details and spatial breakdowns were provided with the data, KGS Group applied their best judgment to isolate Lower Assiniboine damages and to assign the data to one of the five categories. This was done based on results from the GIS analysis for the remaining data, a review of the program descriptions and discussions with the different program administrators.

Several programs such as Disaster Flood Assistance (DFA) included a 20% deductible prior to providing compensation to the different stakeholders. Where this was the case, the 2011 compensation values were increased by 25% to best represent the total damages that actually occurred.

Due to the flood mitigation measures implemented by the Manitoba Government in 2011, the majority of damages that occurred in 2011 were flood response and fighting costs. For example, Operation LUSTRE (Flood Mitigation Manitoba – Assiniboine Dikes and Portage Diversion) carried out by the Canadian Military cost approximately $6.6 M. Only an area of approximately 1 to 2 sq mi was flooded due to the controlled release of flows by the Hoop and Holler bend. Damages due to the controlled release of flows at the Hoop and Holler bend were available through the Hoop and Holler Compensation Program, part of the Flood 2011 – Building and Recovery Action Plan. Approximately $6 M was compensated as part of this program.

Table 8.9 summarizes KGS Group's best estimate of the total Lower Assiniboine River 2011 damages for the five categories described previously and accounted for in the data that was made available for this study. It is recognized that the data used to estimate the 2011 damages may not account for all damages that occurred during the flood due to, for example, an undervaluation by the insurer or by the compensation programs, or because the damaged items are non-insurable or cannot be quantified. Special allowances have been incorporated into the damage model to account for this and have been further discussed in the sections that follow.

Table 8.9
Estimated 2011 Flood Damages for Lower Assiniboine River

Agricultural Damages	$3,700,000
Damages to Buildings and Structures	$0
Flood Response and Fighting Costs	$13,900,000
Damages to Public Infrastructure and Utilities	$4,400,000
Business Losses	$400,000
Total	$22,400,000

The peak flow during the 2011 Flood recorded by Water Survey of Canada on the Assiniboine River downstream of Portage was 528 m^3/s (18,650 cfs). This peak flow was substantially less than what would have occurred without the emergency increase in capacity of the Portage Diversion to approximately 963 m^3/s (34,000 cfs) that was undertaken during the flood. It was estimated that without the emergency increase in flow through the Portage Diversion, a peak flow of 796 m^3/s (28,100 cfs) would have occurred on the Assiniboine River. This flow was based on the 2011 peak on the Assiniboine River of 1,504 m^3/s (53,100 cfs) minus a maximum Portage Diversion capacity of 708 m^3/s (25,000 cfs).

Damages that would have occurred due to flooding in the Lower Assiniboine River without the flood mitigation measures implemented in 2011 were estimated by the Manitoba Government and reported in the October 2013 summary report titled, "2011 Flood: Technical Review of Lake Manitoba, Lake St. Martin and Assiniboine River Water Levels" (Manitoba, 2013b). The report states that:

> *"the emergency flood mitigation measures implemented by the Manitoba Government in 2011 prevented an estimated $300 million in damage on the Lower Assiniboine River".*

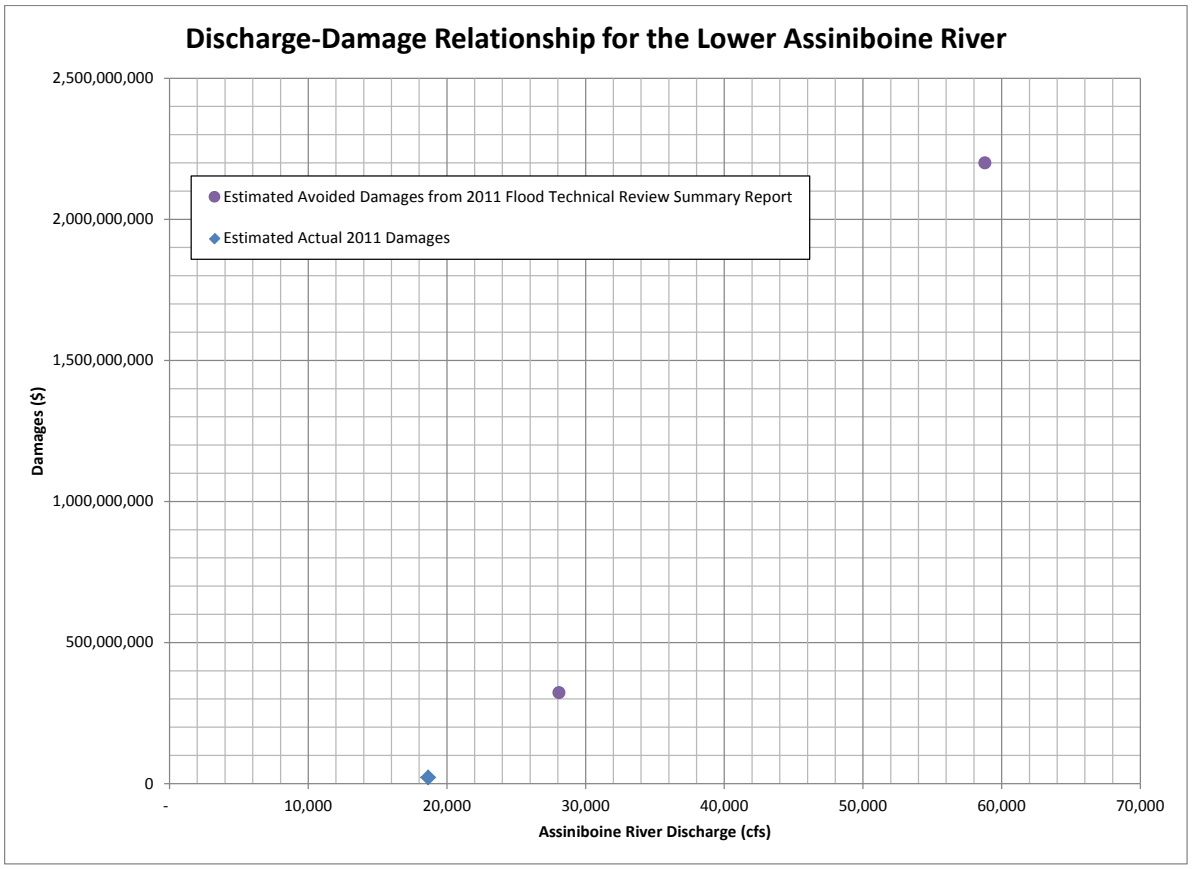
Furthermore the report also states that:

> *"Under unregulated conditions, without the benefit provided by the Portage Diversion and other flood protection infrastructure, the potential damage on the Lower Assiniboine River has been estimated to be $2.2 billion".*

Under unregulated conditions, it was estimated that a flow of 1,665 m^3/s (58,800 cfs) would have occurred on the Assiniboine River downstream of Portage la Prairie. This flow was based on a peak flow of 1,820 m^3/s (64,200 cfs) at Portage la Prairie under unregulated conditions and assumed a natural overflow of 153 m^3/s (5,400 cfs) towards Lake Manitoba, as reported in Manitoba, 2013a.

The estimated damage values of $322 million (the reported avoided $300 million plus the estimated 2011 actual damages of $22 million) at a flow of 796 m^3/s (28,100 cfs), and of $2.2 billion at a flow of 1,665 m^3/s (58,800 cfs) were compared with the results of the damage model developed for this study. These two points, as well as the estimated actual 2011 damages for the Lower Assiniboine River, are shown on Figure 8.79.

Figure 8.79
Estimated Damages for the Lower Assiniboine River in 2011



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

8.6.1.6 Agriculture Damages

Agricultural damages along the Lower Assiniboine River were calculated on the basis of the flooded area and the expected yield of the crops within the flooded land. Based on a review of land use maps of the Lower Assiniboine River and La Salle River basins developed from the LandSAT7 land use classification from the Manitoba Land Initiative (MLI) capture between 2000 and 2002, it was determined that almost the entire area at risk of flooding was classified as agricultural land. A conservative 100% of the total flooded area (the entire area) was, therefore, assumed to be agricultural land in the damage model. Based on the land use maps, 5% of the agricultural land was assumed to consist of grassland or land cultivated for forage crops. Another 5% was assumed to consist of high value vegetable crops. The remaining 90% area was classified as annual crops.

Crop losses were based on average unit rates for the Lower Assiniboine River and La Salle River basins. A unit rate of $800 per hectare was applied to the value of the agricultural land seeded for annual crops based on data obtained from MASC. By using this rate, it was assumed that the losses occurred after the field had been prepared and seeded. In many instances, depending on the forecast and conditions before the flood, it was possible that some farmers might not have prepared and seeded their fields. In this case, the losses would be less. However, to ensure a conservative representation of damages, this scenario was not considered at this conceptual level. For forage crops and grassland, a unit rate of $300 per hectare was applied based on data obtained from the 2010 Shoal Lake Watershed Study (KGS Group, 2010) escalated to 2014$.

For high-value vegetable crops, an average value of $10,000 per hectare was applied based on data obtained from local stakeholders, the Keystone Potato Producers Association, as well as from the Hoop and Holler compensation program. This value was considered representative of the average unit rate for high value vegetable crops such as, but not limited to, potatoes, onions, shallots, carrots, parsnips, beets, etc. High-value vegetable crops are grown within the Lower Assiniboine River and La Salle River basins due to the high quality of soils in the area, the high number of frost free days and higher summer temperatures, as well as the availability of water from the Assiniboine and Elm Rivers. Together, these factors combine for excellent conditions for the production of vegetables, which are sold at much higher rates than regular annual crops. Similar to annual crops, by using the rate of $10,000 per hectare, it was assumed that the losses occurred after the field had been prepared and seeded.

For annual and forage crops under a breach scenario where the flow in the river was 708 m^3/s (25,000 cfs) or less, it was assumed that a producer would still be able to manage and collect approximately 50% of the expected yield from an average season. At a flow 1,303 m^3/s (46,000 cfs), a 50% yield value was assumed for the “low” damage estimate; however, for the “high” damage estimate, it was assumed that 100% of the crops would be lost. Since any flooding, small or large, would have significant impact on high value vegetable crops, 0% yield was assumed for all sizes of floods on this type of land.

To account for other types of agricultural damages such as damages to agricultural infrastructure, fences, crop inventory or cost for debris cleanup, a 50% increase was applied to

the total agriculture damages described above. This increase was selected to be consistent with the method used to define agricultural damages around Lake Manitoba and discussed in Section 8.6.2. In the Lake Manitoba analysis, the 50% increase was determined based on data from the 2011 Flood.

Based on the flooded areas determined from the overflow analysis discussed in Section 8.6.1.1, Table 8.10 shows the agricultural damages computed for the overtopping and structural failure scenarios at a flow of 708 m^3/s (25,000 cfs). Corresponding damages, computed at a flow of 1,303 m^3/s (46,000 cfs), are shown on Table 8.11.

Table 8.10
Agricultural Damages for 708 m^3/s (25,000 cfs)

Flood Zone	Overtopping Scenario		Structural Failure Scenario	
	Flooded Area (ha)	Cost of Flood Damages	Flooded Area (ha)	Cost of Flood Damages
1	180	$ 234,000	21,240	$ 27,639,000
2	1,920	$ 2,498,000	5,960	$ 7,755,000
3	36,260	$ 47,183,000	36,260	$ 47,183,000
4	25,900	$ 33,702,000	25,900	$ 33,702,000
5	0	0	10,620	$ 13,819,000
Total	64,260	$ 83,618,000	99,980	$ 130,099,000

Note: Areas can be converted to acres by multiplying metric values by 2.47.

Table 8.11
Agricultural Damages for 1,303 m^3/s (46,000 cfs)

Flood Zone	"Low" Estimate		"High" Estimate	
	Flooded Area (ha)	Cost of Flood Damages	Flooded Area (ha)	Cost of Flood Damages
1	25,900	$ 33,702,000	25,900	$ 47,980,000
2	5,960	$ 7,752,000	5,960	$ 11,035,000
3	36,260	$ 47,183,000	36,260	$ 67,171,000
4	25,900	$ 33,702,000	25,900	$ 47,980,000
5	10,620	$ 13,819,000	33,670	$ 62,374,000
Total	104,637	$ 136,158,000	127,867	$ 236,539,000

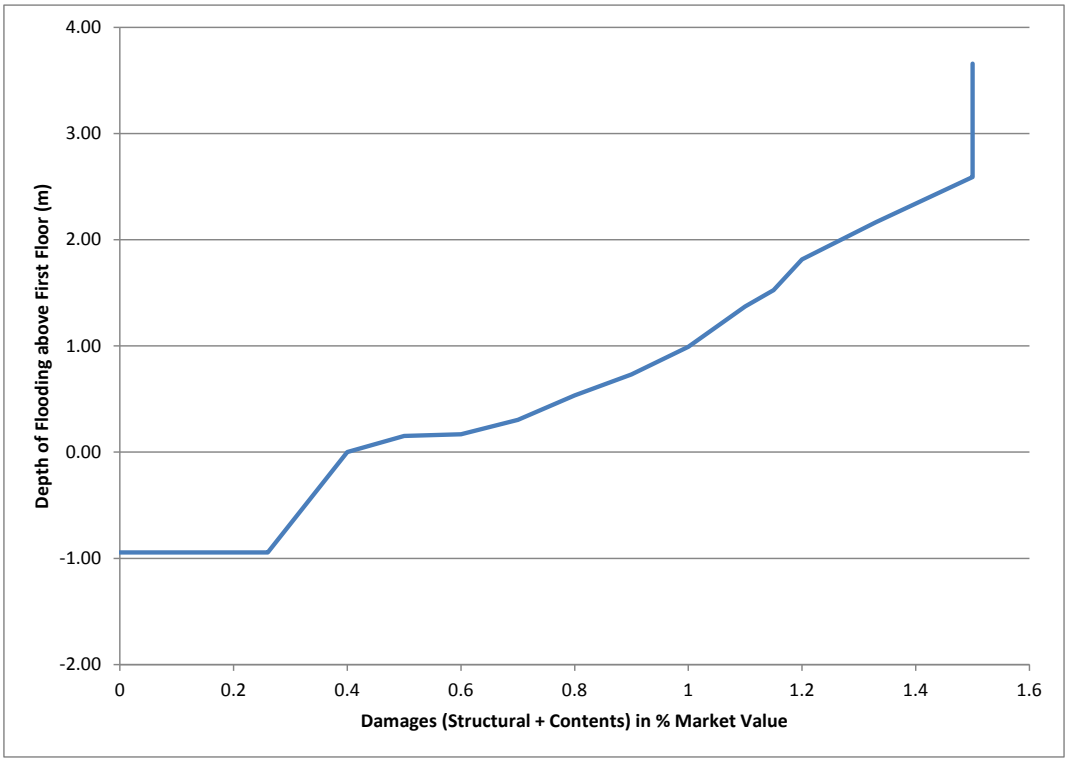
Note: Areas can be converted to acres by multiplying metric values by 2.47.

As discussed in Section 8.6.1.4, it was assumed that without dikes or dikes that were 100% unreliable, flood damages would start to occur at a flow of approximately 283 m^3/s (10,000 cfs). Therefore, damages at flows below 708 m^3/s (25,000 cfs) were determined by interpolating from the damages computed at 708 m^3/s (25,000 cfs) and zero damages at 283 m^3/s (10,000 cfs). However, since the base condition for this study assumed that the existing dikes along the Assiniboine River were reliable at a flow of 425 m^3/s (15,000 cfs), the point of incipient flooding in the final damage-discharge relationship was set to 425 m^3/s (15,000 cfs). This is further discussed in Section 8.6.1.12.

8.6.1.7 Damages to Buildings and Structures

Damages to buildings and structures were based on a depth-damage relationship for the Assiniboine River (Figure 8.80). The adopted relationship for this study was based on a similar relationship developed by KGS Group for the International Joint Commission in 1999 in the studies of flood protection for Winnipeg (KGS Group, 2000b).

Figure 8.80
Depth – Damage Curve for Structures



Since the data available for the current study did not differentiate between the types of structures affected (i.e., house vs garage vs barn, etc.), and the elevation of the main floor compared to the adjacent ground was not available, it was assumed that the main floor elevation of each structure was 0.75 m (2.5 ft) above existing ground.

As discussed in Section 8.6.1.3, the breach analysis at 708 m^3/s (25,000 cfs) assumed an average flood depth of 0.9 m (3 ft). This is equivalent to a 0.15 m (0.5 ft) depth of flooding above the main floor elevation of each structure. From Figure 8.80, the corresponding damages to structures and buildings were equal to 50% of market value. To ensure a conservatively high estimate in the damage model, a rate 70% of market value was applied in the model. For a flow of 1,303 m^3/s (46,000 cfs), it was assumed that the flood depth would be 0.3 m (1ft) more than at 708 m^3/s (25,000 cfs). From Figure 8.80, this corresponds to a market value of approximately 80%. To ensure a conservative result, it was assumed that all structures lost 100% market value.

Damages for structures and buildings within larger communities such as Elie, Sandford and St. Eustache were separated from those in the rural area of the basin. From air photos and MLI data, it was determined that the number of structures that existed within the rural areas were approximately one structure per 100 hectares. This ratio was applied to the flooded area in the model to determine the number of structures in rural areas that were flooded for a given scenario. The estimated number of structures for a flow of 708 m^3/s (25,000 cfs) was approximately 650 for the overtopping scenario and 900 for the structural failure scenario. For a flow of 1,303 m^3/s (46,000 cfs), the estimated number of structures was approximately 1050 for the “low” estimate, and 1280 for the “high” estimate. An average market value of $220,000 per structure was adopted for this study based on a review of property assessment reports from the Government of Manitoba – Property Assessment website (Manitoba, 2014a) escalated to 2014$.

For the larger communities, damages were calculated based on the total actual market value of all the individual structures in the community determined from property assessment reports available from the Government of Manitoba – Property Assessment website (Manitoba, 2014a) escalated to 2014$, instead of using an average assessed value per structure. Table 8.12

summarizes the total assessed value of buildings and structures used in the model for the larger communities.

Table 8.12
Summary of Assessed Value of Buildings and Structures for Larger Communities

Name of Community	Total Assessed Value (2014)	Location of Community
Newton	$ 8,656,000	Zone 1
Oakville	$ 37,329,000	Zones 1 & 2
Elie	$ 71,412,000	Zones 1, 2 & 3
St. Eustache	$ 29,755,000	Zone 3
Poplar Point	$ 10,820,000	Zone 4
Baie St. Paul	$ 25,427,000	Zone 4
Starbuck	$ 34,624,000	Zone 5
Sandford	$ 77,524,000	Zone 5
La Salle	$ 96,124,000	Zone 5

Potential damages to the sewer and wastewater treatment system for these communities was also considered in the analysis. Based on information received from the Manitoba Water Services Board (MWSB, 2014), it was determined that five waste water treatment lagoons and three reservoir pumping stations could be at risk of flooding within the communities. Based on KGS Group's past experience, the total construction cost for the five lagoons and three pumping stations would likely be less than $20 M. Furthermore, damages to the sewer and wastewater treatment system in the event of a flood would be expected to be only a fraction of this construction cost and accounts for about only 1% or less of the total damages that could occur within the study area. This estimated value of damages to the sewer and wastewater treatment system was incorporated into the total assessed values presented in Table 8.12.

Since detailed modeling of overland flooding would be required to determine the extent of flooding within these communities, which is beyond the scope of this concept phase study, judgment was used to determine the potential flooding extent in these communities. It is recognized that if any breach of the dikes were to occur along the Lower Assiniboine River during a flood, temporary emergency efforts would be put in place within the communities to minimize damages. Nevertheless, it is expected that some level of flooding would occur. For the breach scenarios due to overtopping at the flow of 708 m^3/s (25,000 cfs), it was assumed that

10% of the structures in the communities within Zones 1 to 4 would flood. However, for the structural failure scenario, due to the increased overflow from the larger breach sizes and due to the unexpected nature of the breach type, it would be more difficult to deploy temporary measures to protect the communities. It was assumed that 75% of the structures in the communities within Zones 1 to 4 would be flooded. In Zone 5, based on the results of the overflows to the La Salle River discussed previously, it was assumed that 10% of Starbuck and Sandford would experience some flooding, for the structural failure scenario only. Similarly, for the damage estimates to the structures in the rural areas (discussed above), a rate 70% of market value was applied in the model at the flow of 708 m^3/s (25,000 cfs) based on an assumed 0.9 m (3 ft) depth of flooding.

At the flow of 1,303 m^3/s (46,000 cfs), a "low" and "high" damage estimate was computed to represent the approximate range in uncertainty of the estimate. For the "high" estimate, it was assumed that all communities within all five zones would be completely flooded. This would represent the maximum extent of damages that could be expected for the communities. For the "low" estimate, a more reasonable assumption that only Zones 1 to 4 would be completely flooded, as well as 25% of Starbuck and Sandford, was adopted. Similarly, for the damage estimates to the structures in the rural areas discussed earlier, a rate 100% of market value was applied in the model at the flow of 1,303 m^3/s (46,000 cfs), based on an assumed 1.2 m (4 ft) depth of flooding.

Tables 8.13 and 8.14 summarize the computed damages to buildings and structures for both the overtopping and structural failure scenarios at a flow of 708 m^3/s (25,000 cfs) and for the "low" and "high" estimate values at a flow of 1,303 m^3/s (46,000 cfs). The difference in the number of structures between scenarios is attributed to the difference in flooded area between scenarios obtained from the overflow analysis discussed previously. On the tables, only the totals for all zones combined are provided, as the totals show what was used to develop the discharge damage curve discussed in Section 8.6.1.12. A breakdown of the flooded area by zone is provided in Tables 8.10 and 8.11.

Table 8.13
Damage to Buildings and Structures for 708 m^3/s (25,000 cfs)

Description	Cost of Flood Damages	
	Overtopping Scenario	Structural Failure Scenario
Rural	$ 99,342,000	$ 138,146,000
Communities	$ 12,838,000	$ 104,135,000
Total	$112,180,000	$ 242,281,000

Table 8.14
Damage to Buildings and Structures for 1,303 m^3/s (46,000 cfs)

Description	Cost of Flood Damages	
	"Low" Estimate	"High" Estimate
Rural	$ 231,090,000	$ 281,996,000
Communities	$ 211,436,000	$ 391,671,000
Total	$ 442,526,000	$ 673,667,000

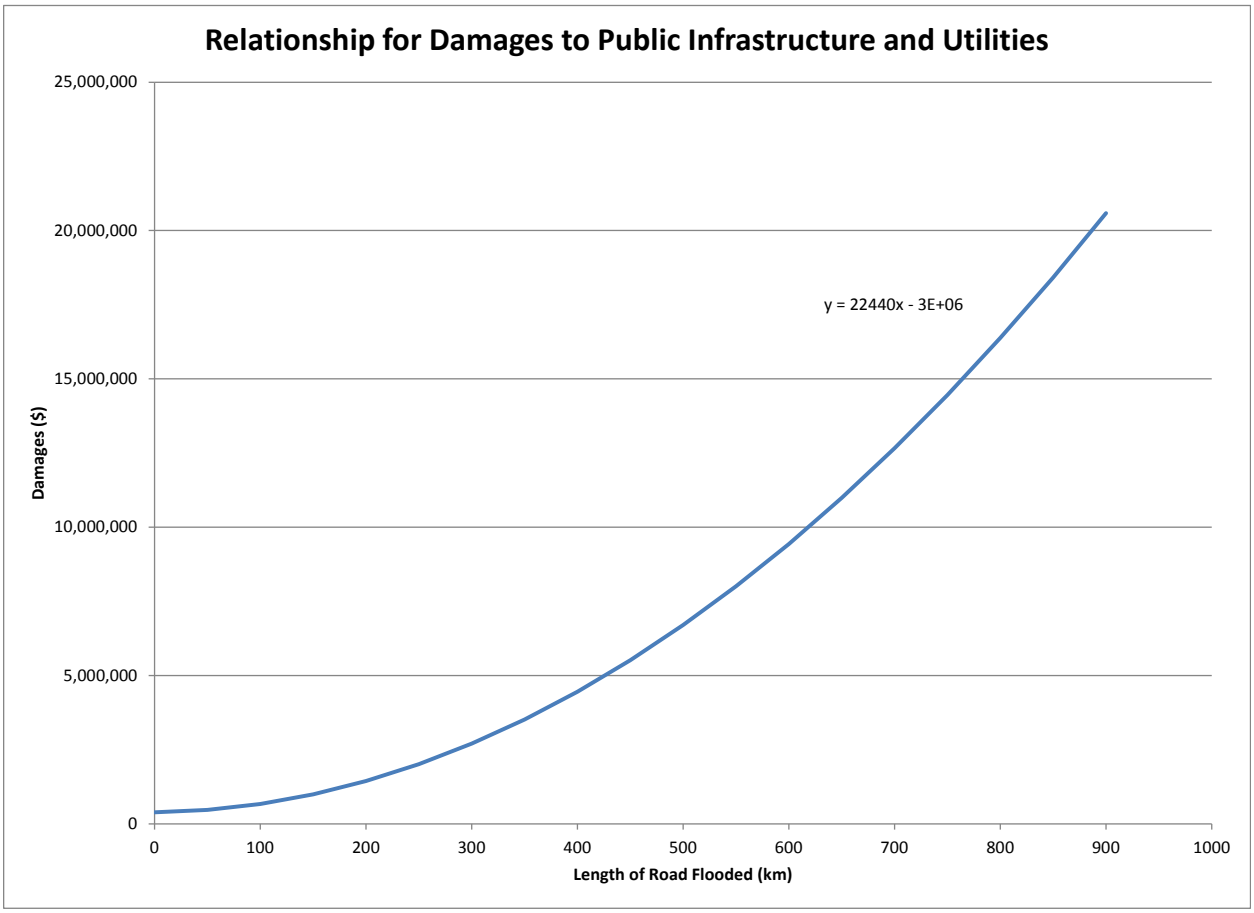
As discussed in Section 8.6.1.4, it was assumed that without dikes or dikes that are 100% unreliable, flood damages would start to occur at a flow of approximately 283 m^3/s (10,000 cfs). Therefore, damages at flows below 708 m^3/s (25,000 cfs) were determined by interpolating from the damages computed at 708 m^3/s (25,000 cfs) and zero damages at 283 m^3/s (10,000 cfs). However, since the base condition for this study assumes that the existing dikes along the Assiniboine River are reliable at a flow of 425 m^3/s (15,000 cfs), the point of incipient flooding in the final Damage-Discharge Relationship was set to 425 m^3/s (15,000 cfs). This is further discussed in Section 8.6.1.12.

8.6.1.8 Damages to Public Infrastructure and Utilities

Since the largest portion of damage costs to public infrastructure and utilities are typically roads, ditches and bridge and culvert crossing repairs, damages for this category were based on the total length of roads located within the flooded area. The relationship of flood damages to the total length of roads flooded developed by KGS Group for the International Joint Commission in 1999 in the studies of flood protection for Winnipeg (KGS Group, 2000b), updated to 2014$, was used for this study. This relationship was considered representative of the damages that

would be expected within the Lower Assiniboine River and La Salle River basins due to the similarity in land use and topography of these two basins to the Red River Basin, as well as their proximity to each other. The adopted relationship of damage costs vs length of flooded road is shown on Figure 8.81.

Figure 8.81
Damage Costs with Respect to Length of Roads Flooded



In the damage model, the total length of flooded road in each of the five zones of overflow discussed in Section 8.6.1.2 was estimated from the flooded area determined from the breach analysis. It was assumed that a 3.2 km (2 mi) length of road would be flooded for every section of land flooded (1 mi^2).

Tables 8.15 and 8.16 summarize the computed damages to public infrastructure and utilities for both the overtopping and structural failure scenarios at a flow of 708 m^3/s (25,000 cfs) and for

the "low" and "high" estimate values at a flow of 1,303 m^3/s (46,000 cfs). The difference in length of road flooded between scenarios is attributed to the difference in flooded area between scenarios obtained from the overflow analysis discussed previously. On the tables, only the totals for all zones combined are provided, as the totals are what were used to develop the discharge damage curve discussed in Section 8.6.1.12. A breakdown of the flooded area by zone is provided in Tables 8.10 and 8.11.

Table 8.15
Summary of Damages to Public Infrastructure and Utilities at 708 m^3/s (25,000 cfs)

Overtopping Scenario		Structural Failure	
Length of Road Flooded (km)	Cost of Flood Damages	Length of Road Flooded (km)	Cost of Flood Damages
798	$ 9,406,000	621	$ 12,229,000

Table 8.16
Summary of Damages to Public Infrastructure and Utilities at 1,303 m^3/s (46,000 cfs)

"Low" Estimate		"High" Estimate	
Length of Road Flooded (km)	Cost of Flood Damages	Length of Road Flooded (km)	Cost of Flood Damages
1300	$ 13,084,000	1586	$ 17,051,000

As discussed in Section 8.6.1.4, it was assumed that without dikes or dikes that are 100% unreliable, flood damages would start to occur at a flow of approximately 283 m^3/s (10,000 cfs). Therefore, damages at flows below 708 m^3/s (25,000 cfs) were determined by interpolating from the damages computed at 708 m^3/s (25,000 cfs) and zero damages at 283 m^3/s (10,000 cfs). However, since the base condition for this study assumes that the existing dikes along the Assiniboine River are reliable at a flow of 425 m^3/s (15,000 cfs), the point of incipient flooding in the final Damage-Discharge Relationship was set to 425 m^3/s (15,000 cfs). This is further discussed in Section 8.6.1.12.

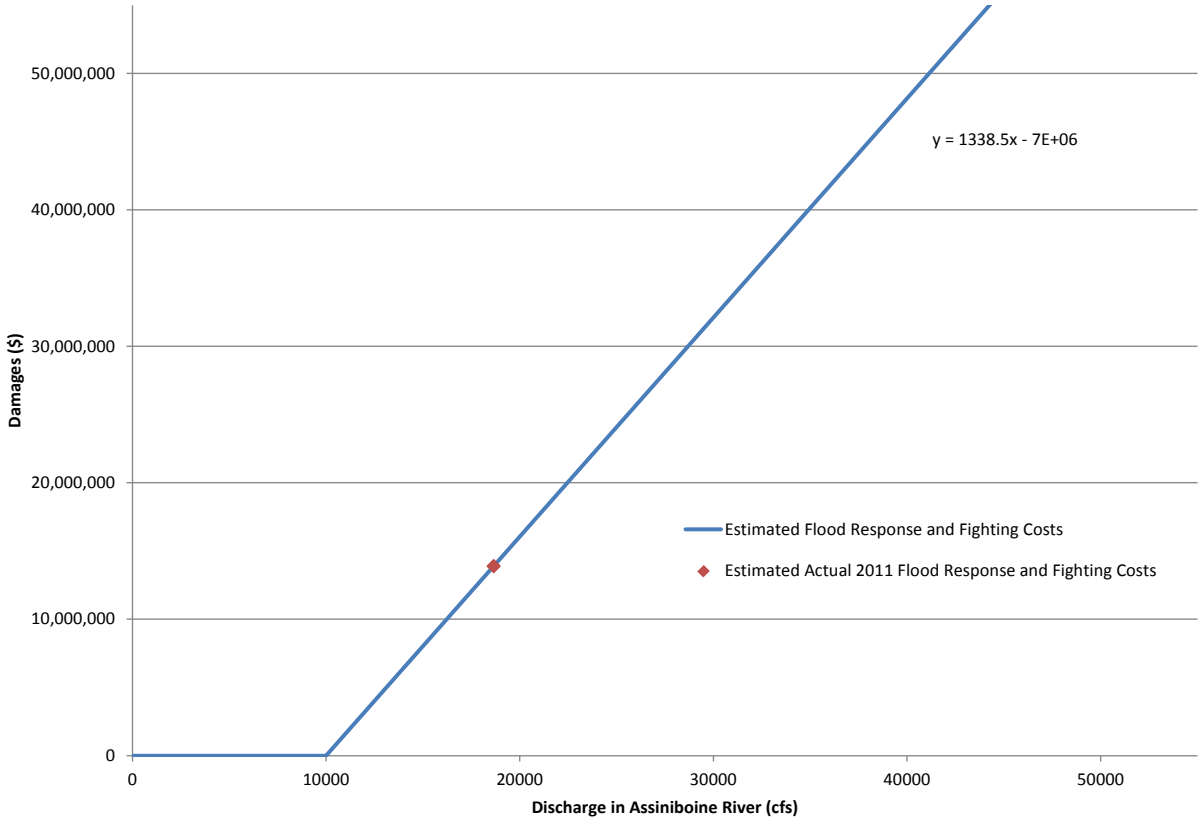
8.6.1.9 Flood Response and Fighting Costs

During the 2011 Flood, the majority of the $13,900,000 flood response and fighting costs on the Lower Assiniboine River outlined in Section 8.6.1.5 were associated with efforts to maintain an

approximate 510 m^3/s (18,000 cfs) capacity on the dikes that border the river. This formed the basis for a relationship of flood response and fighting costs with respect to discharge.

As discussed in Section 8.6.1.4, at a flow of approximately 283 m^3/s (10,000 cfs), the Assiniboine River water surface elevation approximately equaled the natural bankfull conditions in the river (Hatch, 2011a) and was, therefore, near the toe of the existing dikes. As a result, it was assumed that flood response and fighting costs should be zero at 283 m^3/s (10,000 cfs). The adopted relationship is shown on Figure 8.82.

Figure 8.82
Relationship for Flood Response and Fighting Costs



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

Since the relationship for flood response and fighting costs was based on the Lower Assiniboine River discharge, the costs of damages for this category were independent of the breach

scenario. The same value was also used for the "low" and "high" estimates at a flow of 1,303 m^3/s (46,000 cfs).

8.6.1.10 Disruption to Transportation

In the event of a severe flood on the Lower Assiniboine River, there is the potential for severe disruption of transportation, not only impacting the interests of Winnipeggers and other stakeholders and communities in Manitoba, but also neighbouring Provinces, and the country as a whole. For a severe flood condition where the Lower Assiniboine River flows would be 1,303 m^3/s (46,000 cfs), closures (and flooding) of highways such as PTH 1, PTH 13 and PTH 26, as well as provincial roads such as PR 248 and PR 424 would likely be unavoidable. Depending on how the flood flows translated downstream on the La Salle River, PTH 2 and PTH 3 could also be affected, as well as PR 332, PR 334 and PR 247 to name a few. Railway transportation would also be affected, most notably the Canadian National (CN) Rivers Subdivision, which is CN's mainline connecting Winnipeg to the west.

For smaller floods, with flows of 708 m^3/s (25,000 cfs) or less, disruption to transportation would likely be minimal in extent and localized only. This was the case during the flood of 1955, when widespread flooding occurred along the Lower Assiniboine River and flows reached approximately 637 m^3/s (22,500 cfs). On this basis, damages from disruption to transportation were assumed to be zero in the damage model for flows of 708 m^3/s (25,000 cfs) or less.

At a flow of 1,303 m^3/s (46,000 cfs), the procedure used to assess the direct damages from such disruption was to estimate the volume of traffic affected and the additional costs in routing that traffic via other routes that were unaffected by the flooding. This method was consistent with the methods used in other studies such as the studies of flood protection for Winnipeg for the International Joint Commission in 1999 (KGS Group, 2000b). Furthermore, the inconvenience and delay caused by this impact could adversely affect the ability of businesses in other parts of Canada in their normal production of goods and services. This indirect influence was not included in the estimation of damages.

According to the University of Manitoba's "Manitoba Highways and Traffic Information System" (U of M, 2014), an annual average daily traffic total of approximately 13,000 vehicles travelled

on PTH 1 in 2012. Traffic counts on other smaller highways and provincial roads vary with ranges between approximately 500 to 2000 vehicles. On this basis, a minimum traffic count of approximately 15,000 vehicles was used for the "low" estimate, while a conservative value of 20,000 vehicles was used for the "high" estimate. During the flood, some of the traffic could be rerouted to the north on PR 227, and to the south on PTH 3 or as far as PTH 23. The detour would add approximately 50 to 100 km kilometers of travel between Portage la Prairie and Winnipeg depending on the route and could add an additional two hours of time to each trip due to busier roadways causing traffic delays. For the damage estimates, an average detour length of 75 km was used for both the "low" and "high" estimates. This was assumed to last for two months, based on the design hydrographs for the study shown on Figure 8.74 and considering the additional time necessary for cleanup and repairs after the flood.

Assuming an average rate of $0.50/km per vehicle for the cost of fuel and vehicle maintenance, as well as an average rate of $50/h for driver time and other delays, the total cost of damages due to road closures at a flow of 1,303 m^3/s (46,000 cfs) was $90 million for the "low" estimate and $165 million for the "high" estimate.

Based on discussions with the railways, it was estimated that a closure to a mainline would cost approximately $100,000 an hour. Assuming that the CN Rivers subdivision mainline is closed for a duration of two months during a 1,303 m^3/s (46,000 cfs) flood, this would equate to $144 million. To account for the uncertainty with this very crude estimate, a rounded value of $125 million was used for the "low" damage estimate and of $175 million for the "high" damage estimate.

8.6.1.11 Business Losses, Stress and Anxiety, and Other Miscellaneous Damages

Due to complexities in accurately defining the number of businesses affected within the Lower Assiniboine region during a flood and the extent of damages they incur, business losses were estimated based on a percentage of the sub-total of all other damages computed for the region. As shown in Table 8.9 of Section 8.6.1.5, business losses during the 2011 Flood event were estimated to equal approximately 2% of the total damages. Due to uncertainties with the data and whether all business losses were truly accounted for, a conservatively high 5% value was adopted for this study.

Also, as discussed in Section 7.2 of the report, due to challenges in quantifying stress and anxiety into a stage-damage model, this uncertain type of flood damage was accounted for with a percentage allowance. Similar to the Flood Protection Studies for Winnipeg (KGS Group, 2000c), an additional allowance equivalent to 20% of the building and agricultural damage estimates was included to account for stress and anxiety.

Based on discussions with a representative cross section of stakeholders and government officials during project meetings for this study, it was recognized that the level of compensation and damages for the 2011 Flood discussed in Section 8.6.1.5 underestimated the true cost incurred by the stakeholders around the Province. To account for these unidentified additional damages, KGS Group applied a 25% increase to the total damages estimated using their best judgment and the approaches described above.

8.6.1.12 Final Damage-Discharge Relationship

The total computed damages for the overtopping and structural failure scenarios at a flow of 708 m^3/s (25,000 cfs) is summarized in Table 8.17. The total computed damages at a flow of 1,303 m^3/s (46,000 cfs) is summarized in Table 8.18.

Table 8.17
Total Computed Damages at 708 m^3/s (25,000 cfs)

Overtopping Scenario	Structural Failure Scenario
$ 349,883,000	$ 629,496,000

Table 8.18
Total Computed Damages at 1,303 m^3/s (46,000 cfs)

"Low" Estimate (Overtopping or Structural Failure)	"High" Estimate (Overtopping or Structural Failure)
$ 1,323,698,000	$ 1,966,673,000

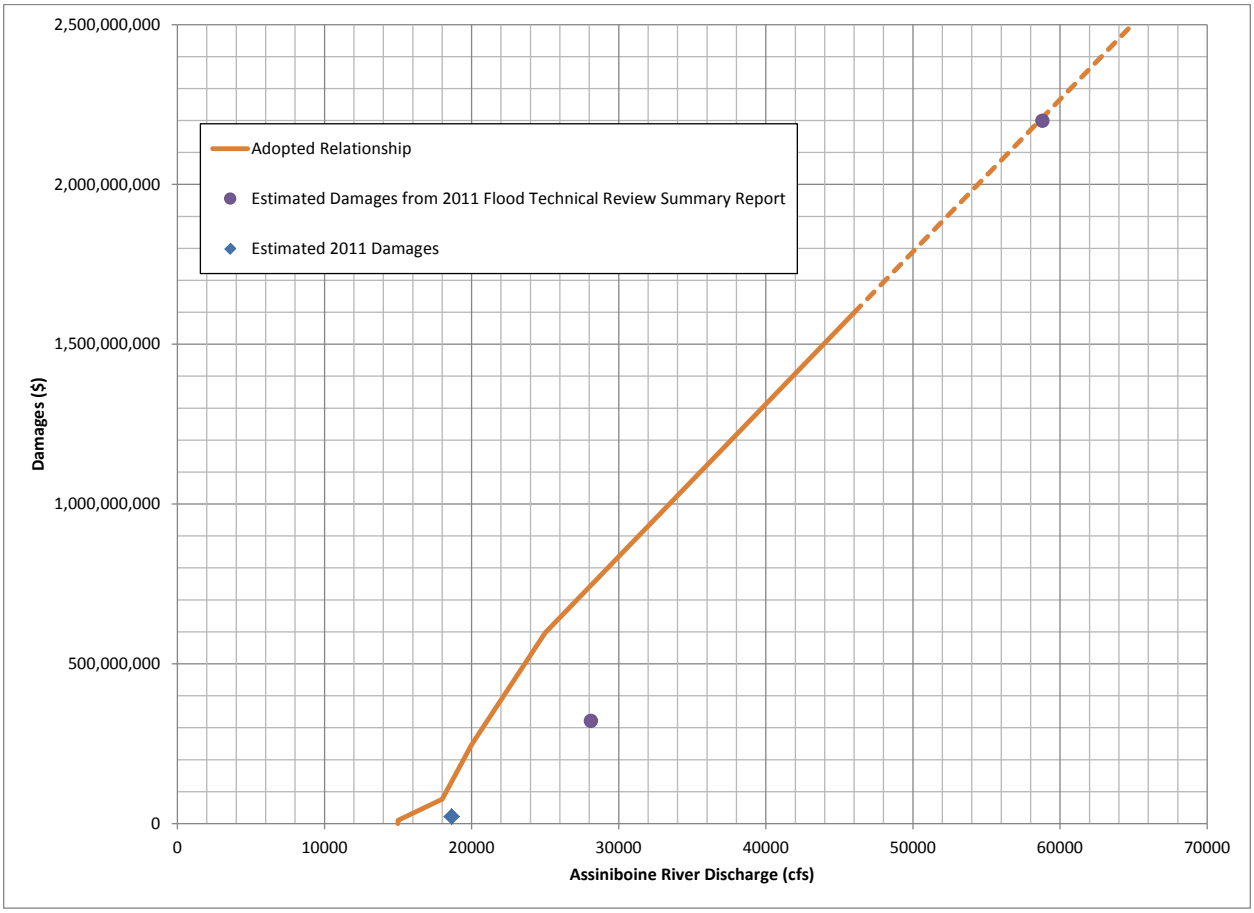
Based on the computed damages at a flow of 708 m^3/s (25,000) cfs and 1,303 m^3/s (46,000 cfs) outlined in Tables 8.17 and 8.18, as well as the various assumptions discussed in this report, a

final discharge-damage curve was developed to represent the base condition for this study. The following lists the key assumptions that formed the basis of the final discharge-damage curve:

- The existing dikes along the Assiniboine River are reliable at a flow of 425 m^3/s (15,000 cfs) or less.
- Without dikes (or completely unreliable dikes), damages would start to occur at a flow of 283 m^3/s (10,000 cfs).
- The point where overtopping starts to occur is 651 m^3/s (23,000 cfs).
- Above 425 m^3/s (15,000 cfs), the existing dikes are 80% reliable at 509 m^3/s (18,000 cfs), 40% reliable at 566 m^3/s (20,000 cfs) and no longer reliable at 708 m^3/s (25,000 cfs).
- At a flow of 1,303 m^3/s (46,000 cfs), a midpoint between the "high" and the "low" estimated damage values was adopted to represent an average condition during the economic analysis.
- Above 1,303 m^3/s (46,000 cfs), the damages can be extrapolated linearly based on the slope of the adopted curve between 708 m^3/s (25,000 cfs) and 1,303 m^3/s (46,000 cfs).

The final discharge-damage curve adopted for this study is provided on Figure 8.83. On the figure, the points representing the actual estimated cost of the 2011 damages, and the potential estimated damages outlined in the "2011 Flood: Technical Review of Lake Manitoba, Lake St. Martin and Assiniboine River Water Levels – Summary Report" (Manitoba, 2013b), discussed in Section 8.6.1.5, are also included for comparison.

Figure 8.83
Discharge-Damage Relationship for the Lower Assiniboine River



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

As shown on the figure, for a 2011 Flood magnitude of 528 m^3/s (18,650 cfs), the computed damages from the model are approximately five times larger than the actual estimated values for that flood. This is because an uncontrolled breach of the dikes in 2011 did not occur. If a breach of the dikes were to have occurred in 2011, the damages along the Lower Assiniboine River would have been many times larger, possibly five times larger, as per the model results, or possibly even more and exceeding the computed values. That is because the adopted damage curve shown on Figure 8.83 represents the average cost of damages that would be expected for any given flood magnitude. The actual damages that would be expected to occur during a flood would be more or less than the average values, depending on the unique conditions of that flood.

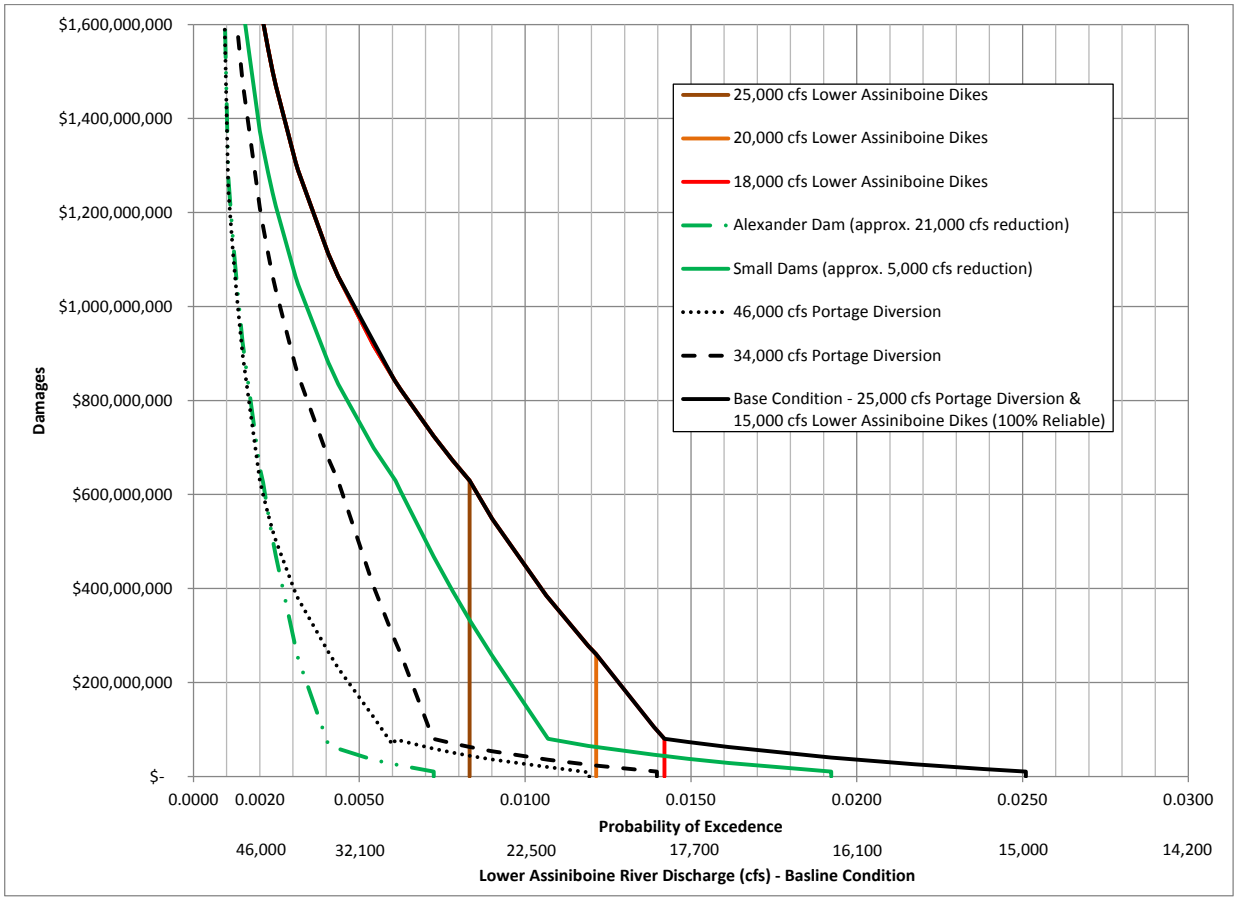
Also, as shown on the figure, the adopted curve for this study is in the same order of magnitude as the estimated damages from the 2011 Flood Technical Review Summary Report (Manitoba, 2013b) at flows of 796 m^3/s (28,100 cfs) and 1,665 m^3/s (58,800 cfs). This demonstrates that the damage model developed for this study yields results that are consistent with other independent estimates. The comparison also suggests that adopting the conservative assumptions discussed in the document was not an unreasonable assumption. This approach is consistent with that used to develop the Lake Manitoba damage model, as well as the Red River and City of Winnipeg damage model developed for the current study (Sections 8.6.2 and 8.6.3). The consistent approach will allow for a relative comparison between the benefit / cost ratios of the different flood mitigation projects.

Qualitative consideration should also be given to the relative effects of intangible aspects such as stress and anxiety by acknowledging the number of people affected, the duration of stress imposed and generally comparing the event to other catastrophes. This will be analyzed and documented separately as part of this study.

8.6.1.13 Estimated Benefits of Flood Mitigation Alternatives

Based on the Discharge-Damage Relationship outlined in Section 8.6.1.12, as well as the discharge frequency curve of the Lower Assiniboine River presented in Section 5.2, a frequency-damage curve for the Lower Assiniboine River was developed and is shown in Figure 8.84. This formed the baseline condition from which to calculate the benefits of the different flood mitigation alternatives considered for this study. As discussed in Section 8.6.1.12, the baseline condition assumes a 708 m^3/s (25,000 cfs) Portage Diversion capacity and Assiniboine River dikes reliable at flows of 425 m^3/s (15,000 cfs) or less. The frequency-damage relationships of several different flood mitigation alternatives are also presented on Figure 8.84.

Figure 8.84
Frequency-Damage Relationship for the Lower Assiniboine River



Note: Flows can be converted to m^3/s by dividing imperial values by 35.3.

As shown on Figure 8.84, the frequency-damage curves that represent different capacities of the Lower Assiniboine River dikes are essentially the same (i.e., the curves overlap), except that the point of incipient flooding changes depending on the dike capacity.

For the options of increasing the Portage Diversion capacity, or constructing storage reservoirs, the frequency-damage curves are “shifted”. This is because these types of flood mitigation alternatives reduce the discharge in the Lower Assiniboine River for a given return period. For example, the flood mitigation alternative of small dams reduces the flow by approximately 142 m^3/s (5,000 cfs) on the Assiniboine River. The magnitude of the flow reduction from those alternatives is based on the results of the analysis of the flood mitigation alternatives discussed in this report.

Based on the frequency-damage curves presented in Figure 8.84, the average annual damages of the baseline condition and of the different flood mitigation alternatives were computed. The average annual damages were determined by calculating the area on the graph that was under the frequency-damage curve. The difference between the average annual damages of the baseline condition and those of the flood mitigation alternatives formed the average annual benefits of the options. Based on a 50-year economic life, and assuming a 7% interest rate, 3% inflation rate, and 1% growth rate, the present value of benefits of the options were computed. These values are summarized in Table 8.19.

Table 8.19
Summary of Flood Damages and Benefits
for Different Flood Mitigation Alternatives

Flood Mitigation Alternative	Average Annual Damages ($ - Million)	Average Annual Benefits ($ - Million)	Present Value of Benefits ($ - Million)
Base Condition (15,000 cfs Lower Assiniboine River Dikes)	14.68	n/a	n/a
18,000 cfs Lower Assiniboine River Dikes	14.20	0.48	12.30
20,000 cfs Lower Assiniboine River Dikes	13.86	0.83	21.05
25,000 cfs Lower Assiniboine River Dikes	12.30	2.39	60.72
34,000 cfs Portage Diversion	8.49	6.20	157.56
46,000 cfs Portage Diversion	5.51	9.17	233.22
Multitude of Small Dams	11.06	3.63	92.24
Alexander Dam	4.95	9.73	247.46

The results outlined in Table 8.19 form one component of the benefits that were used to determine the benefit / cost ratio of the different flood mitigation alternatives. Benefits (or dis-benefits) in other areas of the Lake Manitoba and Assiniboine River basins are discussed in the next sections of the report.

8.6.2 Lake Manitoba

An integrated Excel and GIS based model was developed to compute damages along the shoreline of Lake Manitoba. Damages were computed for ten separate flood magnitudes to capture the entire range of probable flood events on the lake.

A peak water level on Lake Manitoba was determined for each flood event based on the Still Water Level (SWL) frequency curve presented in Section 8.5.1, as well as the Combined Water Level frequency Curves (CWL) presented in Section 8.5.2. Since the potential effects of wind setup to peak water levels vary by location around the lake (as discussed in Section 8.4), Lake Manitoba was subdivided into 14 zones in the damage model to account for the varying frequency curves of the CWL by location. Each zone had a unique water level for a given flood. To achieve a conservatively high estimate of potential damages in the analysis, it was assumed that wind setup would contribute to all damages on Lake Manitoba, even if certain types of damages were not caused by wind setup (for example, temporary flooding from wind setup may not necessarily cause damage to a crop, but it has been included nevertheless).

For damages to buildings and structures only, a 0.7 m (2.25 ft) allowance for wave uprush was added to the water levels obtained from the CWL frequency curves. As discussed in Section 8.4.3, this value was considered to be conservative when applied to all buildings and structures around Lake Manitoba, regardless of location.

For each of the 14 zones of Lake Manitoba, a Digital Elevation Model (DEM) was developed from readily available GIS data; and the following relationships were generated:

- Flooded area (by land use) vs stage.
- Length of road flooded vs stage.
- Number of buildings and structures flooded vs stage.

Land use information is based on the LandSAT7 land use classification from the Manitoba Land Initiative (MLI) captured between 2000 and 2002. All infrastructure including transportation lengths, buildings and structures were based on vector data obtained from MLI. The vector data was derived from 1990 to 1996 1:60,000 imagery that was ortho-rectified to obtain ground accuracy of +/-5 m. Based on a comparison of recent aerial photos to the MLI data at representative locations around Lake Manitoba, it was determined that the number of buildings and structures around the lake had increased by approximately 38% over the past 25 years. Therefore, the number of buildings and structures identified in the model was increased by a conservative 40% value.

The Shuttle Radar Topography Mission (SRTM) Level 1 regional digital elevation model was chosen to determine Lake Manitoba shoreline elevation. The SRTM DEM is currently the only dataset that provides a continuous and seamless elevation model for the Lake Manitoba shoreline. Existing 1:50,000 topography is also available, but is not seamless and is only available in 10 m contour intervals. The regional accuracy of the SRTM DEM is 3 to 5 m (approximately 10-16 ft). As a result, extrapolations and adjustments to the data were necessary to obtain the level of detail required for the damage model. However, even after the adjustments, it was evident that the elevation of incipient damages was difficult to determine with precision from the GIS data. Based on a review of referenced reports available for this study, it was determined that there were little to no damages when still water levels on Lake Manitoba were below 247.3 m to 247.5 m (811.5 to 812.0 ft). On this basis, zero damages were assumed for a SWL of 247.5 m (812.0 ft) or less on the lake. This was approximately equivalent to a 1:4 year event. This SWL formed the base condition for this study. The sensitivity associated with applying a lower incipient flood level to the results of the economic analysis has been documented in Section 22.2 of this report.

For each magnitude of flood considered, damages were estimated in each of the 14 zones based on the methods discussed in the sections that follow. The summation of the damages from the different zones formed the total damages for Lake Manitoba.

8.6.2.1 Estimation of Damages

A list of damages considered for this study is outlined in Section 7.2 of the report. To facilitate the estimation of damages on Lake Manitoba, damages were grouped into the following five categories:

1. Agricultural Damages.
2. Damages to Buildings and Structures.
3. Damages to Public Infrastructure and Utilities.
4. Flood Response and Fighting Costs.
5. Business Losses, Stress and Anxiety and other Miscellaneous Damages.

The sections that follow describe how the damages for each category were applied to Lake Manitoba. Data from the 2011 Flood was used to calibrate the flood damage model as discussed below.

8.6.2.2 Data on 2011 Flood Damages and Mitigation

The 2011 Flood damage data and post-flood mitigation initiatives were obtained from various provincial organizations for the entire Province of Manitoba, including the Emergency Measures Organization (EMO), Manitoba Agricultural Services Corporation (MASC), Manitoba Agriculture, Food and Rural Development (MAFRD), and Manitoba Infrastructure and Transportation (MIT). Each organization was responsible for separate types of programs, which provided financial assistance or compensation for damages related to the 2011 Flood, or for post-flood mitigation initiatives. For example, the total provincial awarded amount through the Disaster Financial Assistance Program (DFA) for the 2011 Flood, administered by EMO, was approximately $570 million as of fall 2013. Also, the total compensation value ($2014) for the Lake Manitoba Financial Assistance Program, administered by MASC, was approximately $105 million. A summary of the data obtained for the study is provided in Appendix E.

The 2011 Flood damage data was used to calibrate the flood damage model whereas the post-flood mitigation initiatives data were used to estimate what assets were already protected to determine the level of damages that would no longer apply in future floods.

Where possible, a GIS analysis was conducted to isolate Lake Manitoba damages from the Province-wide totals and to subdivide the data into the five categories described in Section 8.6.2.1.

In instances where insufficient details and spatial breakdown were provided with the data, KGS Group applied their best judgment to isolate Lake Manitoba damages and to assign the data to one of the five categories. This was done based on results from the GIS analysis for the remaining data, a review of the program descriptions and discussions with the different program administrators.

Several programs such as the Disaster Flood Assistance (DFA) Program and the Excess Moisture Insurance (EMI) included a 20% deductible prior to providing compensation to the different stakeholders. Where this was the case, the 2011 damage values were increased by 25% to best represent the total damages that actually occurred. Furthermore, some programs such as the Building and Recovery Action Plan – LMB Part C program were limited to a maximum. However, data was not available to determine the extent of damages that were incurred beyond the maximum limit in the compensation program. Consequently, the 2011 compensation values for buildings and structures were doubled to maintain a conservative estimate of the potential flood damages in the analysis.

Table 8.20 summarizes KGS Group's best estimate of the total Lake Manitoba 2011 damages for the five categories described previously. The total value of $269.9 M represents approximately 20% to 30% of the total costs of the 2011 Flood that were estimated at approximately $1.2 billion (Manitoba 2011 Flood Review Task Force, 2013a). This value was, therefore, deemed to be in the right order of magnitude for the actual damages that occurred around Lake Manitoba.

Table 8.20
Estimated 2011 Flood Damages for Lake Manitoba

Agricultural Damages	$35,800,000
Damages to Buildings and Structures	$111,800,000
Flood Response and Fighting Costs	$73,900,000
Damages to Public Infrastructure and Utilities	$45,600,000
Business Losses	$2,800,000
Total	$269,900,000

For the purpose of the calibration, an estimate of the return period for the 2011 Flood event was required. However, depending on the type of damages considered, the event that caused the damages and its magnitude varied. For example, agricultural damages were primarily attributed to the peak still water level on the lake, while structural damages were mostly due to the May wind storm, which impacted each area of the lake differently. On this basis, the return period of the 2011 Flood event was calculated in four different ways based on the frequency curves described in Section 8.5 of this report. Table 8.21 summarizes the estimated range in

magnitude of the 2011 Flood event on Lake Manitoba. For the purposes of the damage model, it was assumed that the 2011 Flood had a return period of 1:340 years (a probability of being equal or exceeded of approximately 0.003). This value is low when compared to the return period determined based on the surveyed water level by MIT at Twin Lakes Beach. Nevertheless, the 1:340 year return period was adopted to maintain a conservative approach in the analysis, as using the high return period value of 1:2130 year shown on the table would have resulted in considerably lower damage estimates in the model.

Table 8.21
Estimated Return Period of 2011 Flood Event

Description	El (ft)	Estimated Return Period (yr)
Instantaneous Peak - WSC at Westbourne (combined event)	817.75	1:230
Instantaneous Peak - MIT at Twin Lakes Beach (combined event)	820.37*	1:2130
Instantaneous Peak - WSC at Steep Rock (combined event)	817.46	1:340
Wind Eliminated Peak (still water level)	817.1	1:310

* Value obtained from MIT February 16, 2012 memorandum titled, "Flood Protection Level Determination – Lake Manitoba South Pool" (MC&WS, 2012a).
** Elevations can be converted to metres by multiplying imperial values by 0.3048.

After the 2011 Flood, the Province of Manitoba launched the Individual Flood Proofing Initiative (IFPI) and Financial Assistance for Cottage Owners (FACO) programs, as well as the Community Flood Protection Program (CFPP) and the Emergency to Permanent (E-to-P) programs. These programs provided financial assistance to communities and owners of flood prone homes, farms, businesses and cottages throughout Manitoba to flood protect their buildings and structures by raising, moving or diking. As a minimum, stakeholders around Lake Manitoba who have benefited from these programs have undertaken mitigation measures to protect against a 2011 type flood. Based on a review of the data available for this study, it was estimated that the majority of the structures around the lake that were previously flood prone to a 2011 type flood would now be protected for a repeat flood of this magnitude. For this study, it was assumed that 80% of structures would be protected to this level and would only start to incur damages for floods larger than the 2011 Flood. An estimate of 20% of all structures was

adopted to represent those that would continue to incur damages at lower floods. This would essentially assume that 20% of structures were not protected or not viable to protect.

However, new information on the IFPI and FACO programs has shown that the number of stakeholders who participated in the programs to undertake mitigation measures was closer to 25%, which is less than originally anticipated. Since this information became available after the analyses described herein were already completed, this has not formed the basis of the damage-frequency relationship used in the benefit / cost analysis. Rather, a sensitivity analysis was conducted to determine how the resultant average annual damages reported in Section 8.6.2.8 would have changed if 25% of the structures were to be protected to a 2011-type flood instead of the 80% assumed (i.e., 75% of structures would not be protected instead of the 20% assumed). The results of this sensitivity analysis are discussed in Section 8.6.2.8.

8.6.2.3 Agricultural Damages

Agricultural damages on Lake Manitoba were calculated on the basis of area, based on established relationships between stage and flooded areas of the lake. Crop losses were based on average unit rates for Lake Manitoba. A unit rate of $800 per hectare was applied to the value of the agricultural land seeded for annual crops based on data obtained from MASC while a unit rate of $300 per hectare was applied to forage crops and grassland based on data obtained from the 2010 Shoal Lake Watershed Study (KGS Group, 2010) escalated to 2014$.

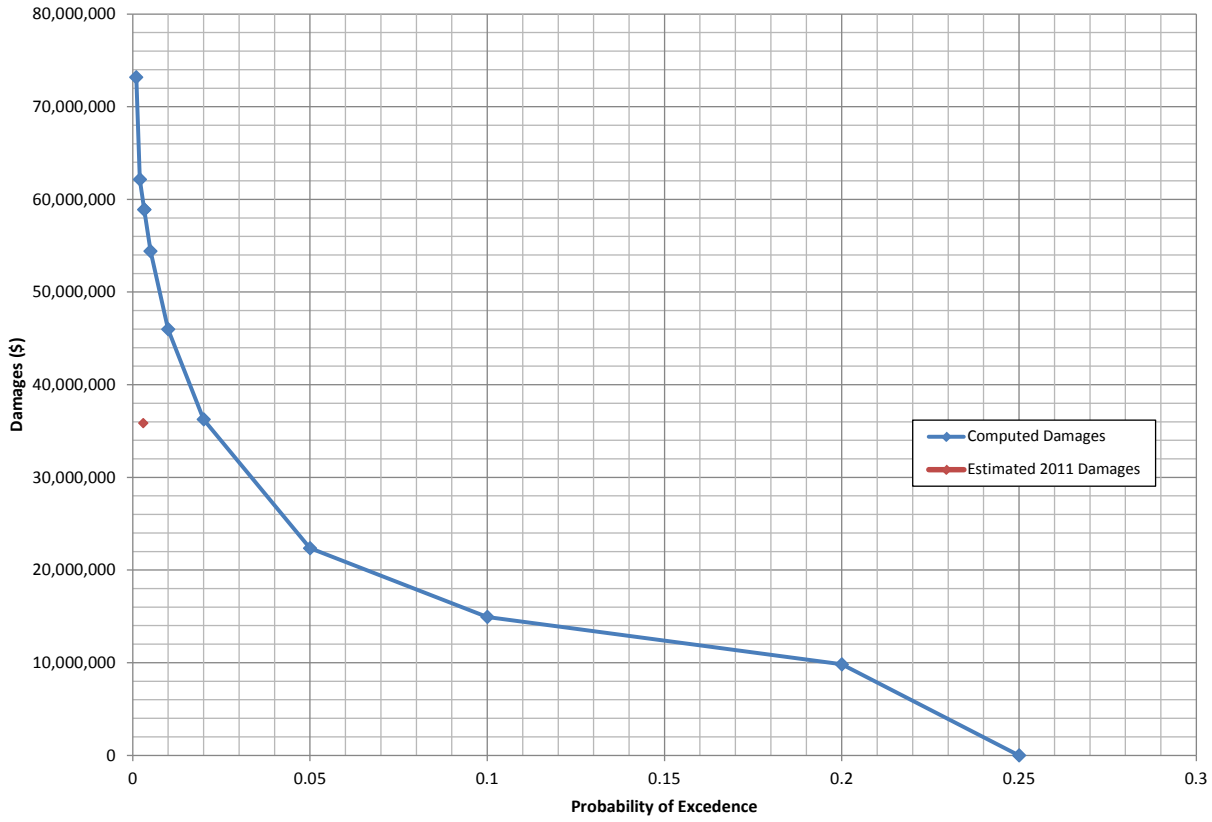
Since damages linked to livestock are closely related to the extent of forage crops and grassland flooded, an increase of 50% was applied to the damages to forage and grassland crops to include losses linked to livestock.

Other types of agricultural damages such as damages to agricultural infrastructure, fences, crop inventory or cost for debris cleanup were difficult to quantify and were generally not separated from crop losses in the 2011 data. To account for these damages, a 50% increase was applied to the total agriculture damages described above based on judgement and input received from various stakeholders who contributed to the study.

Due to the duration of the floods around Lake Manitoba, which often occur over multiple seasons, or extend beyond a single year as was the case in 2011 / 2012, prolonged periods of standing water can cause a reduction in agricultural productivity for several years, due to for example, soil de-nitrification or salinization. To account for this and based on judgement and input received from various stakeholders for the study, an additional 75% increase was applied to the agricultural damages: 50% to represent residual damages that occur one year after the flood, and an additional 25% for the year after.

The relationship between frequency and agricultural damages for Lake Manitoba is provided in Figure 8.85. As shown on the figure, this damage-frequency curve is slightly higher than the estimated damages that occurred in the 2011 Flood at a value of $59.4 million with an annual probability of being equaled or exceeded of approximately 0.003. This demonstrates that the flood damage model provides a conservative, but reasonably accurate representation of estimating the agricultural damages around the lake.

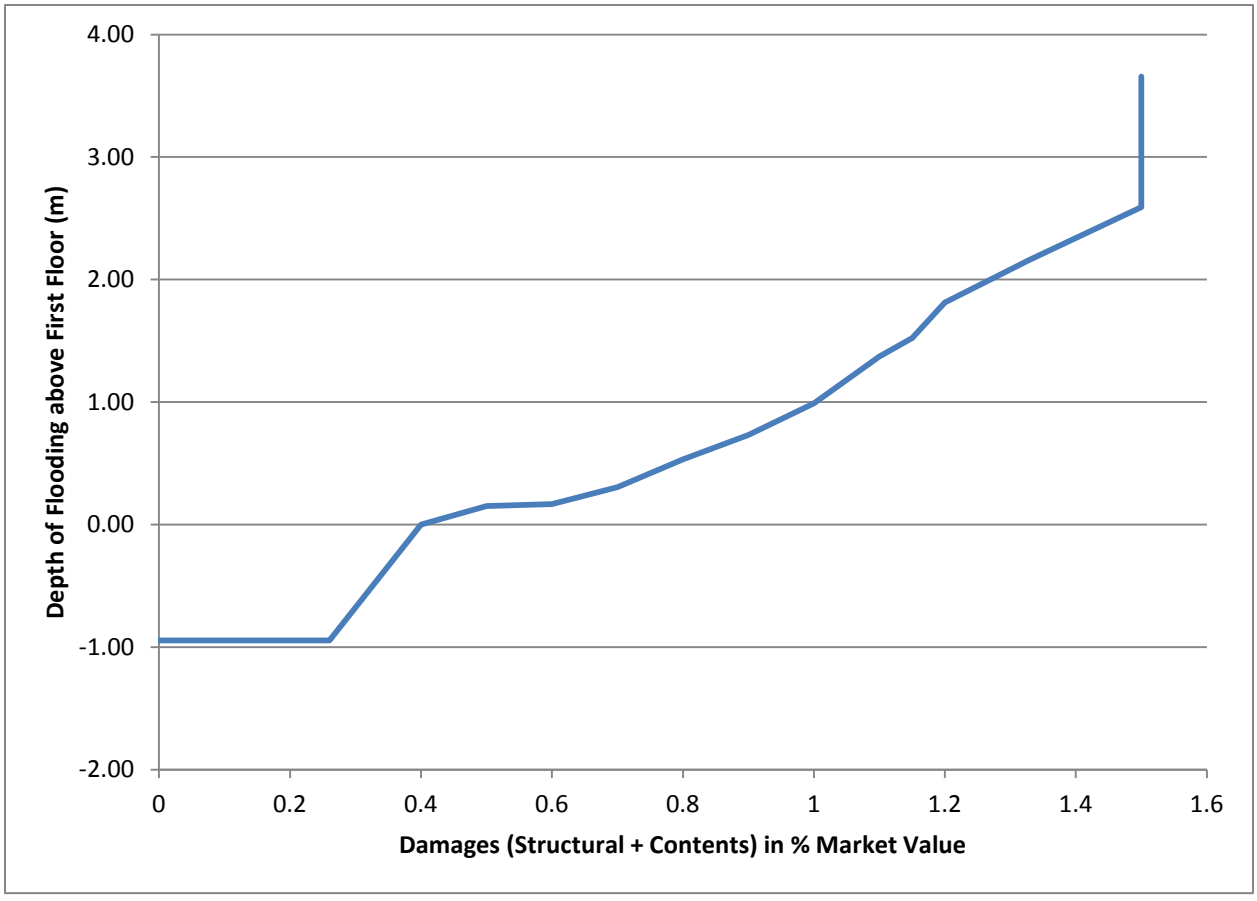
Figure 8.85
Computed Agricultural Damages for Lake Manitoba



8.6.2.4 Damages to Buildings and Structures

Damages to buildings and structures were based on a Depth-Damage Relationship for Lake Manitoba. The adopted relationship for this study was based on a similar relationship developed by KGS Group for the International Joint Commission in 1999 in the studies (KGS Group, 2000b) of flood protection for Winnipeg. It is provided on Figure 8.86.

Figure 8.86
Depth-Damage Relationship for Buildings and Structures for Lake Manitoba



Since the data available for this study did not differentiate between the types of structures affected (i.e., house vs garage vs barn…) and the elevation of the main floor compared to the adjacent ground was not available, the following simplifying assumptions were adopted:

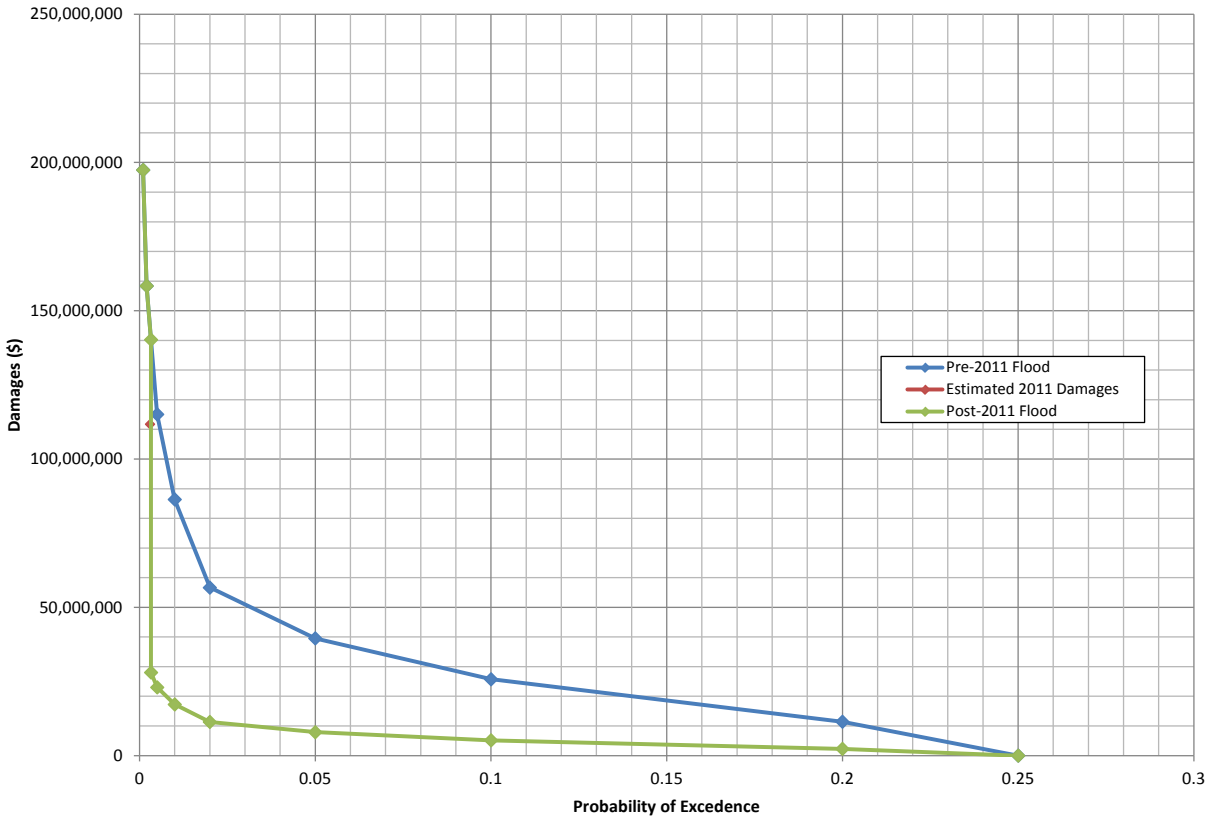
- Each structure has a crawl space.

- The main floor elevation of each structure was estimated to be 0.75 m (2.5 ft) above existing ground (assumed four steps from ground up to main floor, at approximately 0.18 m (0.6 ft) per step).
- The base of the crawl space was assumed to be 0.2 m (0.7 ft) below adjacent existing ground.
- Damages begin to accrue at the base of the crawl space.

An average assessed value of $55,000 per structure was adopted for this study based on a review of property assessment reports available from the Government of Manitoba – Property Assessment website (Manitoba, 2014a). This value appears very low compared to the average cost of a house or cottage around Lake Manitoba. However, since this was an average unit rate for all structure types in the area, including small garages and barns, the lower value should be expected.

Temporary flood mitigation measures that were put in place during the 2011 Flood resulted in a large number of buildings and structures being protected from damages during the flood. Without considering the temporary measures, the model over-estimates the extent of damages for a 2011 type flood. The total number of buildings and structures in the model (estimated at about 700 for a 1:10 year flood and over 3,000 for a 1:500 year flood) was, therefore, reduced by 50% when considering these damages. The resultant frequency-damage curve for buildings and structures for Lake Manitoba is provided in Figure 8.87. As shown on the figure, with this reduction in the model and the increase of the estimated 2011 damages to buildings and structures discussed in Section 8.6.2.2, the damage-frequency curve approximates the estimated damages that occurred in the 2011 Flood at a value of $111.8 M and an annual probability of being equaled or exceeded of approximately 0.003. This demonstrates that the flood damage model provides a reasonably accurate representation of the damages to buildings and structures around the lake.

Figure 8.87
Computed Damages to Buildings and Structures for Lake Manitoba



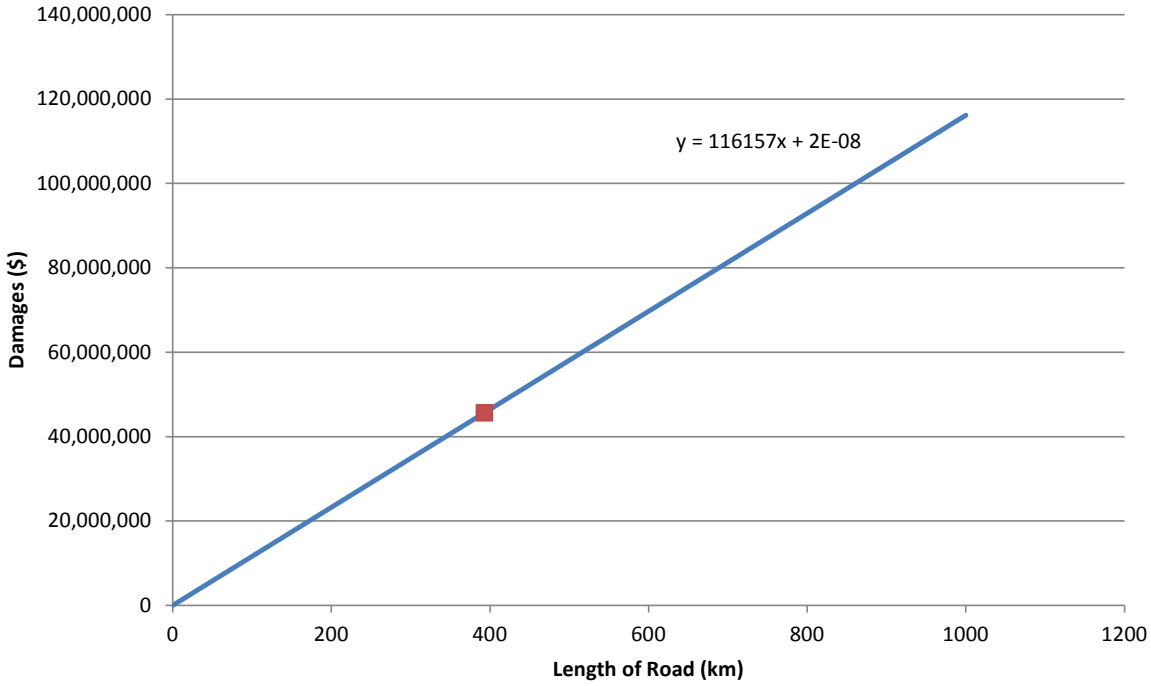
As discussed in Section 8.6.2.2, flood mitigation programs initiated by the Province of Manitoba were anticipated to result in the protection of the majority of buildings and structures against a repeat of the 2011 Flood. As such, the base condition for this study considered that most buildings and structures around the lake would only start to incur damages for floods larger than the 2011 event. The adjusted "post-2011 Flood" frequency-damage curve has been included on Figure 8.87. The extent of future damages were, therefore, expected to be considerably less than the damages would have been in 2011.

However, as discussed in Section 8.6.2.2, the number of stakeholders who participated in the programs to undertake mitigation measures was less than originally anticipated. A sensitivity analysis was, therefore, conducted to determine how this would have affected the resultant average annual damages and is discussed in Section 8.6.2.8.

8.6.2.5 Damages to Public Infrastructure and Utilities

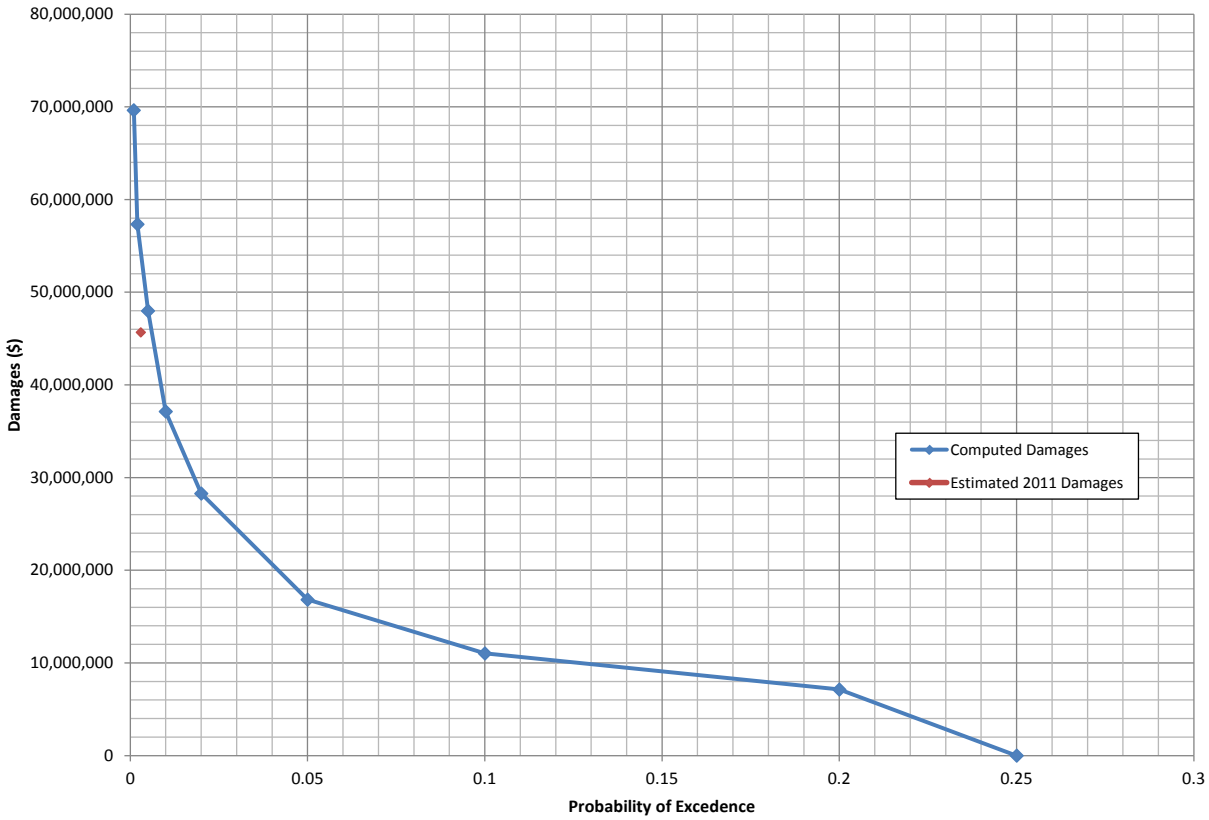
Since the largest portion of damage costs to public infrastructure and utilities are typically road, ditch and water crossing repairs, damages for this category were based on the total length of roads located within the flooded area. A relationship of damage costs vs length of flooded road was developed based on the estimated 2011 damages for public infrastructure and utilities. The total length of flooded road during the 2011 Flood was estimated from the damage model. The adopted relationship for this study is shown on Figure 8.88.

Figure 8.88
Relationship for Damages to Public Infrastructure and Utilities for Lake Manitoba



The frequency-damage curve for public infrastructure and utilities for Lake Manitoba is provided in Figure 8.89. As shown on the figure, the frequency-damage curve intercepts the estimated 2011 Flood damages for public infrastructure and utilities at a value of $45.6 M with an annual probability of being equaled or exceeded of approximately 0.003. This demonstrates that the flood damage model provides a reasonable estimate of these types of damages around the lake.

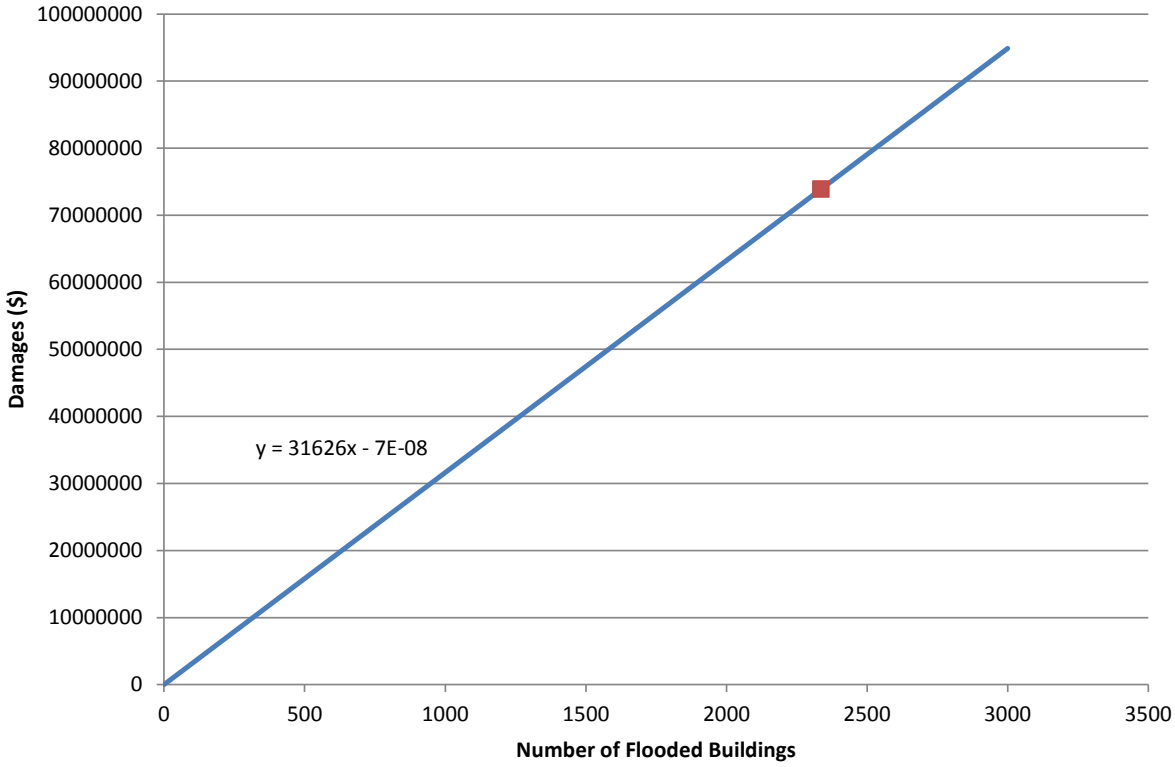
Figure 8.89
Computed Damages to Public Infrastructure and Utilities for Lake Manitoba



8.6.2.6 Flood Response and Fighting Costs

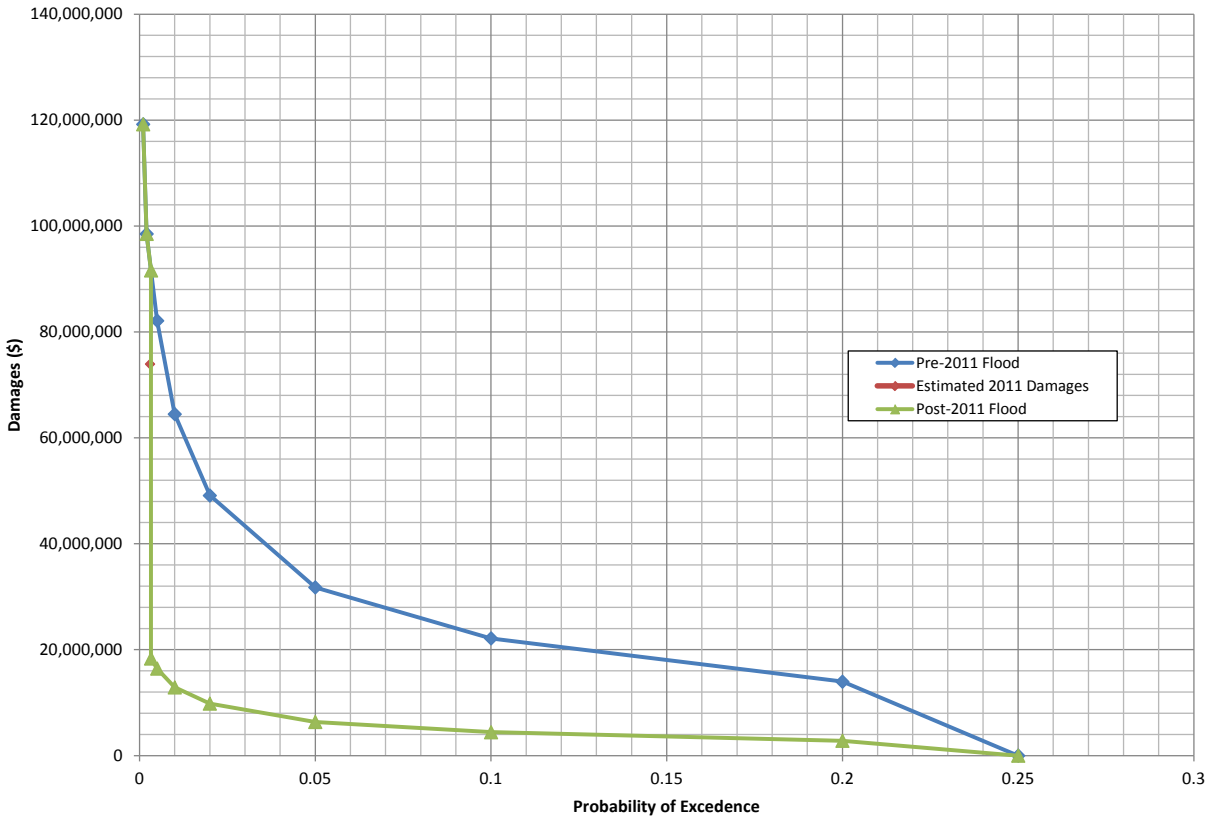
Since the largest portion of flood response and fighting costs are associated with efforts to protect buildings and structures from potential flooding, damages for this category were based on the total number of structures affected by flooding. A relationship of flood response and fighting costs vs number of structures was developed based on the estimated 2011 Flood response and fighting costs. The total number of structures affected by flooding was estimated from the damage model. This relationship is shown on Figure 8.90.

Figure 8.90
Relationship for Flood Response and Fighting Costs for Lake Manitoba



The frequency-damage curve derived for flood response and fighting costs for Lake Manitoba is provided in Figure 8.91. As shown on the figure, the damage-frequency curve approximates the best estimate of the 2011 Flood response and fighting costs at a value of $73.9 M with an annual probability of being equaled or exceeded of approximately 0.003. This demonstrates that the flood damage model provides a reasonably representative estimate of these types of damages around the lake.

Figure 8.91
Computed Flood Response and Fighting Costs for Lake Manitoba



As discussed in Section 8.6.2.2, flood mitigation programs initiated by the Province of Manitoba were anticipated to result in the majority of buildings and structures being protected against a repeat of a 2011 Flood-type event. As such, the base condition for this study considered that flood response and fighting costs would be much less during future 2011-type floods, as well as for smaller events. The adjusted “post-2011 Flood” frequency-damage curve, which forms the base condition for this study has been included on Figure 8.91. The extent of future flood response and fighting costs were, therefore, expected to be considerably less than the damages would have been in 2011.

However, as discussed in Section 8.6.2.2, the number of stakeholders who participated in the programs to undertake mitigation measures was less than originally anticipated. A sensitivity analysis was, therefore, conducted to determine how this would have affected the resultant average annual damages and is discussed in Section 8.6.2.8.

8.6.2.7 Business Losses, Stress and Anxiety, and Other Miscellaneous Damages

Due to complexities in accurately defining the number of business affected around Lake Manitoba during a flood and the extent of damages they incurred, business losses were estimated based on a percentage of the sub-total of all other damages computed for the lake. As shown in Table 8.20 of Section 8.6.2.2, business losses around Lake Manitoba during the 2011 Flood event were estimated to equal approximately 1% of the total damages. Due to uncertainties with the data and whether all business losses were truly accounted for, a conservative value of 5% was adopted for this study.

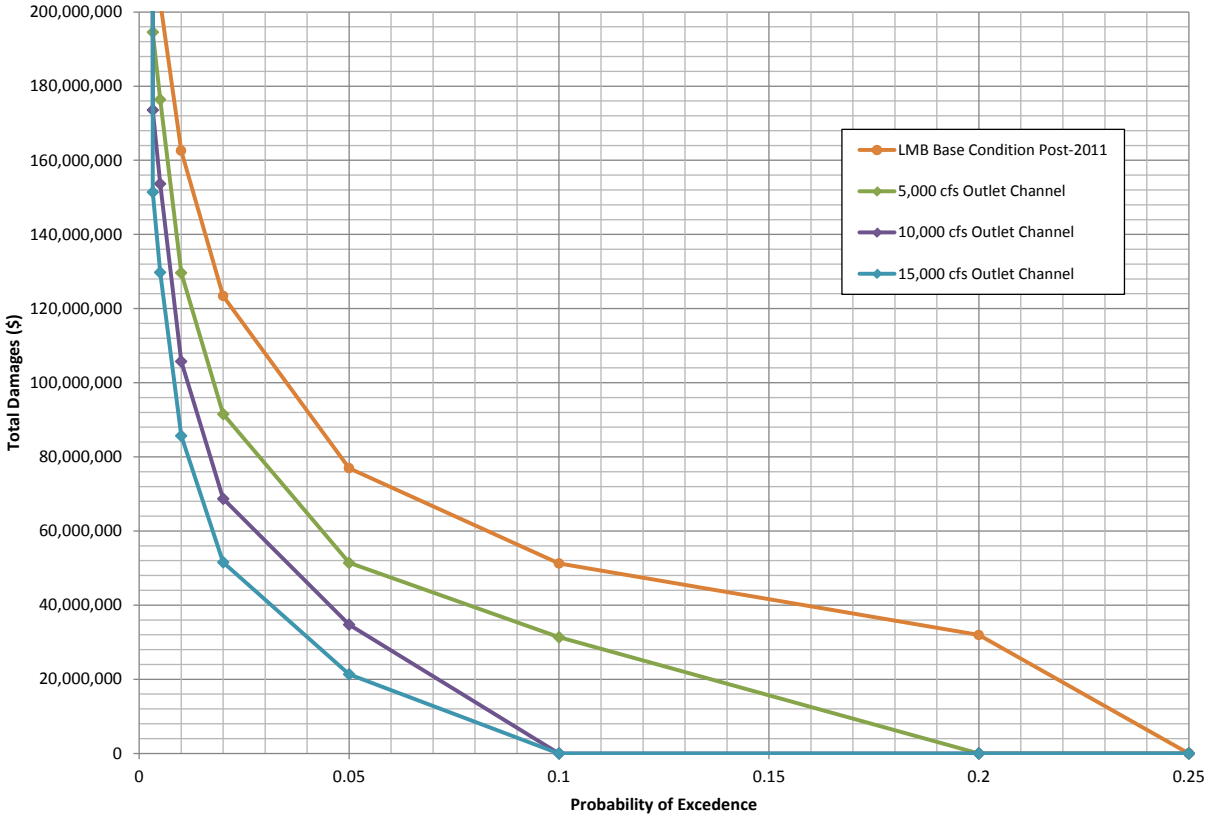
Also, as discussed in Section 7.2 of the report, due to challenges in quantifying stress and anxiety into a stage-damage model, this uncertain type of flood damage was accounted for with a percentage allowance. Similar to the Flood Protection Studies for Winnipeg (KGS Group, 2000b), an additional allowance equivalent to 20% of the building and agricultural damage estimates was included to account for stress and anxiety.

Based on discussions with a representative cross section of stakeholders and government officials during project meetings for this study, it was recognized that the level of compensation and damages for the 2011 Flood discussed in Section 8.6.2.2 underestimated the true cost incurred by the stakeholders around the lake. To account for these unidentified additional damages, KGS Group applied a 25% increase to the total damages estimated using their best judgment and the approaches described above.

8.6.2.8 Final Damage-Frequency Relationship

The final damage-frequency relationship computed for Lake Manitoba is provided on Figure 8.92. This curve represents the baseline conditions for this study, based on the various assumptions discussed in this section of the report. Also included are examples of how the curve changes if flood mitigation alternatives evaluated for the study were implemented. Results for different outlet channel alternatives, based on the reduction capabilities later defined in Section 15.0, have been shown on Figure 8.92.

Figure 8.92
Total Computed Damages for Lake Manitoba



Based on the curves presented in Figure 8.92, average annual damages were calculated for the base condition. Average annual damages were also calculated for different Lake Manitoba outlet channel capacities as an example of how the results changed if the flood mitigation alternatives evaluated for the study were implemented. Table 8.22 summarizes the results of this assessment.

Table 8.22
Estimated Average Annual Damages on Lake Manitoba

Description	Estimated Average Annual Damages ($ / year)
Base Condition	15,764,000
5,000 cfs Outlet Channel	9,538,000
10,000 cfs Outlet Channel	5,586,000
15,000 cfs Outlet Channel	4,305,000

As discussed in Section 8.6.2.2, the base assumption of the analysis assumed that the number of stakeholders who participated in the programs to undertake mitigation measures was less than originally anticipated. A sensitivity analysis was, therefore, conducted to determine how this would have affected the resultant average annual damages. The results of the analysis showed that the estimated average annual damages for the base condition would have increased from approximately $15.8 million per year to approximately $25.5 million per year. However, the estimated average annual damages for conditions with an outlet channel would have also increased proportionally from approximately $9.6 million per year to approximately $15.0 million per year. The resultant average annual benefits, therefore, would have increased from approximately $6 million per year to $10.5 million per year.

As later discussed in Section 22.0 of the report, the results of the economic analyses showed that none the alternatives investigated were calculated to have benefit / cost (B/C) ratios exceeding a value of 1.0. For the Lake Manitoba outlet channel options, the 142 cm^3/s (5,000 cfs) capacity option had the highest B/C ratio at 0.5. With an increase to the average annual damages as discussed above, the resultant B/C ratio for the 142 cm^3/s (5,000 cfs) Lake Manitoba outlet channel option increased to approximately 0.9. On this basis, even though the base assumption regarding the number of stakeholders who participated in the programs to undertake mitigation measures resulted in an underestimation of the Lake Manitoba damages, this did not have an impact on the overall conclusions of the study, as the B/C ratio for the options would remain below 1.0. Further details on the conclusions of the study are provided in Section 26.0 of the report.

8.6.3 City of Winnipeg and Other Interests along the Red River

There are five key areas where changes in Assiniboine River flood flows would affect flood conditions and protection measures along the Red River. They would impact flood damage assessments of options to improve flood protection in the Assiniboine River Valley and are as follows:

1. Impacts on the flood preparation efforts expended by the City of Winnipeg staff and their contractors. The level of effort (and cost) of this work is commensurate with the magnitude of the flood and the water levels anticipated in Winnipeg.

2. If the operation of the Red River Floodway (Floodway) is within the limits of Rule 1 (i.e., control of the water level at the Floodway Inlet to the state of nature water level), then changes in flood flows due to mitigation works on the Assiniboine River would theoretically have no effect upstream of the Floodway Inlet Control Structure. However, it is recognized that significant costs are incurred to implement pre-emptive action as the water level rises towards the limit of 0.62 m (2 ft) above the peak recorded in the 1997 flood. Once a flood is of sufficient magnitude that operation is required to extend the water levels into the range of either of Rules 2 or 3, then variations in inflow from the Assiniboine River would have an effect on potential flood damages in southern Manitoba upstream of the Floodway.

3. Once the operation of the Floodway approaches or reaches the limits of Rule 2, and Rule 3 must be invoked, overtopping of dykes in Winnipeg would commence and overland flooding within the City would occur. The damages from such flooding are potentially enormous, and reduction of any magnitude from the Assiniboine River would be of significant value. There would also potentially be benefits from changes in Assiniboine River flows in Winnipeg that would downgrade a potential operation in Rule 2 back into the range where only Rule 1 would be needed. Conversely, any changes in the Assiniboine River drainage basin that would potentially increase flood flows into Winnipeg could adversely affect flood effects and damages / costs along the Red River.

4. Increases in flood flows along the Assiniboine River in the west end of Winnipeg could cause increases in flood damages over and above those that would normally be predictable under the first point above. The water levels in the upstream reaches of the Assiniboine River within Winnipeg are not strongly related to the water level on the Red River, due to the nature of the backwater conditions that are influenced mostly by the river geometry and slope in that zone.

5. Changes in flood flows downstream of Winnipeg would have the potential to affect stakeholders north of the City of Winnipeg, principally in the area of Selkirk. However, KGS Group is unaware of any comprehensive data on potential flood damages in this area as a function of Red River flow or stage. Furthermore, there are two important factors that have led to excluding these damages from the overall analysis:

    a) Some of the flood potential is due to the ever-present risk of ice jams in the early stage of the flood hydrograph. Precise prediction of the benefits of reduction in flow at this time is complex and to a large extent unpredictable.

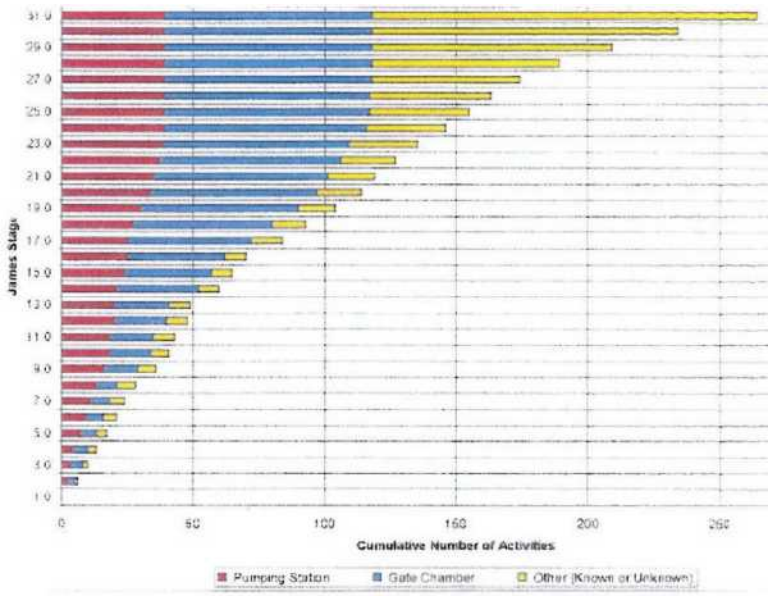
    b) Damages to properties, although serious on a scale that would be important to the local stakeholders, are not anticipated to be large in comparison with the primary benefits being considered in and upstream of Winnipeg. It has been recognized that this exclusion will result in a small degree of underestimation of the impacts on Red River interests.

The first three issues introduced above are discussed and described in more detail in the following subsections.

8.6.3.1 Impacts on City of Winnipeg Flood Protection Efforts

The efforts needed to protect Winnipeg from rising flood waters were aptly described in the report by the Red River Floodway Operation Review Committee (RRFORC, 2001). It demonstrated conceptually how the efforts required by the City increased dramatically as a function of river stage. River stage is reported in Figure 8.93 relative to the James Avenue Pump Station Datum (JAPSD) that has a zero elevation at geodetic El 221.76 m (El 727.57 ft). Since that report was submitted, significant efforts were expended to develop a computerized model called the "Flood Manual", which identified the proactive activities that would be required by the City of Winnipeg prior to and during a flood event

Figure 8.93
Number of Flood Activities vs. the River Stage at the James Avenue Pumping Station



Quantification of the costs of these activities and how they would be affected by increases or decreases in flood flows in Winnipeg was based on information provided by the City of Winnipeg. That information was provided to KGS Group in 2009 in connection with benefit / cost analyses of the possible expansion of the West Gap in the East Embankment of the Red River Floodway. The analysis was carried out in 2009 and is documented in memoranda prepared by KGS Group for Manitoba Water Stewardship (KGS Group, 2009). Table 8.23 lists a summary of the information provided by the City of Winnipeg and used for this study.

Table 8.23
Summary of Costs of Flood Protection Activities as Provided by the City of Winnipeg
(in Thousands of Dollars)

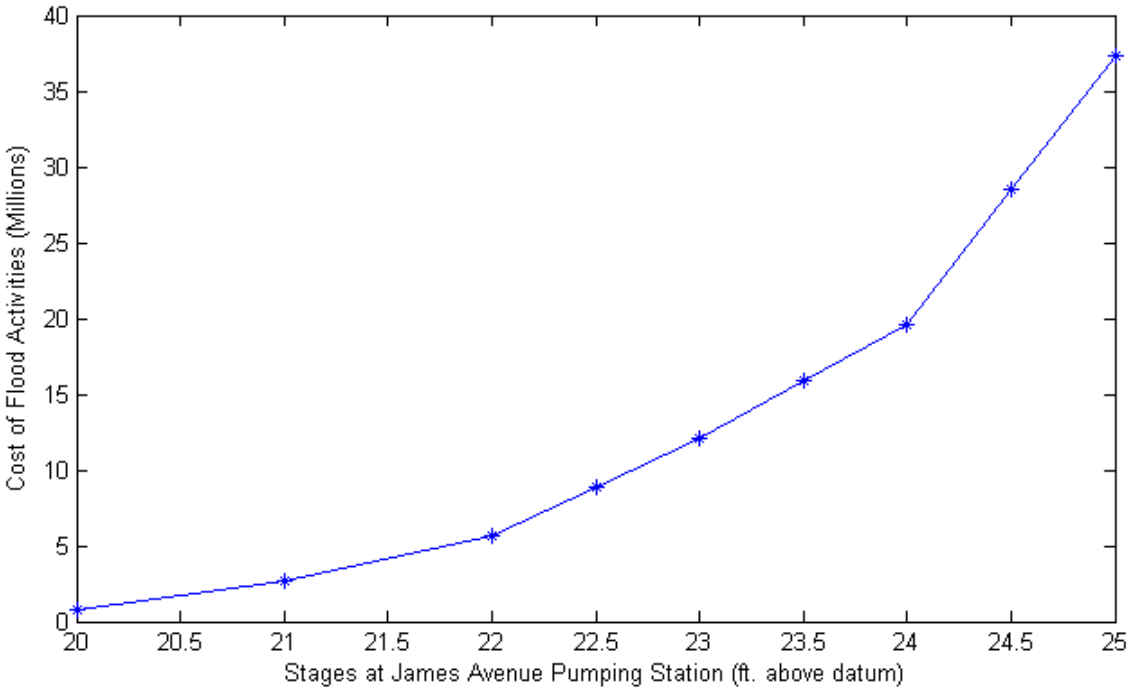
Water Level at James Avenue (ft) (JAPSD)	25	24	23	22	21	20	19	18	17
Sewer System Control by WWD	772	703	627	520	445	389	312	224	0
Other Costs by WWD	6,920	5000	3,080	1,706	878	50	0	0	0
Sandbag Costs by Public Works	26,235	12,120	7,270	2,866	1,085	220	0	0	0
Total Costs (2009$)	33,927	17,823	10,977	5,092	2,408	660	312	224	0
Total Costs Inflated to 2014$	37,320	19,605	12,075	5,600	2,649	726	343	246	0

Note: WWD denotes Water and Waste Department of the City of Winnipeg.

The last line of Table 8.23 indicates the costs used in the current study assuming an inflation factor of 2% per year from 2009 to 2014 based on Consumer Price Index data from Statistics Canada.

Figure 8.94 shows the data developed from Table 8.23 as a graph.

Figure 8.94
Cost of Flood Protection Activities in the City of Winnipeg
as a Function of River Stage
(to the James Avenue Pump Station Datum)



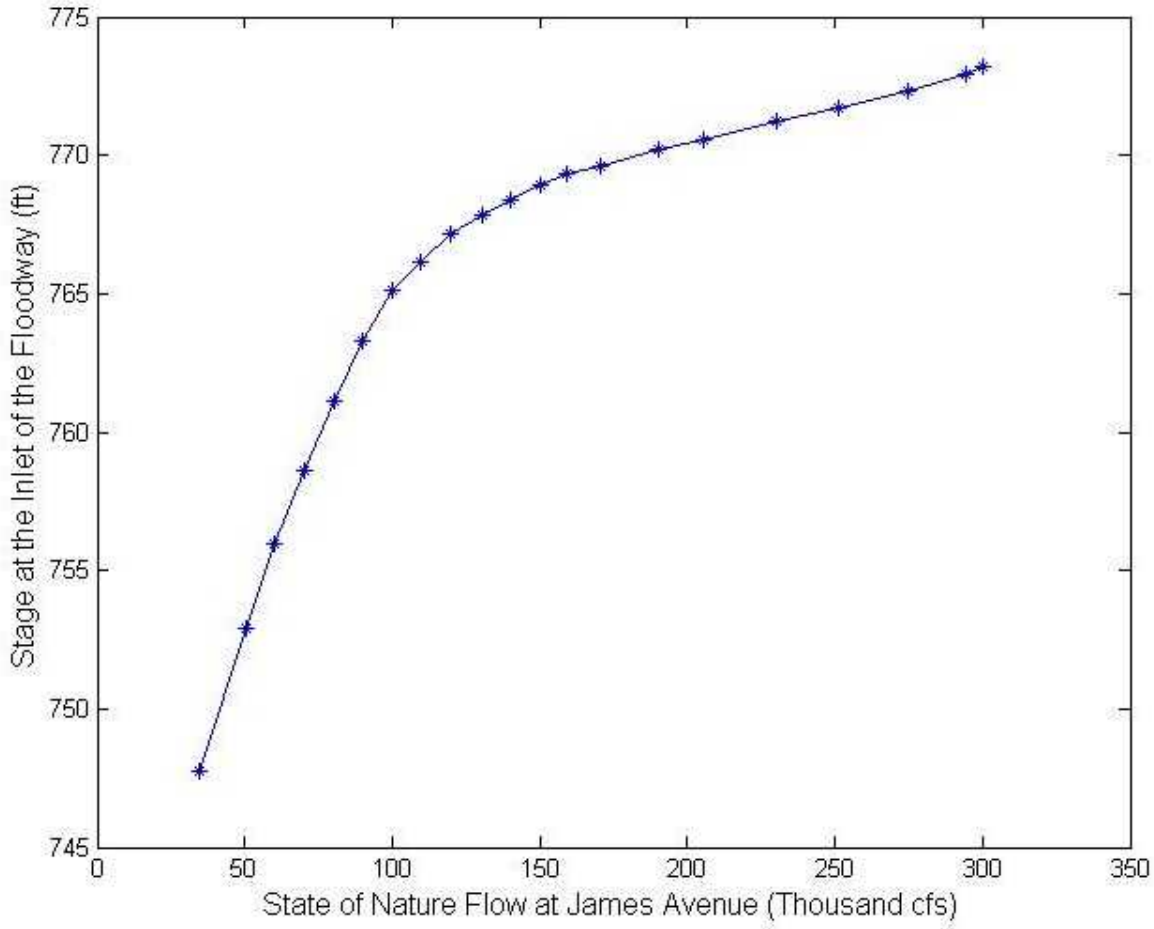
8.6.3.2 Flood Damages Upstream of Winnipeg

As stated in the introduction to this section, if operation of the Floodway is within the limits of Rule 1, changes in flood flows in the Assiniboine River would have virtually no impact upstream of the Floodway. Operation at the Floodway Inlet would continue to strive to control the water level at that location at the level it would have been in the "state of nature".

On the other hand, once the limits of Rule 1 have been reached and the water level in Winnipeg threatens to exceed El 24.5 ft (JAPSD), then operation at the Floodway Inlet would commence to follow Rule 2, and ultimately, Rule 3. Explanation of the details of these rules is beyond the scope of this study, and the reader is directed to the 1999 report by the Red River Floodway Operation Review Committee (RRFORC, 1999). The definition of the state of nature rating curve for the Red River at the Floodway Inlet has been based for the purposes of this study on the findings reported by Acres Manitoba Ltd. (Acres, 2004), and is also summarized in

Figure 8.95. Figure 8.95 is a simplified version of a family of curves that would indicate the effect of variations in the coincident flows in the Assiniboine River.

Figure 8.95
State of Nature Rating Curve of the Red River at the Floodway Inlet
(Acres, 2004)



Notes: Flows can be converted to m^3/s by dividing imperial values by 35.3.
Water levels can be converted to metres by multiplying imperial values by 0.3048.

However, that family of curves is tightly clustered around the single curve shown in Figure 8.95. Examples are based on findings reported by Acres Manitoba Limited (Acres, 2004) in Table 4-7:

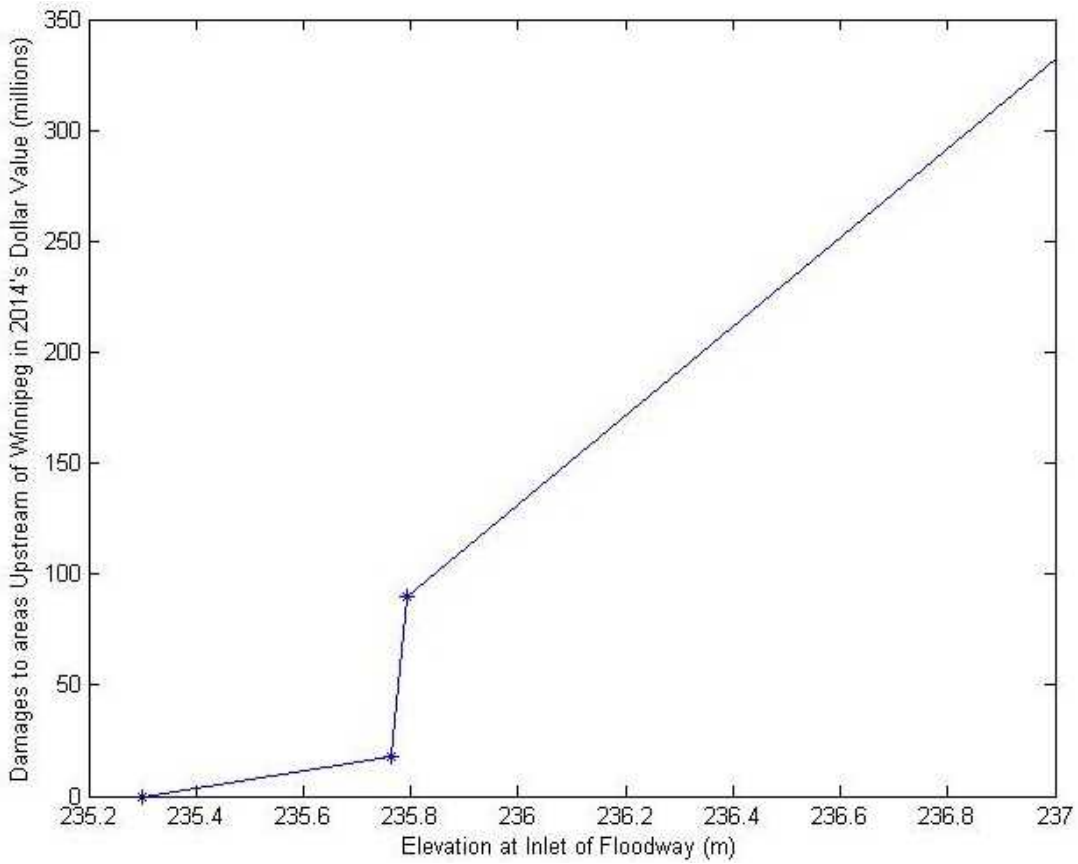
- At a state of nature flow at the Floodway Inlet of 150,000 cfs (4,250 m^3/s), the family of curves that reflect the variation in coincident flow in the Assiniboine River are all within +/-12 cm (5 inches) of the curve shown in Figure 8.95. However, at this relatively low flow, the extent of flooding upstream of the Floodway is relatively small, regardless of the water level within that range of curves.

- At a state of nature flow at the Floodway Inlet exceeding 200,000 cfs (5,660 m^3/s), the family of curves fall within approximately +/-7.5 cm (3 in) of the curve shown in Figure 8.95. This is the most important range where flood damages need to be estimated.

Considering the insensitivity demonstrated by the facts listed above, it was considered that the single curve shown in Figure 8.95 is sufficiently accurate in this planning study to justify ignoring the complications due to variations in the proportion of Assiniboine River flows relative to the Red River. This is particularly true in the context of the AR&LMB Study where estimation of absolute water levels is not of paramount importance, and only the relative differences in water levels among the alternatives being compared have importance.

The incipient point of transitioning from Rule 1 into Rule 2 is dependent on the flood characteristics and the relative contribution of flood flow from each of the Red and Assiniboine Rivers. However, it can be quantified as the point at which the water level at the Floodway Inlet reaches approximately El 235 m (El 771 ft). Studies by KGS Group for the International Joint Commission in 1999 (KGS Group, 2000b) developed estimates of potential flood damages in the Red River Valley as a function of the water level at the Floodway Inlet. Since that time, almost all residences and commercial establishments in the Red River Valley have been protected up to the criterion of the peak water level that occurred in 1997, plus 0.6 m (2 ft). The resulting relationship is shown in Figure 8.96. This figure represents conditions almost 15 years ago, with an added correction for inflation during that period. It has also been increased by 35% to recognize an annual growth rate in the area of approximately 2% per year. This would recognize the growth in population, as well as development of new infrastructure and expansion of economic activities known to have occurred in zones such as Grande Pointe. In lieu of definitive data on the growth in the area, the growth factor of 35% has been based on judgment of what might be reasonably expected.

Figure 8.96
Potential Flood Damages Upstream of the Floodway Inlet

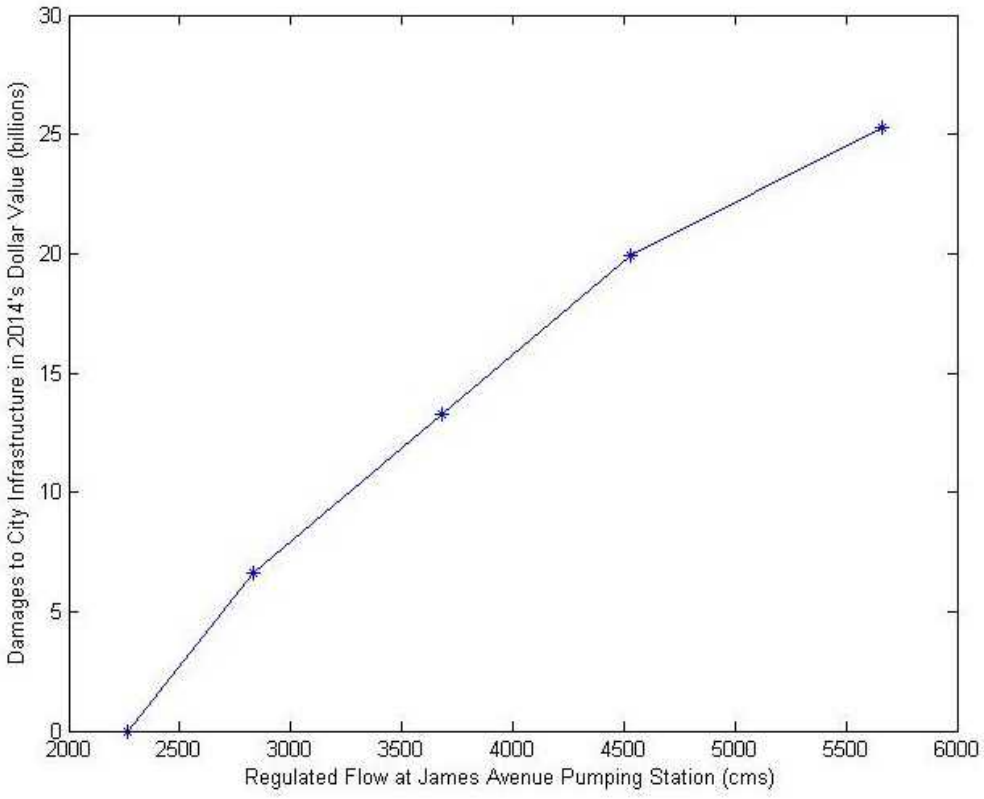


Note: Water levels can be converted to feet by multiplying metric values by 0.3048.

8.6.3.3 Flood Damages in Winnipeg

Once flood flows reach the upper end of operation in Rule 2 for the Floodway (approximately a 1:700 year flood condition), and flows within Winnipeg exceed approximately 2,090 m^3/s (74,000 cfs), there is a high risk of overland flooding in Winnipeg, as well as flooding through the sewer system. This would be approximately equivalent to a water level at El 26.5 ft (JAPSD), and assumes that the City of Winnipeg would be successful in raising the levels of the primary dikes to protect against such a high water level. This was examined by KGS Group in a study for the International Joint Commission in 1999 (KGS Group, 2000b). Flood damages were estimated for that study using a detailed GIS-based computer model using property tax data supplied for that purpose by the City of Winnipeg. The basic relationship derived is shown in Figure 8.97, with corrections for inflation from 1999 to 2014.

Figure 8.97
Flood Damages in Winnipeg

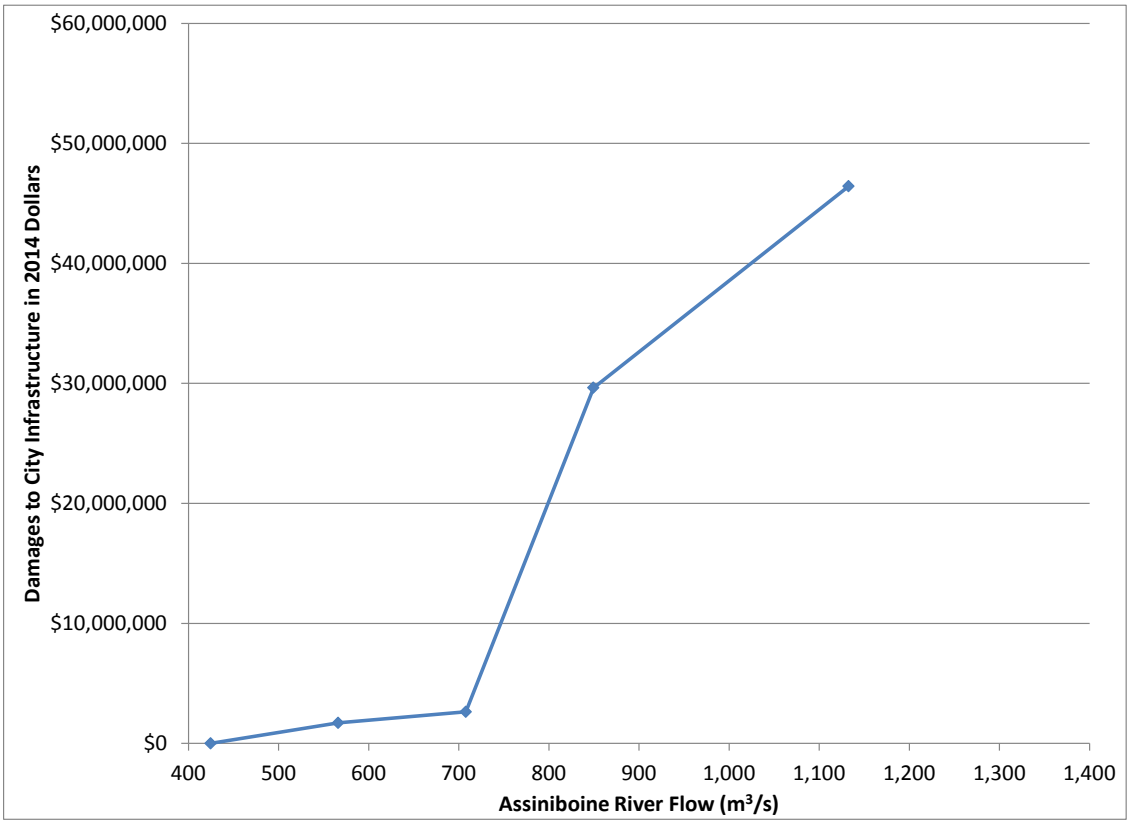


Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Along the Assiniboine River in the west end of Winnipeg, the water levels are not strongly related to the water level on the Red River due to the nature of the backwater conditions that are influenced mostly by the river geometry and slope in that zone. Flood damages in this area, assumed to extend from the City of Winnipeg Route 90 Bridge on the Assiniboine River up to Headingley, were calculated based on an estimate of the number of structures located within the Assiniboine River flood zone, and a rate of damages per structure from the KGS Group study for the International Joint Commission in 1999 (KGS Group, 2000b) with corrections for inflation from 1999 to 2014. The number of structures was estimated from flooded area maps (attached in Appendix J), developed for a range of flows based on the water levels obtained from the Assiniboine River backwater model discussed in Section 8.3.1, plus 1.8 m (6 ft) for potential basement flooding. For flows of 708 m^3/s (25,000 cfs) or less, it was assumed that temporary measures implemented by the City of Winnipeg would successfully mitigate flood damages most of the time. Nevertheless, a conservative allowance of 10% of the damages was

incorporated into the damage model for this range in flows. The discharge-damage curve derived for the Assiniboine River in the west end of Winnipeg is shown on Figure 8.98.

Figure 8.98
Flood Damages on the Assiniboine River in West End of Winnipeg



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

8.6.3.4 Hydraulic Conditions in Winnipeg

The estimation of the costs of pre-emptive actions by the City of Winnipeg described in Section 8.6.3.1 is linked to the water level of the Red River at the James Avenue Pump Station. The hydraulic information incorporated into the City of Winnipeg's Flood Manual (Acres, 2004) (specifically the data embedded in the Hydraulic Calculator used by the Flood Manual) has been adopted as the basis for estimating water levels at this location. That procedure and the data on water levels upon which it is based is believed to be the most accurate means currently available for estimating flood stages in Winnipeg. The reader is referred to the User Manual for

the Hydraulic Profile Calculator ((KGS Group and UMA, 2003) for details on the methods and data used in that database.

8.6.3.5 Modeling of Damages

The estimation of the probability distribution(s) of the annual flood damages in the Red River and Assiniboine River within Winnipeg has been based on stochastic generation of a large number of synthetic flood events. This so-called Monte Carlo procedure generates a vast number of synthetic flood events that can be statistically analysed to represent the overall probability distribution of the existing (or base) case, as well as to represent the impacts from alternative schemes for flood mitigation along the Assiniboine River. The important aspect of this procedure is that the probability distributions of the annual peaks of the flood hydrographs must be representative of the hydrologic and statistical characteristics of the two rivers. This prerequisite has been the focus of considerable engineering effort for this study and is described in the Sections 8.5.3.

8.6.3.6 Platform for Damage Model

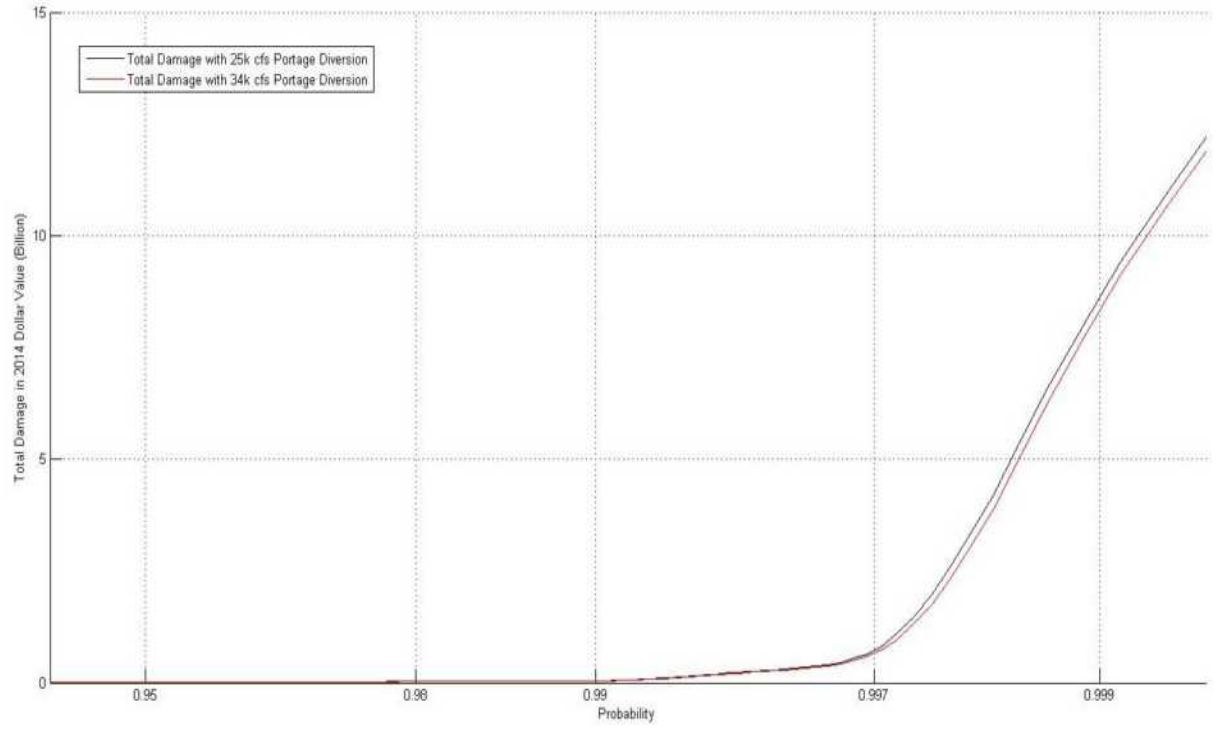
The individual steps described above in estimating damages in the Red River and the Assiniboine River within Winnipeg have been formulated mathematically in the software "MATLAB". This platform provides a basis for generation of a large number of flood events and the damages that would be associated with those events. The objective of the MATLAB model is to estimate the probability distribution of the flood damages for any particular scenario, whether it be for the base case (existing conditions), or for hypothetical flood protection schemes that would be compared to the base case.

8.6.3.7 Results

The MATLAB model described above was applied to compare the average annual damages of the base case that represents current conditions in Winnipeg and in the Assiniboine River to the different flood mitigation scenarios discussed in this report. The results for the base case, as well as for an example flood mitigation scenario consisting of increasing the discharge capacity of the Portage Diversion to 963 m^3/s (34,000 cfs), have been summarized in the Damage

Frequency Relationships shown in Figure 8.99. Results of the economic analysis for all options are discussed in Section 22.0 of this report.

Figure 8.99
Damage Frequency Relationships with (red line)
and without (black line) Expansion of the Portage Diversion



The average annual damages for the base case can be derived in the traditional way as the area under the damage-frequency curve. This has been estimated to be $22.67 million. Similarly, the lower frequency curve with the improved discharge capacity of the Portage Diversion has been estimated to result in average annual damages of $21.73 million. The difference between these values represents an average annual benefit of $0.94 million per year. This benefit is believed to approximate the impacts along the Red River and in the Assiniboine River within the limits of the City of Winnipeg. However, it does not include the more direct benefits that would be derived in the Assiniboine River upstream of the city limits of Winnipeg. That component of the benefits was discussed in Section 8.6.1.

To further clarify the intermediate results derived during analyses such as those described above, three “snapshots” of the results were extracted to show the conditions and damages

computed by the model in the analysis. These samples have been composed for three flood magnitudes:

1. The first where operation of the Floodway is clearly within the range of Rule 1.
2. An intermediate level of flood where operation is within Rule 2.
3. A third extreme case where operation would be forced into the range where Rule 3 would apply.

These three cases are summarized in Table 8.24 below.

Table 8.24
Sample Intermediate Calculations of Water Levels and Damages within the City of Winnipeg and along the Red River

Location	Rule 1 Event	Rule 2 Event	Rule 3 Event
Red River State of Nature Flow (m^3/s)	500	4000	5500
Assiniboine River Flow at Holland (m^3/s)	100	1400	2500
Winnipeg Local Inflow (m^3/s)	28.3	28.3	28.3
State of Nature Flow at James Avenue (m^3/s)	628	5428	8028
Flow in Portage Diversion (m^3/s)	0	708	708
Flow in Winnipeg Floodway (m^3/s)	0	2126	3917
Water Level at Floodway Inlet (m)	227.5	235.0	237.1
Over Flow From Assiniboine River	0	78	440
Regulated Flow at James Avenue (m^3/s)	628	2517	2963
Water Level at James Avenue (m)	224.7	229.8	231.3
Water Level at James Avenue (JAPSD ft)	9.6	26.5	31.28
Water Level in Red River at Floodway Outlet (m)	220.2	226.2	229.6
Cost of Pre-emptive Activities (2014 million $)	0	37.3	37.3
Damages to City Infrastructure (2014 million $)	0	0	7506
Damages to Area Upstream of Floodway (2014 million $)	0	0	350.1
Damages to West End of Winnipeg (2014 million $)	0	1	3.8
Total Estimated Costs/Damages (2014 million $)	0	38.3	7897.2

Notes: Flows can be converted to cfs by multiplying metric values by 35.3.
Water levels can be converted to feet by dividing metric values by 0.3048.

8.6.4 Dauphin Lake

The study considered several flood mitigation alternatives for Dauphin Lake. To evaluate the benefits of the options on Dauphin Lake, a damage model was developed for the lake and is described below

There are many physical similarities between Dauphin Lake and Lake Manitoba including topography and land use. For example, both lakes are widely used for recreational purposes and are surrounded by several communities and other developed areas that support a mix of permanent residences and seasonal cottagers. Agricultural land also surrounds both lakes up to the shoreline in many locations. As well, though Dauphin Lake's watershed and surface area are several times smaller than those of Lake Manitoba, there are many similarities between flood events on both lakes. For example, large flood events on both lakes often occur over several months and can extend well into the summer. Also, both lakes are most impacted by severe wind events from the south-east direction.

As discussed in Section 8.6.2, an integrated Excel and GIS based damage model was developed for Lake Manitoba. Due to the similarities between Dauphin Lake and Lake Manitoba outlined above, results from the Lake Manitoba model were used and applied to Dauphin Lake. The rate of damages around Lake Manitoba ranged between $3,300/ha and $10,100/ha depending on the magnitude of the flood event. To maintain a conservative representation of damages in the economic analysis, the higher value of $10,100/ha was adopted for Dauphin Lake.

Using this assumed value of damage, a frequency-damage curve was developed for Dauphin Lake by incorporating the following assumptions:

- The point of incipient damages occurs at EL 260.5 m (El 858 ft), which is equivalent to the flood stage on Dauphin Lake defined by MIT and has a flooded area that is negligible.
- Approximately 35 ha (86 ac) of land would be flooded at El 262.49 m (El 861.2 ft) based on a flood forecast map of Dauphin Lake from 2011.
- The flooded area for other water surface elevations on Dauphin Lake can be determined by interpolating between the values identified above.

- The baseline frequency curve developed for Dauphin Lake and documented in Section 13.0 (Figure 13.16) of this report can be used to define the return period of different Dauphin Lake water levels.

The resultant damage-frequency relationship for Dauphin Lake is shown on Figure 8.100. Also shown on the figure is an example of the resultant damage-frequency relationship if a 30 m (100 ft) wide weir outlet channel (discussed in Section 13.0) was implemented. Estimated average annual damages with and without the flood mitigation options presented in this report are summarized on Table 8.25.

Figure 8.100
Damage-Frequency Relationships for Dauphin Lake

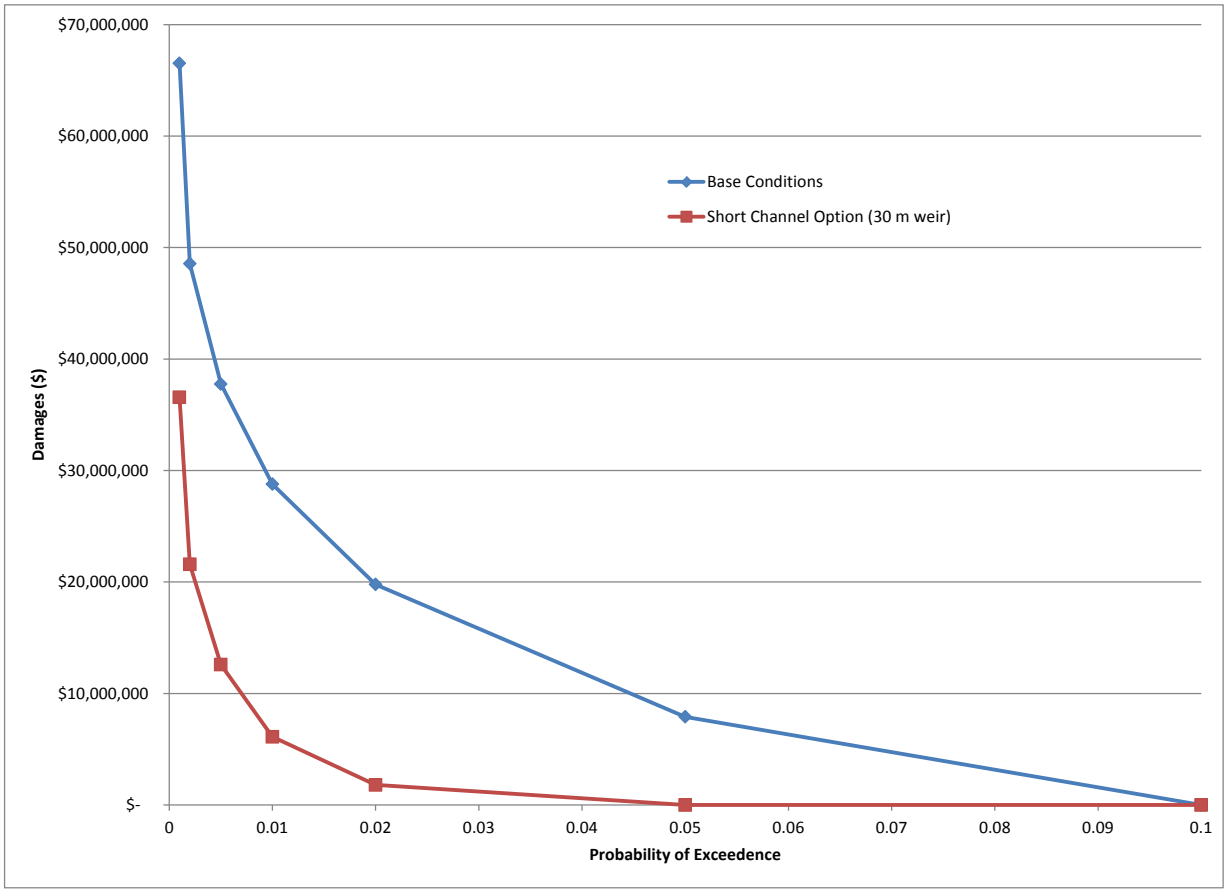


Table 8.25
Estimated Average Annual Damages on Dauphin Lake

Description	Estimated Average Annual Damages ($ / Year)
Base Condition	$1,276,000
30 m Wide Weir Outlet Channel	$230,000
Excavate Mossey River 0.3 m	$704,000
Excavate Mossey River 0.5 m	$439,000

8.6.5 Upper Assiniboine River Valley

The Assiniboine River Valley from Shellmouth to Brandon contains 16,200 ha (40,000 ac) of fertile farmland, which is used for agricultural production. The portion from the Shellmouth Dam to St. Lazare is used primarily for hay production whereas the portion from St. Lazare to Brandon is used mostly for grain and specialty crops.

Agricultural production has been frequently impeded by flooding of the valley due to excessive runoff from Saskatchewan and Manitoba, which has often exceeded the capacity of the Shellmouth Reservoir to control Assiniboine River flows downstream of the dam. Spring flooding (March to mid-May) has been fairly common throughout, but late spring and summer flooding greatly impeded agricultural production during the 1950s and from 2010 to 2013.

Options to prevent or reduce flood damages on the Assiniboine River downstream of the Shellmouth Dam have been considered for the current study and are documented in Section 9.0 of this report. The damages that constitute the potential benefits of these options are discussed below.

8.6.5.1 Agricultural Damages

Agricultural flood damages result primarily from valley farmlands being inundated by overbank river flows. The extent of the damages is a function of the maximum river discharge and the date at which flooding ceases. The bank full capacity of the river varies from one location to another, even within smaller portions (reaches) of the river such as from St. Lazare to Miniota. As a result, potential damages begin when the river flows rise close to the estimated bank full

capacity at given locations, and increase to the river level where all the lands in the river reach are flooded. Damages also increase with increased river flow since larger overbank flows usually have a longer duration of flooding, which exacerbates damage.

The date at which flooding ceases also has an impact on the extent of damages. Late flooding may impede or prevent seeding of crops, may drown crops which have already emerged, or may prevent harvesting of crops when flooding occurs later in the year. After the river falls below bank full capacity, time is needed for the water to drain off the fields and for the lands to dry before seeding can begin. This typically takes about two weeks, but it varies depending on weather conditions. The date that overbank flows cease appears to be the most important factor affecting losses in agricultural productivity, even more important than the peak water flows and levels. To account for these variable factors, discharge-date-damage relationships were developed for estimating losses in agricultural productivity along the Upper Assiniboine River Valley.

8.6.5.2 Discharge-Date-Damage Relationships

The percentage of farmlands affected by overbank flows for different river reaches under various flow conditions was estimated. Detailed simulations of flooded areas were not available, so these estimates were made based on average bank full flow capacities for each river reach as provided on MIT flood sheets. The average bank full capacities define when overbank flows begin in a river reach, but considerable variability is known to occur within each reach depending on location. The percentage of the valley flooded for a given discharge in each reach of the river was estimated based on the average bank full capacity and based on information from landowners on the flow magnitudes, which flooded the entire valley. Most of this information was obtained from Assiniboine Valley Producers via interaction during flood events since 1990.

The date that overbank flows end, incremented by two weeks for draining and drying, affects decisions on what crops are grown, since producers must consider what crop types still have a reasonable chance of maturing in a shortened growing season. This decision also involves consideration of crop insurance deadlines, in terms of the latest seeding dates, as defined by Manitoba Agricultural Services Corporation (MASC). The relationships between the last date of

overbank flows, the type of crop planted (if any) and the reduced yield due to the shortened growing season was defined for each crop type based on data from MASC and on information from an experienced producer in the Assiniboine River Valley.

Typical yields and unit prices of crops determined based on data from MASC and from readily available data on Manitoba commodity prices in recent years were then used to compute the dollar value of crops under normal conditions without flooding and for conditions based on restricted crop types and reduced yields due to late (or no) planting. The resultant discharge-date-damage relationships (in 2014$) for six different reaches along the Upper Assiniboine River are provided in Appendix D.

The results of this analysis show that agricultural damages in the Assiniboine River Valley can reach (in 2014$) up to $2.8 million from Shellmouth Dam to St. Lazare and up to $15.2 million from St. Lazare to Grand Valley, for years when all the farmlands are flooded up to mid-June or later.

8.6.5.3 Frequency-Damage Relationships

Based on the discharge-date-damage relationships discussed in Section 8.6.5.2 that provided the maximum amount of damages (escalated to 2014$), worst case frequency-damage relationships representative of conditions for the entire Assiniboine River Valley were developed for the reaches of the Shellmouth Dam to St. Lazare and from St. Lazare to Brandon. This assumed that flooding ceased to occur on June 14 or later; therefore, seeding would not occur, regardless of the duration of the flood. The worst case condition was selected to ensure a conservatively high estimate of the damages for the economic analyses later discussed in Section 22.0 of this report.

To account for other types of damages to farmland caused by floods, such as erosion, gullying, debris etc., a 50% increase was applied to the total damages described above. This value was selected to be consistent with the methods applied in other areas of the study. As well, an additional allowance equivalent to 20% of the damage estimates was included to account for stress and anxiety, as discussed in Section 7.2 of the report. The resultant frequency-damage curves are provided on Figures 8.101 and 8.102.

Figure 8.101
Frequency-Damage Curve of the Upper Assiniboine River from Shellmouth Dam to St. Lazare

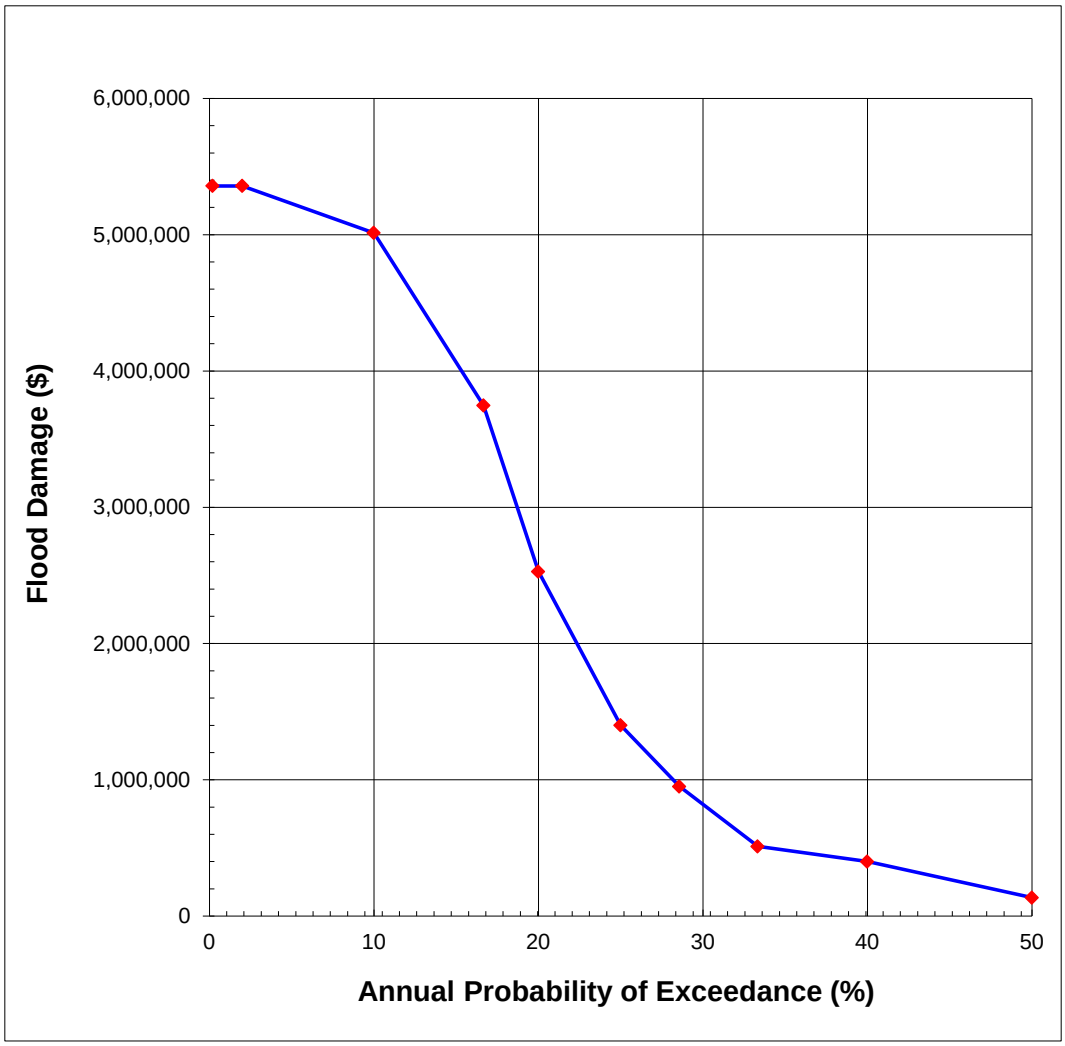
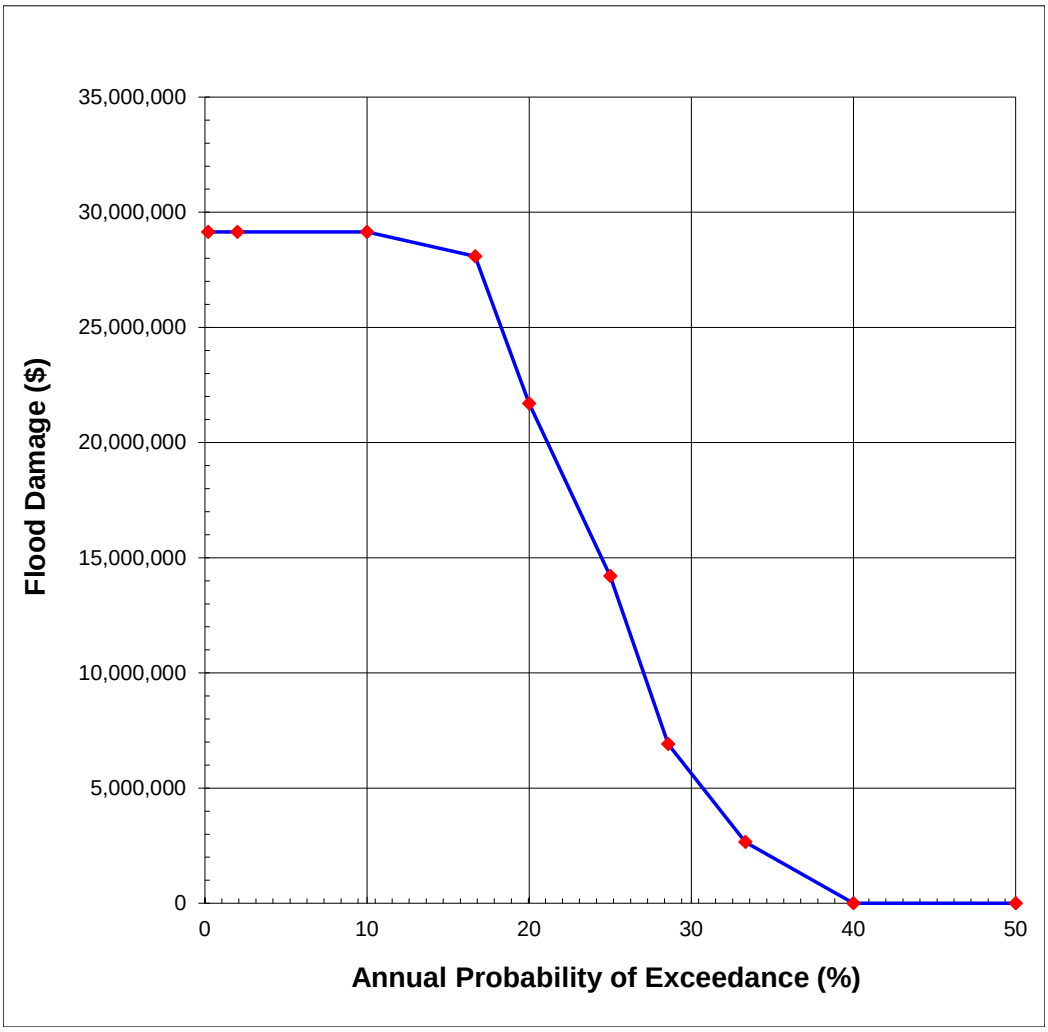


Figure 8.102
Frequency-Damage Curve of the Upper Assiniboine River from St. Lazare to Brandon



8.6.6 Souris River

The study considered dike upgrade schemes in the communities of Melita, Souris and Wawanesa to increase the flood protection level to the 1:200 year and 1:500 year flood events. Benefits of the flood protection schemes in these communities were estimated based on a flood damage model developed by KGS Group for the Town of Melita and the R.M. of Arthur. The details of this model are discussed in the Hydrologic Assessment, Internal Drainage Design & Benefit / Cost Analysis Town of Melita / R.M. of Arthur Final Report – Rev 2 (KGS Group, 2014b).

The model applied similar principles to those that were described in detail in Sections 8.6.1 to 8.6.5 for other areas of the Assiniboine River and Lake Manitoba Basins Flood Mitigation Study. Detailed surveys for the Town of Melita were available from KGS Group (2014b) and identified all structures at risk of flooding. For the communities of Souris and Wawanesa, structures were identified based on aerial imagery.

Based on the analyses conducted for the conceptual design of dike upgrades in the three communities, as later discussed in Section 12.4.1, the Town of Melita is currently protected up to approximately a 1:180 year flood event; and the Town of Souris is protected up to approximately a 1:150 year flood event. In Wawanesa, the current level of protection exceeds the 1:200 year flood. Incremental damages for floods that exceed the current level of protection were then calculated for the communities; the results have been included in Appendix O. Table 8.26 summarizes the average annual benefits that were estimated for the 200 and 500 year floods in the three communities for the proposed dike upgrade schemes.

Table 8.26
Average Annual Benefits of Dike Upgrades for Souris River Communities

Community	1:200 year FPL	1:500 year FPL
Melita	$1,400	$11,200
Souris	$25,600	$85,500
Wawanesa	n/a	$35,700

8.7 LEGACY TOOLS

The terms of reference requested that KGS Group provide to MIT at the end of the study a package of legacy tools that were developed during the course of the study and would be of use in future examination of issues in the study area. The following is a list of the tools and the information that has been assembled for submission to MIT (also attached in Appendix F):

- Background Reports and Synopses.
- HEC-RAS Backwater Models.
- Flood Routing Models.
- Flood Damage Models.

- Other Miscellaneous Models.

Where applicable, descriptions have been included to provide as much value to MIT as possible in the understanding and future use of these tools.

9.0 UPPER ASSINIBOINE RIVER

9.1 INTRODUCTION

The Upper Assiniboine River was defined for this study as the reach of river that extends from the Manitoba border with Saskatchewan to Portage la Prairie. At the upstream-most end of the reach is the Shellmouth Dam, which was constructed in the 1960s to reduce the flood risk on the Assiniboine River and in Winnipeg. Downstream of the Shellmouth Dam, the Assiniboine River meanders at the bottom of an approximately 1 km (0.6 mi) wide valley. Agricultural land located along the river at the bottom of the valley is prone to flooding and at times has been negatively impacted by operation of the Shellmouth Dam.

The Flood Review Task Force recommended in their report (MFRTF, 2013) (Recommendation #77) that alternative means to prevent or reduce flood damages on the Assiniboine River downstream of Shellmouth Dam be investigated. Specifically, they recommended:

1. The examination and quantifying of the flood control benefits of the spillway gates on the Shellmouth Dam.
2. Investigation of the merits of a buy-out of flood prone valley bottom lands downstream of the Shellmouth Dam.
3. An investigation of the feasibility of constructing dikes along the Assiniboine River below the Shellmouth Dam.

All three of these recommendations were carried forward as part of the current study and have been discussed in Sections 9.2, 9.3 and 9.4 of this report.

Several options for new large storage reservoirs were investigated in this study. Two of these reservoirs would be located on the Assiniboine River, and two more would be located on the upper tributaries of the Assiniboine River. The conceptual designs for these reservoirs, as well as the evaluation of their potential hydraulic benefits to the Assiniboine River & Lake Manitoba Basins, have been discussed in Section 9.5.

In addition, the communities of St. Lazare, Sioux Valley Dakota Nation and Brandon are located within this reach of river. St. Lazare and Sioux Valley Dakota Nation are both currently protected

to the highest flood on record (2011 Flood), which exceeds the provincial FPL target of a 1:200 year flood event. In Brandon, commitments are currently in place to increase the level of protection to the highest flood on record (2011 Flood). However, this level does not exceed the 1:200 year FPL. Details of the flood protection system for the City of Brandon, as well as the conceptual design of upgrades to the diking system to increase the FPL above the 1:200 year target have been provided in Section 9.6.

9.2 IMPACT OF OPERATION OF THE SHELLMOUTH DAM GATES ON MAJOR FLOODS

9.2.1 Introduction

Manitoba is considering improving the effectiveness of the discharge capacity for flood control at the Shellmouth Reservoir by installing 1.83 m (6 ft) gates on the presently uncontrolled concrete spillway at the dam. In the early 1990s when the gates were first envisaged, they were intended to compensate for lost storage capacity for floods in the event of concurrent increases in the summer operating level. The gates are now being considered strictly for flood control, with the operation of the dam following the current target for winter drawdowns of the reservoir level and current summer levels for the reservoir.

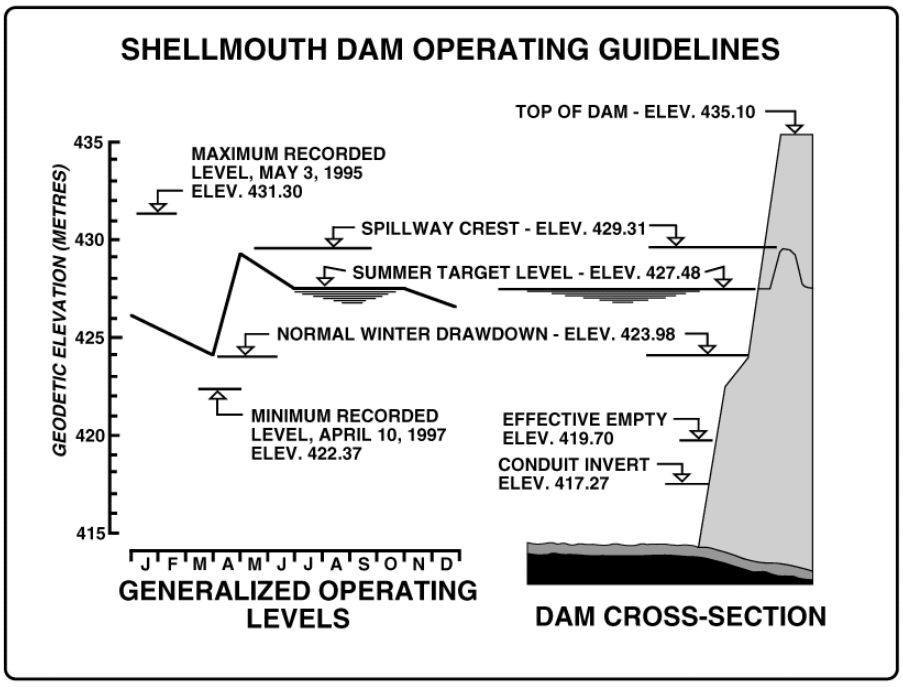
The installation of gates would permit temporary surcharging of the reservoir above the spillway crest by 1.83 m (6 ft) providing an additional 123,600 dam^3 (105,200 ac-ft) of controlled flood storage. Also, the gates would allow conduit flows to be controlled to El 431.14 m (El 1,414.5 ft), thereby providing additional peak shaving capabilities. Currently, with closure of the conduits and using the spillway, peak “shaving” can occur to El 430.59 m (El 1,412.7 ft). This elevation corresponds to a spillway discharge of about 176 m^3/s (6,200 cfs).This is equivalent to the total discharge capacity of the conduits.

It was assumed that the installation of gates would not change the current spring operations. These ensure that the reservoir can provide reasonable levels of both flood protection and water supply capability. Detailed analyses beyond the scope of the current study would be necessary to identify whether a change in operating strategy of the Shellmouth Dam could increase the potential benefits of installing gates on the Spillway. The key elevations and existing operating guidelines are depicted in Figure 9.1. Under the spring operations:

- The reservoir is gradually lowered over the fall and winter period to provide flood storage capacity for the spring runoff, which usually begins at approximately the beginning of April. The winter drawdown target level is somewhat variable, dependent upon the spring inflow forecast. In 2011 and 2013, the reservoir was drawn down to pre-spring water levels of El 421.796 m (El 1,383.85 ft) and El 421.57m (El 1,383.07 ft), respectively.
- During spring runoff, primary consideration is to control reservoir outflows to minimize downstream flooding. Excess inflows are stored in the reservoir until the water level nears the spillway crest elevation of 429.3 m (1,408.5 ft). The intent is to minimize overbank flooding along the river below the dam and during flood periods to keep outflows to a value no greater than what the river flow would have been without the dam in place. At present, an attempt is being made to maintain outflows to a maximum of about 45.3 m^3/s (1,600 cfs) to prevent flooding below the dam and facilitate land drainage. If inflow forecasts indicate that the spillway will be overtopped, the operations are planned such as to minimize the peak outflow through peak shaving.

The reservoir is currently operated for a summer target level of El 427.48 m (El 1,402.5 ft).

Figure 9.1
Summary of Key Parameters of the Shellmouth Dam



Note: Elevations can be converted to feet by dividing metric values by 0.3048.

9.2.2 Assiniboine River Water Management Model

9.2.2.1 Development History

To evaluate the hydrologic impacts of the proposed gates on Shellmouth Dam, the operation of the dam with and without a gated spillway was simulated for the hydrologic period 1913 to 2012. Two versions of a simulation model for the Shellmouth Reservoir were developed during lead up to the Shellmouth Dam Consolidation Agreement (SDCA). One version of the so-called Assiniboine River Water Management Model (ARWMM) addressed the current operation strategy with no gates and an uncontrolled spillway. The second version was configured to include a gated spillway. The intent of the SDCA was to study the enhanced water supply that would be possible by operating the reservoir at higher summer target levels with a gated spillway. The ungated model simulates the current operation of the Shellmouth Dam with the uncontrolled spillway. The other version simulates the operation of Shellmouth Dam with spillway gates operated solely for flood control. For the current analysis in the study reported herein, the gated version of the model was modified so as to follow the current target for winter drawdowns and the same summer levels that have been adopted in the ungated model. The results of the two simulations were compared to explore the relative benefits of gates on Shellmouth Dam.

The ARWMM is considered an effective planning tool that may be used to determine the relative impacts of changes to the management of Shellmouth Reservoir and the Assiniboine River based on long-term hydrologic statistics. The modelling results have been used successfully by the Province in the past for the following projects:

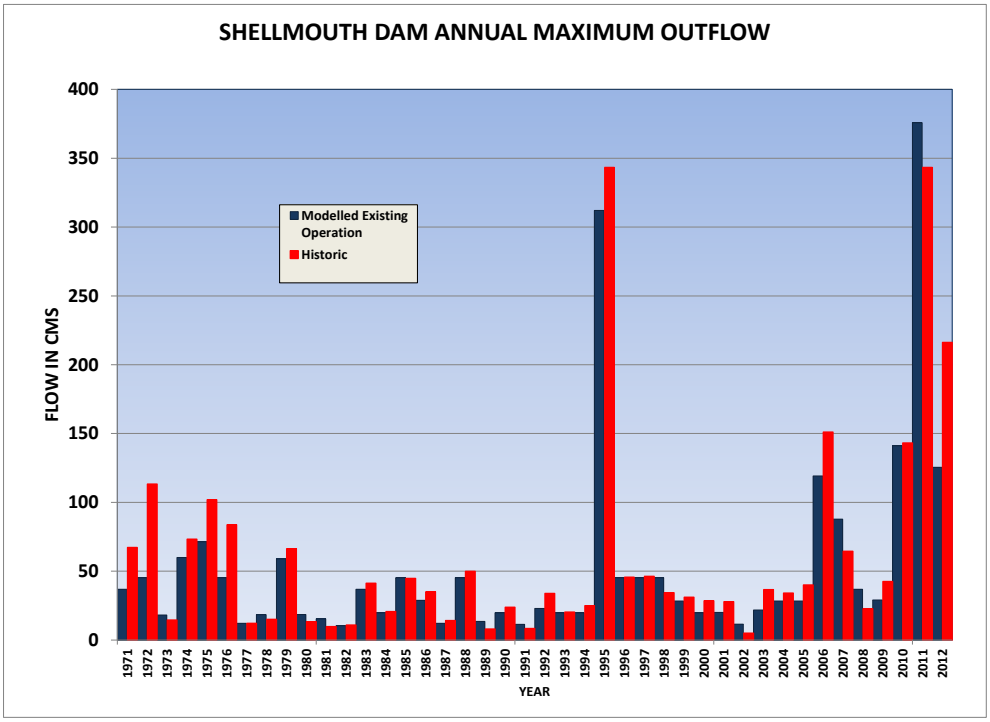
- 1993 Clean Environment Commission (CEC) on Pembina Valley Water Cooperative Inc. Regional Water Supply Proposal.
- Assiniboine River Water Quality Design Flows for CEC Hearings on:
  - City of Brandon Industrial Wastewater Treatment Facility Expansion and Maple Leaf Foods Inc. Hog Processing Plant Alteration.
  - City of Portage la Prairie Water Pollution Control Facility Alterations.
  - City of Winnipeg Wastewater Collection and Treatment Systems.
- CEC Hearings 2000 Simplot Canada Ltd, Potato Processing Plant.

- Assiniboine IFN Study.
- Shellmouth Dam Consolidation Agreement.

As shown on Figure 9.2, the ARWMM does a good job in replicating the current flood control operations of Shellmouth Dam. The only exception was 2012. The modelling indicates that the 2012 Flood peak could have been significantly reduced by more aggressive peak shaving. With more aggressive peak shaving, the maximum outflow could have been reduced from 216 m^3/s (7,630 cfs) to approximately 125 m^3/s (4,420 cfs).

As shown on Figure 9.2, the actual peak outflow in 2011 (and the associated peak water level not shown in the figure) was lower than the simulation predicted. The reason for this is that the Shellmouth Reservoir level was actually drawn down in the special circumstances of 2011 to a pre-spring water level of El 421.796 m (El 1,383.85 ft). In comparison, the drawdown in the model is limited to El 422.75 m (El 1,387 ft). The drawdown level of El 422.75 m (El 1,387 ft) utilized in the model was based on the historic minimum pre-spring drawdown target level prior to 2011.

Figure 9.2
Comparison of Shellmouth Dam Historic and Modelled Flood Control Operations



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

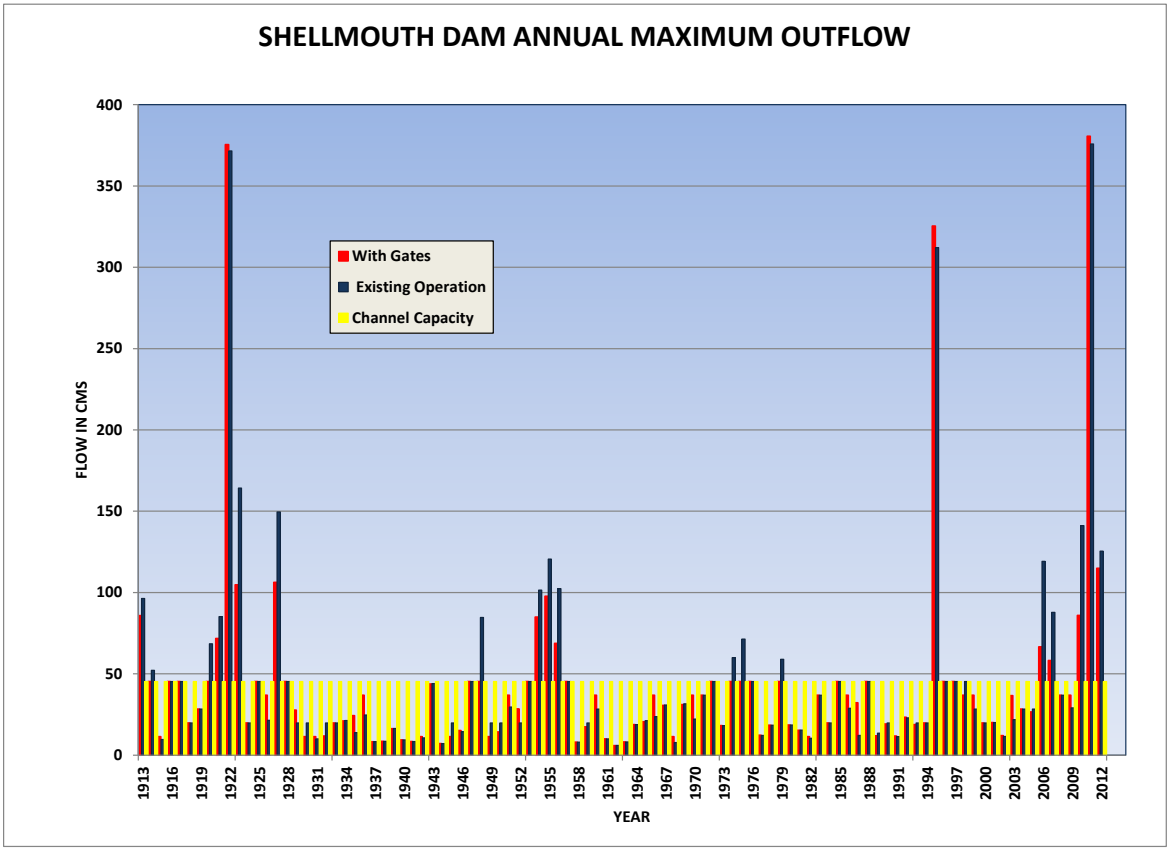
9.2.2.2 Downstream Flood Control Benefits

The modelled peak outflows with and without gates on the Shellmouth Dam for flood control operations are shown on Figure 9.3. The modelling shows that the greatest benefits of gates on Shellmouth Dam are for moderate flood events.

For large flood events such as 2011, the gates could potentially result in slightly higher peak outflows. The reason is that the gates allow the water level in the fall to be higher than it would be with an uncontrolled spillway crest. Consequently, given the restricted outflow of 45.3 m^3/s (1,600 cfs), the drawdown of the reservoir level that follows over the winter does not reach the depths that it would without the gates. For a 2014-type flood, which occurred at the beginning of summer as the result of a large rainfall event, the gates could also potentially result in slightly higher peak outflows. In this case, the available flood storage provided by the gates would have been utilized from the spring runoff resulting in the need to lower the gates to prevent them from being overtopped from the extremely large inflow volume.

Notwithstanding this minor anomaly in some circumstances, a simulation of the 100-year period from 1913 to 2012 shows that gates would provide some modest flood control benefits in about one out of six years on average.

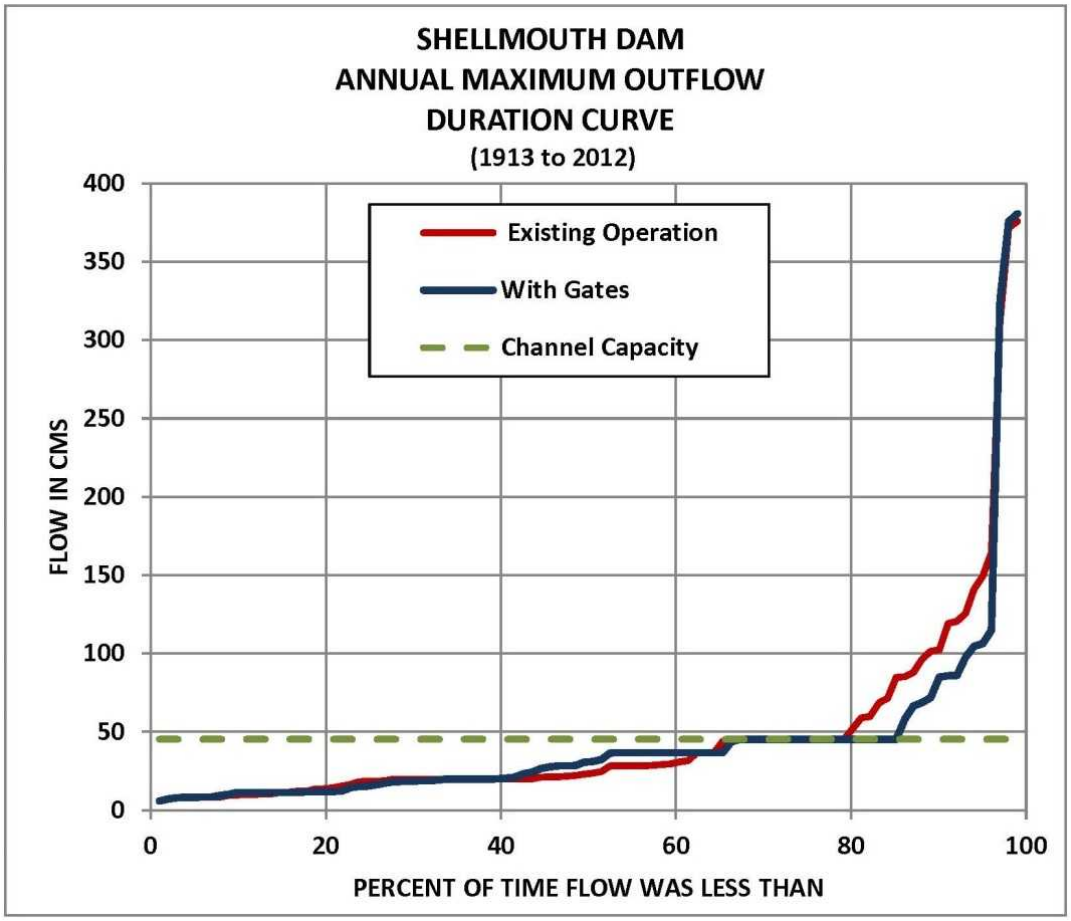
Figure 9.3
Shellmouth Dam Annual Peak Outflows
with and without the Proposed Gates



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

A duration curve of outflows from the Shellmouth Dam effectively illustrates the flood control benefits to be realized by the installation of gates. Duration curves of the annual maximum modelled outflow from Shellmouth Dam with and without the proposed gates are shown on Figure 9.4.

Figure 9.4
Duration Curve of Shellmouth Dam Annual Maximum Outflow with and without Gates (1913 to 2012)



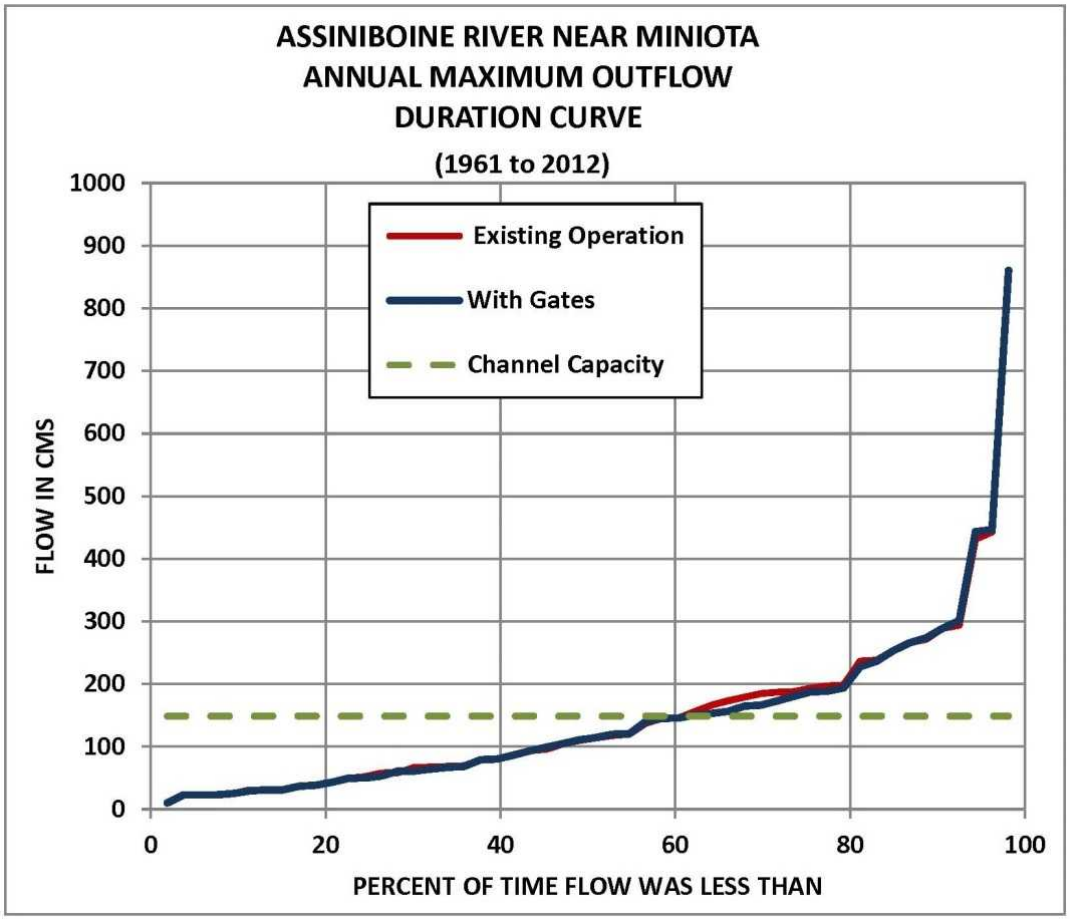
Note: Flows can be converted to cfs by multiplying metric values by 35.3.

The duration curves of annual maximum outflow shown on Figure 9.4 indicate the gates would provide the following benefits immediately below the Dam:

- The gates would reduce the frequency of flooding below the dam from about 78% of the time to 86% of the time.
- The gates would be effective in providing some flood peak shaving. For moderate flood events defined as outflows greater than 45.3 m^3/s (1,600 cfs), and up to approximately 176 m^3/s (6,200 cfs), this additional peak shaving would be provided through the additional flood storage made available, and the ability to control outflows up to El 431.14 m (El 1,414.5 ft). Generally, the gates could result in peak outflows being reduced by as much as 40%.
- The gates would have no benefit on major floods events such as 1995 and 2011.

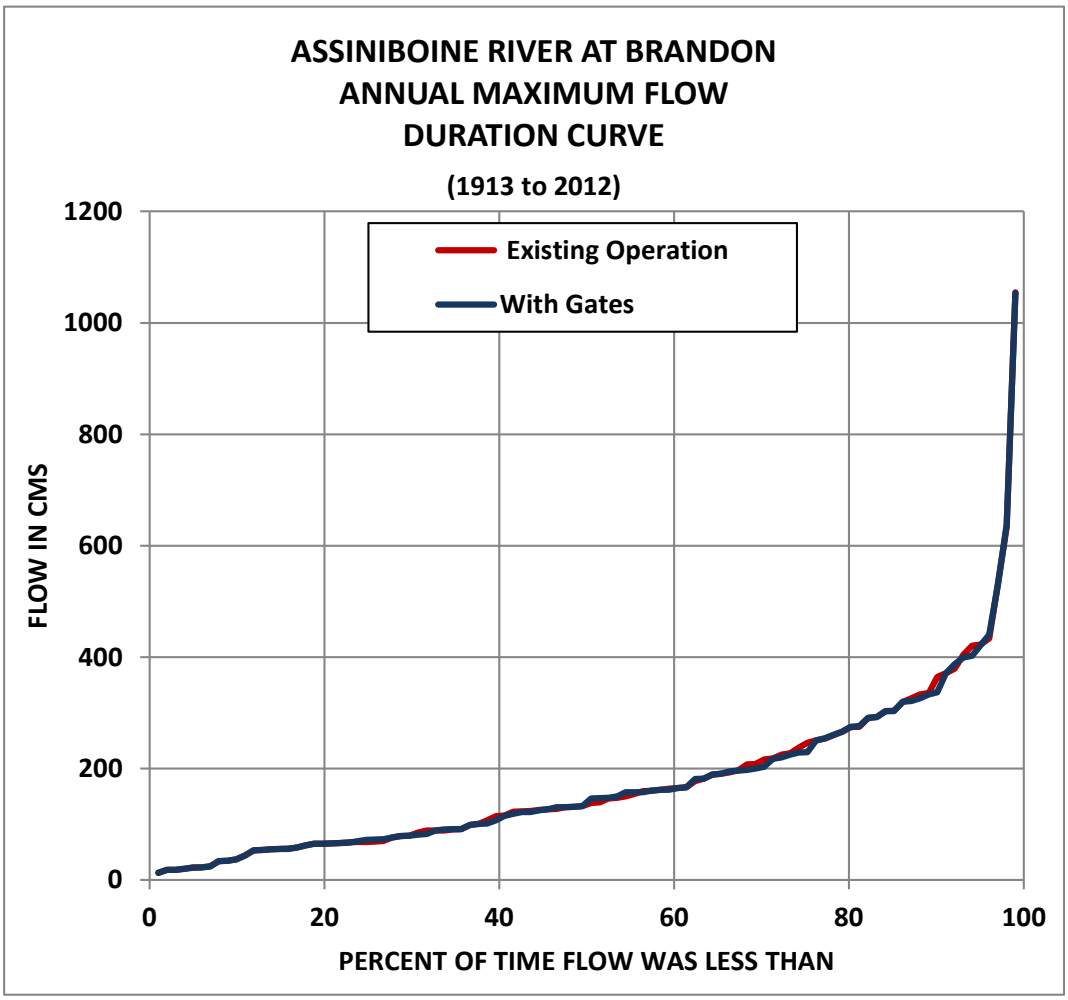
To further understand the downstream flood control benefits of the proposed gates, duration curves of the annual peak discharges were developed for the Assiniboine River at Miniota, Brandon and Portage la Prairie. These duration curves are shown on Figures 9.5, 9.6 and 9.7, respectively.

Figure 9.5
Duration Curve of Annual Maximum Peak Discharge for the Assiniboine River near Miniota with and without Gates (1961 to 2012)



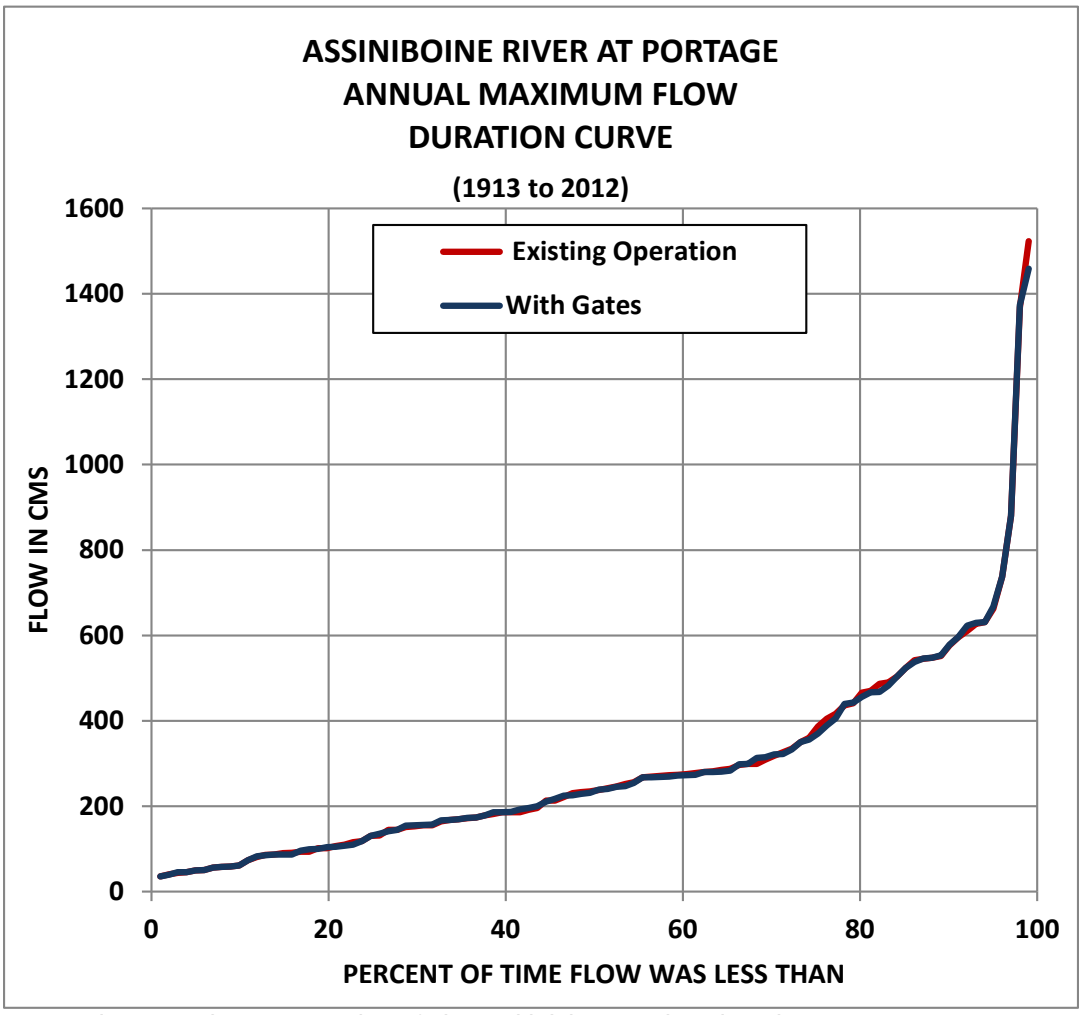
Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 9.6
Duration Curve of Annual Maximum Peak Discharge
for the Assiniboine River at Brandon with and without Gates (1913 to 2012)



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 9.7
Duration Curve of Annual Maximum Peak Discharge
for the Assiniboine River at Portage la Prairie with and without Gates (1913 to 2012)



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figures 9.5 to 9.7 indicate that the flood control effects of the gates diminish as the distance from the dam increases and disappear entirely by approximately Brandon. This is due to river channel storage effects and the contribution and timing of downstream tributary runoff such as the Souris and Qu'Appelle rivers and the Manitoba tributaries. The annual flood peaks at Brandon and downstream are generally produced by the Manitoba tributary runoff downstream of Shellmouth Dam and are, therefore, essentially unaffected by the releases from the dam. At Miniota, installation of gates on Shellmouth Dam would still have a noticeable impact, but much less than immediately below the Shellmouth Dam.

As per the terms of reference for the current study, it was assumed for the analyses of the various flood mitigation alternatives that the gates on Shellmouth Dam are in place as the baseline condition. However, as discussed above, the effects of the gates are localized immediately downstream of the dam and for minor to moderate floods only. As such, the inclusion of gates only effected the options of diking and land purchases along the Upper Assiniboine River Valley. These options are discussed further in Sections 9.3 and 9.4.

9.2.3 Conclusions

The findings described above indicate that the installation of gates on the Shellmouth Spillway, assuming that the installation of gates would not change the current operating strategy, would have only minor, negligible potential to reduce the outflows during major floods such as 2011.

This conclusion appears to be counter-intuitive. One would expect that the increased control capability provided by spillway gates would surely provide some potential to reduce the peaks of major floods. The reason that this cannot be achieved stems directly from the original design of the Shellmouth Dam. The elevation of the crest of the dam was selected in that design process to be based on the maximum reservoir level that would occur under the design flood. The computation for that design condition was based on the assumption of an ungated spillway. The floods of 1995 and 2011 had runoff volumes several times greater than the available flood storage in the reservoir. If the gates had been operated to reduce outflows during such large floods, the additional water stored behind the dam would have added to the already high levels in the reservoir, potentially threatening to overtop the dam. For large flood events, the spillway gates must remain wide open until the peak inflows have passed, thereby minimizing the risk of a catastrophic overtopping.

Detailed analyses beyond the scope of the current study would be necessary to identify whether a change in operating strategy of the Shellmouth Dam could increase the potential benefits of installing gates on the Spillway.

9.3 DIKING THE UPPER ASSINIBOINE RIVER VALLEY

9.3.1 Introduction

The 2011 Flood Review Task Force recommended that the Province investigate the feasibility of constructing dikes along the margins of the Assiniboine River downstream of Shellmouth Dam to reduce future flood damages on flood prone valley bottom lands and to increase the operational flexibility of the Shellmouth Dam. This recommendation was considered for the current study.

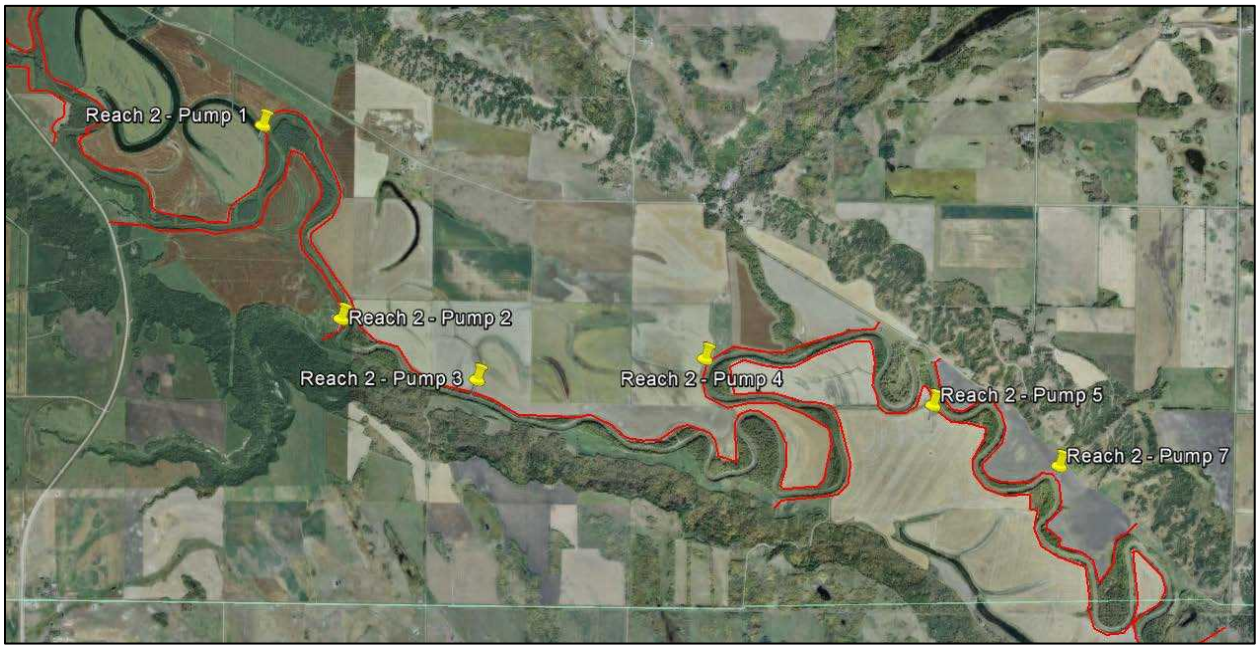
This section of the report describes the conceptual design of two diking schemes: one extends from the Shellmouth Dam to St. Lazare, and the other extends from the Shellmouth Dam to Brandon. The conceptual design was prepared for the 1:5 year and the 1:50 year flood events.

9.3.2 Layout of Dikes

The Assiniboine River channel meanders from one side of the valley to the other and is also intertwined among numerous oxbows and old abandoned channels. Developing a dike alignment along the entire Assiniboine River Valley was determined to be impractical for a conceptual type study due to the complexity of the river channel alignment. Instead, a 13 km (8.1 mi) reach of the Assiniboine River Valley near Shellmouth Dam, and another 16 km (9.9 mi) reach near Miniota, both considered representative of conditions along the entire valley, were selected for the conceptual design; and the results were extrapolated for the entire length of the diking schemes.

The conceptual alignment of the dikes along the two representative reaches was determined based on aerial imagery of the valley and generally followed the limits of the cultivated land along the river. Judgement was applied to determine cut-off areas where diking appeared impractical due to difficult conditions such as river meanders or oxbows. A sketch showing an example dike alignment along the river near Miniota is provided on Figure 9.8.

Figure 9.8
Conceptual Layout of Dikes and Location of Pump Stations



Once a conceptual alignment was determined for the two representative reaches, a ratio of dike length per length of river valley was determined. The length of river valley was defined as the distance between two points calculated by following the centreline of the valley. The average ratio between the two representative reaches was determined to be 2.8 km (1.7 mi) of dikes per kilometre of river valley. Based on this ratio, the estimated total length of dikes required from Shellmouth Dam to St. Lazare was approximately 180 km (111.8 mi), and from Shellmouth Dam to Brandon was approximately 570 km (354.2 mi).

9.3.3 Dike Construction Quantities

Rating curves provided by MIT at eight locations along the river were used to estimate the base elevation and top elevations of the dikes. The rating curves are provided in Appendix D and are located at bridge crossings along the river. The base elevation of the dikes was set equal to the existing channel capacity of the river as defined by MIT in the 2011 Flood sheets, with 0.6 m (2 ft) added to the top of the dike as a freeboard allowance.

A potential shift in the rating curve due to a loss in conveyance from elimination of the floodplain was not considered in the analysis. In reality, the dikes would have to be constructed higher,

and would, therefore, be more costly. However, this conservative approach was determined to be acceptable for a conceptual level study, considering that a more detailed analysis would reduce the economic viability of the project due to the increased capital cost.

Based on local knowledge of the study area by KGS Group team members, it was identified that dikes had previously been constructed along the river in the late 1950s in a few flood prone areas. However, these dikes were ineffective in providing flood protection during the most recent flood events along the Assiniboine River, even after attempts were made by locals to restore the dikes. On this basis, existing dikes were not considered in the analysis. Additionally, due to uncertainties with the suitability of the material that formed these dikes, it was assumed for this study that any existing dikes within the area could not be incorporated into the proposed design of new dikes. Furthermore, it was determined that any potential benefit that would be gained from being able to use residual material from existing dikes would be quickly offset if the loss in conveyance from elimination of the flood plain was considered.

The height of dikes required for the 1:5 year and the 1:50 year flood events at the eight bridge locations considered in the analysis are provided on Tables 9.1 and 9.2. The volume of fill material required for the dikes is also provided on the tables, based on an assumed 4 m (13 ft) top width and 4:1 side slopes. The flows corresponding to the 1:5 year event (assuming that the Shellmouth Dam gates are in place as discussed in Section 9.2) gradually increase from approximately 57 m^3/s (2,000 cfs) at Shellmouth to approximately 267 m^3/s (9,400 cfs) in Brandon. For the 1:50 year event, the flows gradually increase from approximately 190 m^3/s (6,700 cfs) at Shellmouth to approximately 697 m^3/s (24,600 cfs) in Brandon. Flood frequency curves at various locations along the Assiniboine River are provided in Appendix C.

Quantities for land acquisition were estimated based on the footprint of the proposed dikes. An additional 15 m (49.2 ft) width was then added for internal drainage requirements. The same area was also assumed to require revegetation. The total area of land acquisition and revegetation was then calculated by interpolating between each of the eight bridge locations considered in the analysis. A summary of quantities for land acquisition and re-vegetation is included in Tables 9.1 and 9.2.

Table 9.1
Estimated Quantities for a 1:5 Year Flood Protection Level

Location	Length of Dike along Upstream Reach	Height of Dikes	Volume of Dike Fill Along Upstream Reach	Area of Land Acquisition and Revegetation along Upstream Reach
	(km)	(m)	(m^3)	(ha)
PR 549 near Shellmouth	18	1.0	145,099	48
PTH 16 near Russell	42	0.8	148,855	108
PR 579 at Millwood	38	1.0	166,861	102
PTH 49 at St. Lazare	83	0.7	218,805	205
PTH 83 near Miniota	123	1.5	930,154	378
PR 259 near Virden	91	1.5	799,192	282
PTH 21 near Griswold	98	1.0	456,237	264
PTH 1 at Grand Valley	79	1.5	670,640	247
Total – Shellmouth to St. Lazare	**181**		**680,000**	**463**
Total – Shellmouth to Brandon	**571**		**3,536,000**	**1,634**

Table 9.2
Estimated Quantities for a 1:50 Year Flood Protection Level

Location	Length of Dike along Upstream Reach	Height of Dikes	Volume of Dike Fill Along Upstream Reach	Area of Land Acquisition and Revegetation along Upstream Reach
	(km)	(m)	(m^3)	(ha)
PR 549 near Shellmouth	18	2.5	630,536	69
PTH 16 near Russell	42	2.9	1,792,729	179
PR 579 at Millwood	38	3.3	2,335,426	172
PTH 49 at St. Lazare	83	1.8	1,352,660	275
PTH 83 near Miniota	123	2.9	3,246,696	519
PR 259 near Virden	91	2.3	2,001,186	336
PTH 21 near Griswold	98	2.0	1,541,130	343
PTH 1 at Grand Valley	79	3.2	2,720,909	351
Total – Shellmouth to St. Lazare	**181**		**6,111,000**	**696**
Total – Shellmouth to Brandon	**571**		**15,621,000**	**2,245**

Due to the complexity of the river channel along the Upper Assiniboine River Valley, riprap requirements for erosion protection were not quantified at this conceptual stage of design. Requirements for clearing and grubbing, as well as for granular toe drain material for the dikes, were also not quantified. Rather, a miscellaneous quantity allowance equivalent to 20% of the total direct costs was added to the cost estimate to account for these items. Detailed analyses and field investigations would be necessary to accurately define these quantities, and would likely show that the 20% allowance is low. However, this conservative approach was determined to be acceptable for a conceptual type study, considering that a more detailed analysis would reduce the economic viability of the project.

Additionally, a provision for granular road topping and for access roads at regular intervals for heavy construction equipment in an emergency situation was not considered at this stage of design. Should the diking concept ever proceed to a subsequent level of analysis, the necessities and specific requirements for providing all season access would have to be considered in detail. This would add to the overall cost of the project, further reducing its economic viability.

9.3.4 Pumping Requirements

Pumping stations would be required at regular intervals to drain runoff from the agricultural fields in periods of floods. To minimize pumping requirements, internal drainage could be routed through old oxbows or small storage ponds. Based on an assessment of the existing drainage network using air photos of the two representative reaches identified in Section 9.3.2, it was estimated that approximately one pump station would be required every 1.5 km (0.9 mi) of river valley. On this basis, approximately 40 pump stations would be required between the Shellmouth Dam and St. Lazare, and approximately 140 pump stations between the Shellmouth Dam and Brandon. Figure 9.8 shows as an example the general concept of where pump stations would be required along the river near Miniota.

Based on a hydrologic analysis of the area, it was determined that on average 0.5 m^3/s (18 cfs) capacity pumps with an average head of 2.5 m (8 ft) would be of sufficient size to drain local runoff, assuming a 1:10 year rainfall event and that pumping would occur over a period of one week. The estimated cost for pumping was assumed at $500,000 per pump, based on a minimum fixed cost of $120,000, plus a unit rate of $300,000 per cubic metre per metre of head.

This was determined based on review of the actual construction costs of recent, small sized, pumping stations constructed in Winnipeg and at other locations in Manitoba.

9.3.5 Other Considerations

Construction of dikes along the Upper Assiniboine River valley would eliminate the floodplain and natural storage available along the river. Not only would this increase the river water levels locally, it would also significantly impact the conveyance system and likely increase discharges downstream on the Assiniboine River, potentially all the way to Winnipeg. The magnitude of these potential effects have not been quantified at this conceptual level of study, however, it is anticipated that mitigating these impacts would further increase the cost of the proposed project.

Other environmental impacts are expected to be minor for this scheme. The dikes would likely cut-off numerous oxbows and old abandoned channels from the river. During large flood events which would overwhelm the diking system, flood waters could remain trapped behind the dikes. To allow flood waters to re-enter the river, cutting sections of dikes may be required. This would be difficult to achieve on an emergency basis considering that the diking system would have limited access. Cutting off dike sections could also result in localised erosion and changes to the flow patterns, which would not be desirable. Increased water levels and potential flow patterns from elimination of the flood plain could also change the sedimentation and erosion processes in the river. On this basis, an allowance for environmental mitigation has been included in the cost estimate of the scheme.

9.3.6 Cost Estimate

The estimated capital costs of the diking scheme are summarized in Tables 9.3 to 9.6.

Table 9.3
Estimated Costs for 1:5 Year Dikes from Shellmouth Dam to St. Lazare

Item	Description	Cost
Volume of Dike Material	680,000 m^3 @ \$20 / m^3	\$ 13,600,000
Pump Stations	40 @ \$500,000 / unit	\$ 20,000,000
Land Acquisition	463 ha @ \$5,000 / ha	\$ 2,300,000
Re-vegetation	463 ha @ \$5,500 / ha	\$ 2,500,000
Miscellaneous Quantity Allowance	20% of direct costs	\$ 7,700,000
Environmental Mitigation Costs	Minor costs estimated at 5% of direct costs	\$ 1,900,000
Indirect costs (Owner's costs, engineering, administration, approvals)	Estimated at 20% of direct costs	\$ 7,700,000
Contingency Allowance	Estimated at 30% of direct & indirect costs	\$ 16,700,000
	Total Cost	**\$ 72,500,000**

Table 9.4
Estimated Costs for 1:50 Year Dikes from Shellmouth Dam to St. Lazare

Item	Description	Cost
Volume of Dike Material	6,111,000 m^3 @ \$20 / m^3	\$ 122,200,000
Pump Stations	40 @ \$500,000 / unit	\$ 20,000,000
Land Acquisition	696 ha @ \$5,000 / ha	\$ 3,500,000
Re-vegetation	696 ha @ \$5,500 / ha	\$ 3,800,000
Miscellaneous Quantity Allowance	20% of direct costs	\$ 29,900,000
Environmental Mitigation Costs	Minor costs estimated at 5% of direct costs	\$ 7,500,000
Indirect costs (Owner's costs, engineering, administration, approvals)	Estimated at 20% of direct costs	\$ 29,900,000
Contingency Allowance	Estimated at 30% of direct & indirect costs	\$ 65,000,000
	Total Cost	**\$ 282,900,000**

Table 9.5
Estimated Costs for 1:5 Year Dikes from Shellmouth Dam to Brandon

Item	Description	Cost
Volume of Dike Material	3,536,000 m^3 @ \$20 / m^3	\$ 70,700,000
Pump Stations	140 @ \$500,000 / unit	\$ 70,000,000
Land Acquisition	1,634 ha @ \$5,000 / ha	\$ 8,200,000
Re-vegetation	1,634 ha @ \$5,500 / ha	\$ 9,000,000
Miscellaneous Quantity Allowance	20% of direct costs	\$ 31,600,000
Environmental Mitigation Costs	Minor costs estimated at 5% of direct costs	\$ 7,900,000
Indirect costs (Owner's costs, engineering, administration, approvals)	Estimated at 20% of direct costs	\$ 31,600,000
Contingency Allowance	Estimated at 30% of direct & indirect costs	\$ 68,700,000
	Total Cost	**\$ 297,800,000**

Table 9.6
Estimated Costs for 1:50 Year Dikes from Shellmouth Dam to Brandon

Item	Description	Cost
Volume of Dike Material	15,621,000 m^3 @ \$20 / m^3	\$ 156,200,000
Pump Stations	140 @ \$500,000 / unit	\$ 70,000,000
Land Acquisition	2,245 ha @ \$5,000 / ha	\$ 11,200,000
Re-vegetation	2,245 ha @ \$5,500 / ha	\$ 12,300,000
Miscellaneous Quantity Allowance	20% of direct costs	\$ 50,000,000
Environmental Mitigation Costs	Minor costs estimated at 5% of direct costs	\$ 12,500,000
Indirect costs (Owner's costs, engineering, administration, approvals)	Estimated at 20% of direct costs	\$ 50,000,000
Contingency Allowance	Estimated at 30% of direct & indirect costs	\$ 108,700,000
	Total Cost	**\$ 470,800,000**

9.3.7 Conclusions

The estimated capital cost for diking the agricultural land from the Shellmouth Dam to St. Lazare ranges from \$72.5 million for a 1:5 year event FPL to \$282.9 million for a 1:50 year event FPL.

The estimated capital cost for diking the agricultural land from the Shellmouth Dam to Brandon ranges from $297.8 million for a 1:5 year event FPL to $470.8 million for a 1:50 year event FPL.

The cost of the options would be greater if higher dikes due to increased water levels from the elimination of the flood plain were considered in the design. Requirements for granular road topping and for access roads at regular intervals for heavy construction equipment in an emergency situation would also increase the cost of the options if incorporated into the design.

Due to the very high costs of the diking schemes identified, even at the low 1:5 year return period event, and when compared to the proposed land purchasing scheme identified in Section 9.4 of the report, the proposed Upper Assiniboine River diking schemes have not been identified as preferred alternatives for this study. Furthermore, the likely increase in flood flows downstream on the Assiniboine River, potentially all the way to Winnipeg, could have significant impacts that would have to be addressed.

9.4 LAND PURCHASE ALONG THE UPPER ASSINIBIONE RIVER VALLEY

An option to purchase the flood prone lands within the Upper Assiniboine River Valley between Shellmouth Dam and St. Lazare was investigated as recommended by the 2011 Flood Review Task Force. This reach of river is vulnerable to flooding at flows of about 45 m^3/s (1,600 cfs), which is less than the reach between St. Lazare and Brandon where the river has a capacity of about 142 m^3/s to 170 m^3/s (5,000 cfs to 6,000 cfs).

A flow of approximately 85 m^3/s (3,000 cfs) was selected for this scheme as the proposed flood protection level between Shellmouth Dam and St. Lazare because it would almost double the current capacity of the river. It would also increase the return period at which the flood stage was exceeded from an event that occurs approximately 1:3 years to an event that occurs approximately 1:10 years, assuming that the Shellmouth Dam gates are in place as discussed in Section 9.2. Without the gates, the return period corresponding to the 85 m^3/s (3,000 cfs) capacity would be an event that occurs approximately 1:6 years, instead of 1:10 years. The 85 m^3/s (3,000 cfs) capacity would not only significantly reduce the extent of damages within this reach during a flood, but would also provide greater operating flexibility for the Shellmouth Dam, since flow releases from the dam prior and after a flood are influenced by the river capacity.

Flooded area maps of the Upper Assiniboine River provided by MIT and attached in Appendix J formed the basis for the cost estimate for the land purchase option. Based on data from MIT, approximately 700 ha (1,730 ac) of agricultural land is flooded at the flow of 85 m^3/s (3,000 cfs), of which about 215 ha (530 ac) consists of annual crops, and 485 ha (1200 ac) are forage and grasslands. An additional 1,080 ha (2,680 ac) of private non-agricultural land is also flooded at this flow.

Many sections of land within the river valley are not flooded at a flow of 85 m^3/s (3,000 cfs). However, these sections often become inaccessible due to surrounding flooding. This significantly increases the total area of land affected by a flood magnitude of 85 m^3/s (3,000 cfs), as this land cannot be seeded, harvested or used as rangeland during flood periods. It was, therefore, assumed that this land would have to be purchased for this scheme. Furthermore, in some locations, additional land outside the flooded area would also have to be purchased if the remaining area became too small or uneconomical for the landowner to continue using for agriculture purposes. Based on a review of the flood maps in the area, it was assumed that this non-flooded land covered an area twice the size of the flooded area.

Based on assumed unit rates of $10,000/ha ($4,000/ac) for the agricultural land used for annual crops and $5,000/ha ($2,000/ac) for forage and grasslands, as well as $1,000/ha ($400/ac) for the remaining non-arable land, the estimated cost of land purchases along the Upper Assiniboine River Valley between the Shellmouth Dam and St. Lazare, including a 20% allowance for indirect costs (administration, approvals, etc.) and 30% for contingencies, was $24 million. As outlined above, this would provide a 1:10 year level of protection in the area.

When compared to the cost of a linear diking system, as discussed in Section 9.3, the estimated cost of the land purchase option was significantly less in comparison. The land purchase option also did not have the negative impact of cutting off oxbows and abandoned channels from the river, or of eliminating the floodplain storage that could lead to an increase in the discharges downstream, which would be associated with the diking scheme. For these reasons, the land purchase option was the preferred alternative over linear diking and should be considered further.

9.5 LARGE STORAGE RESERVOIRS

9.5.1 Introduction

A total of six large storage reservoirs were evaluated as flood mitigation alternatives for this study. Two of them are located on the Souris River and are discussed in Section 12.3. The remaining four are listed below and discussed in the sections of the report that follow:

1. The Holland Dam on the Assiniboine River west of Portage la Prairie.
2. The Alexander Dam on the Assiniboine River west of Brandon.
3. The Victor Dam on the Qu'Appelle River west of St. Lazare.
4. The Zelena Dam on the Shell River north of Roblin.

The conceptual designs of the four storage reservoir project alternatives were based on results from previous studies. Specific background information, as well as the hydrologic and hydraulic conditions for each of the conceptual projects, is described in Sections 9.5.2 to 9.5.5.

A plan map showing the location of the four alternatives is provided in Drawing 9.1 (located at the end of Section 9.0). Though varying in size, the concepts of the four projects are very similar. Typical plan and cross sections of the Holland Dam project were selected to provide a representative illustration of the proposed concepts for the four alternatives. These were taken from the Saskatchewan Nelson Basin Board (SNBB, 1972) and are provided in Figures 9.9 to 9.11.

Figure 9.9
Representative Plan of Structures

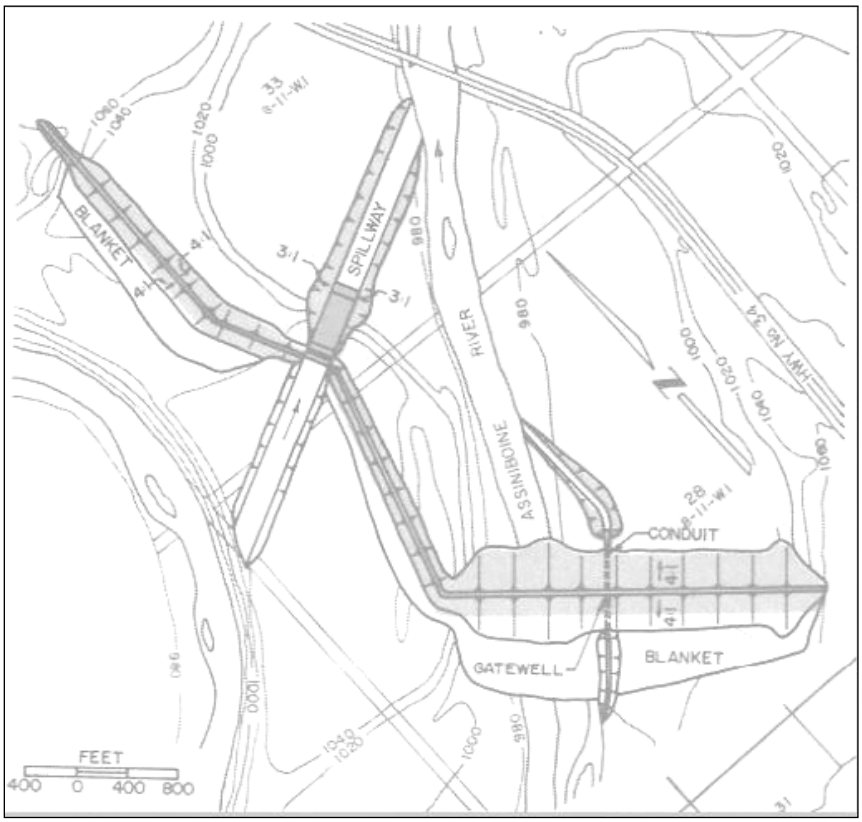


Figure 9.10
Representative Valley Cross Section at Dam Site

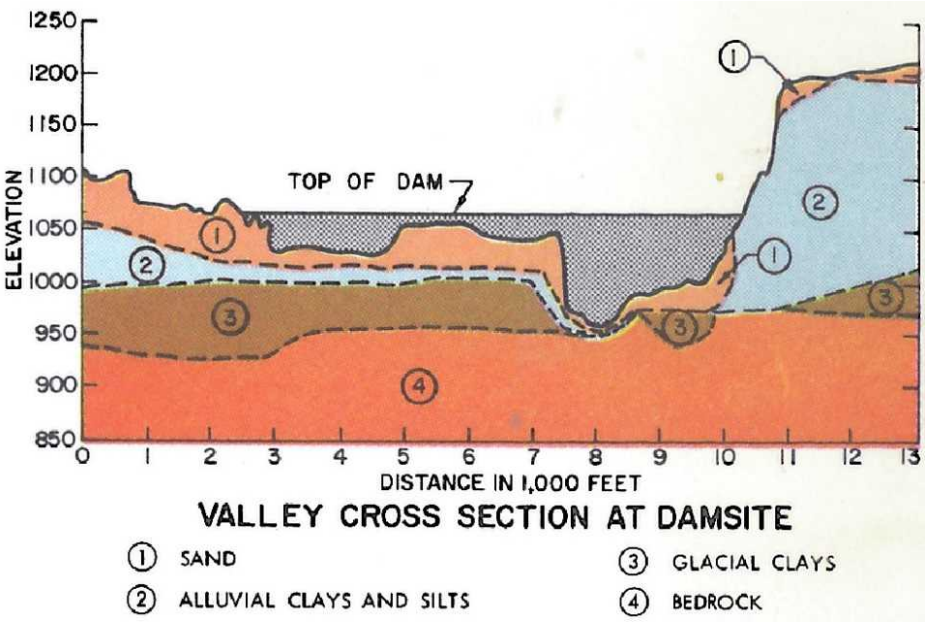
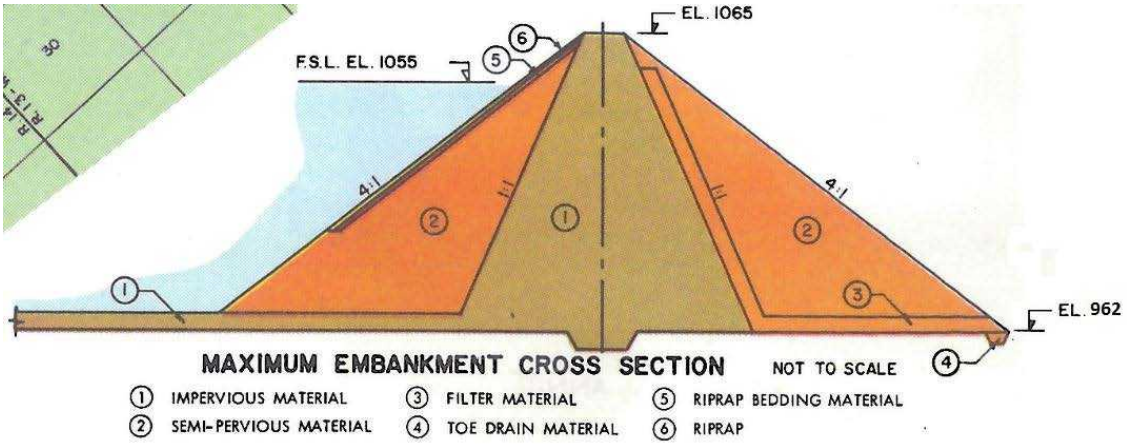


Figure 9.11
Representative Cross Section of Dam Embankment



9.5.2 Holland Dam

9.5.2.1 Background

The Holland Dam would be located on the Assiniboine River approximately 50 km (30 mi) southwest of Portage la Prairie. The reservoir would have a length of 37 km (23 mi). During the 1950s, the Prairie Farm Rehabilitation Administration (PFRA) investigated several potential reservoir sites for water supply on the Assiniboine River between Brandon and Portage la Prairie. Of the sites examined, PFRA determined that the most suitable was located approximately 50 km (30 mi) upstream and southwest of Portage la Prairie, near the village of Holland, Manitoba. In 1958, the Royal Commission on Flood Cost Benefit, having completed its investigations of the situation in the Red River Valley, recommended that the Holland Dam, as well as the Shellmouth Reservoir and the Portage Diversion, should be re-examined with a view toward providing enhanced flood control along the Assiniboine River and for the City of Winnipeg.

In 1960, the Province of Manitoba conducted an analysis of the benefits and costs of the Assiniboine River projects including the Holland Dam. In his January 1961 report to the Province, R. Kuiper (Kuiper, 1961), a professor of engineering at the University of Manitoba, concluded that the most feasible plan for flood control and water conservation would be a combination of the Shellmouth Dam and the Portage Diversion. The Holland Dam would provide

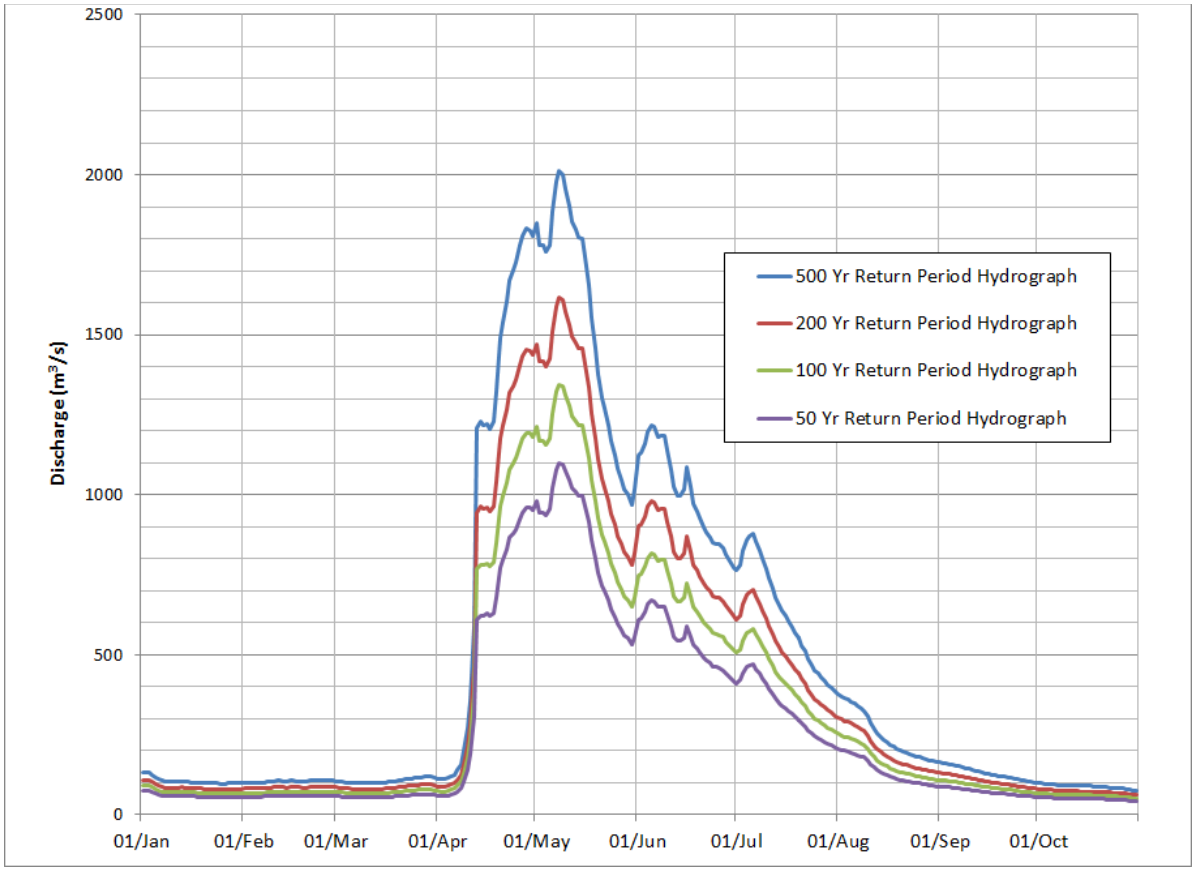
fewer benefits at a cost nearly equal to the cost of building both Shellmouth and the Diversion. In addition, Shellmouth would provide flood control along the Assiniboine from the dam through Brandon, benefits which the Holland Dam could not provide. In addition, the Holland Dam would be less effective at reducing flows on the Assiniboine River. The Holland Dam project was re-visited by the Saskatchewan Nelson Basin Board (SNBB) and documented in the 1972 report titled, "*Water Supply for the Saskatchewan-Nelson Basin*" (SNBB, 1972). Results from the SNBB 1972 report formed the basis of the analysis for the current study.

The proposed project would create a flooded area approximately 37 km (23 mi) long and covering about 7,500 ha (18,500 ac). Several farmsteads would have to be relocated as a result of the flooded area, and PTH 5 would be affected.

9.5.2.2 Hydrology

Design hydrographs for the Assiniboine River at the Holland Dam have been computed based on discharge frequency curves for the Assiniboine River at Holland provided by MIT and volume frequency curves prepared for this study, as documented in Section 5.4 of this report. The design hydrographs for return periods from 50 years to 500 years are shown on Figure 9.12.

Figure 9.12
Assiniboine River at Holland Design Hydrographs



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.2.3 Hydraulics

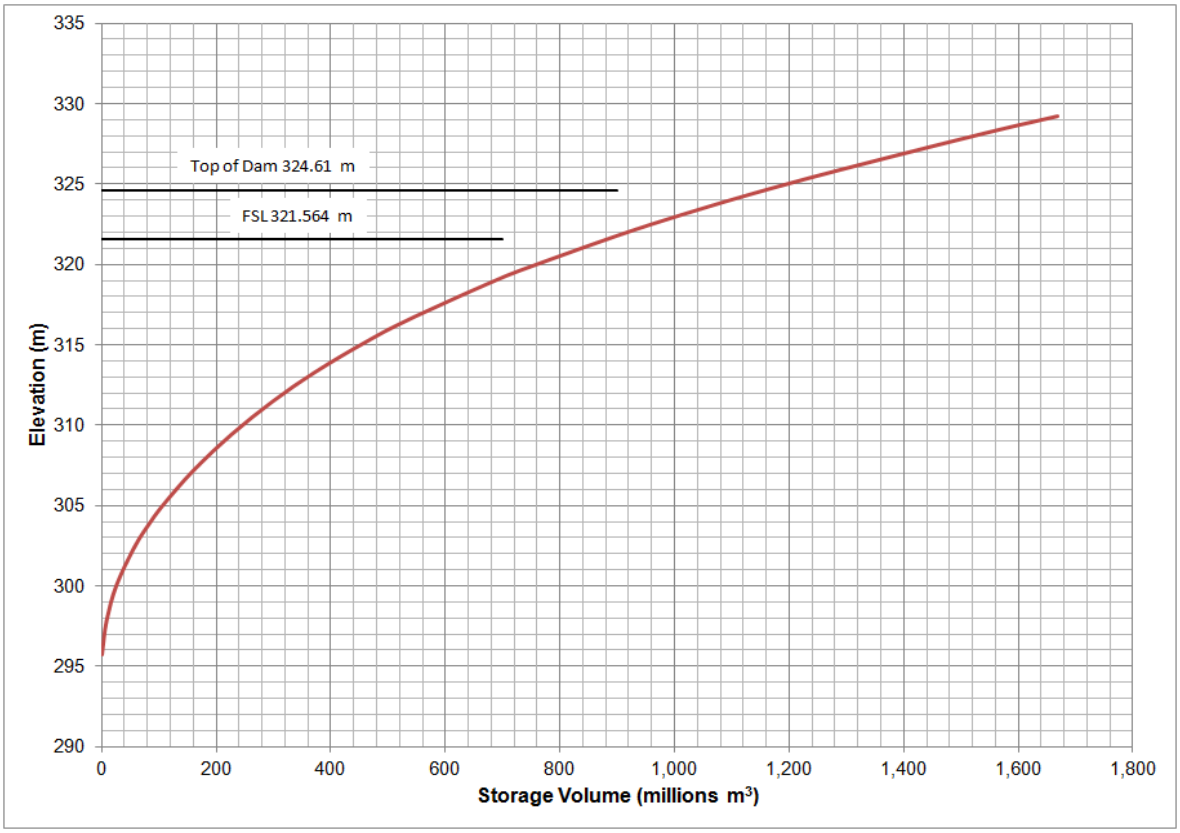
The Holland Dam would include an earth embankment, a spillway and an outlet conduit. The proposed top of dam elevation would be 324.6 m (1,065 ft) with a crest length of 2,438 m (8,000 ft). Full supply level (FSL) of the reservoir would be EL 321.6 m (El 1,055 ft). It would have a storage capacity of 882,000,000 m^3 (720,000 ac-ft).

The concept considered a 4.6 m x 4.6 m (15 ft x 15 ft) horseshoe shaped conduit, which would be built on the east side of the river. The length of the conduit would be 213 m (700 ft), and the riparian capacity at full supply level (FSL) would be 236 m^3/s (8,350 cfs). The conduit invert elevation would be 295.7 m (970 ft). An overflow spillway would be built on the west side of the river. The crest elevation of the spillway would be 317.9 m (1,043 ft).

The original design for the Holland Dam overflow spillway consisted of five, 12.2 m (40 ft) wide bays controlled by 3.7 m high x 12.2 m wide (12 ft x 40 ft) radial gates. For the current study, vertical lift gates were considered instead of radial gates. Outflow can be controlled by operating these gates to achieve maximum available storage. The final arrangement for this study included four additional spillway bays, for a total net width of 109.7 m (360 ft). The additional capacity was included to provide additional outflow during the early part of the flood to reserve capacity for attenuating the peak flood flows.

The structure length would be 185.0 m (605 ft) and the design discharge would be 2,010 m^3/s (71,000 cfs). The stage-storage curve shown on Figure 9.13 was taken from the Saskatchewan Nelson Basin Board Report (SNBB, 1972).

Figure 9.13
Holland Reservoir Storage Curve



Stage-discharge rating curves for the Low Level Conduit and the Overflow Spillway are illustrated in Figure 9.14 and Figure 9.15, respectively.

Figure 9.14
Overflow Spillway Rating Curve

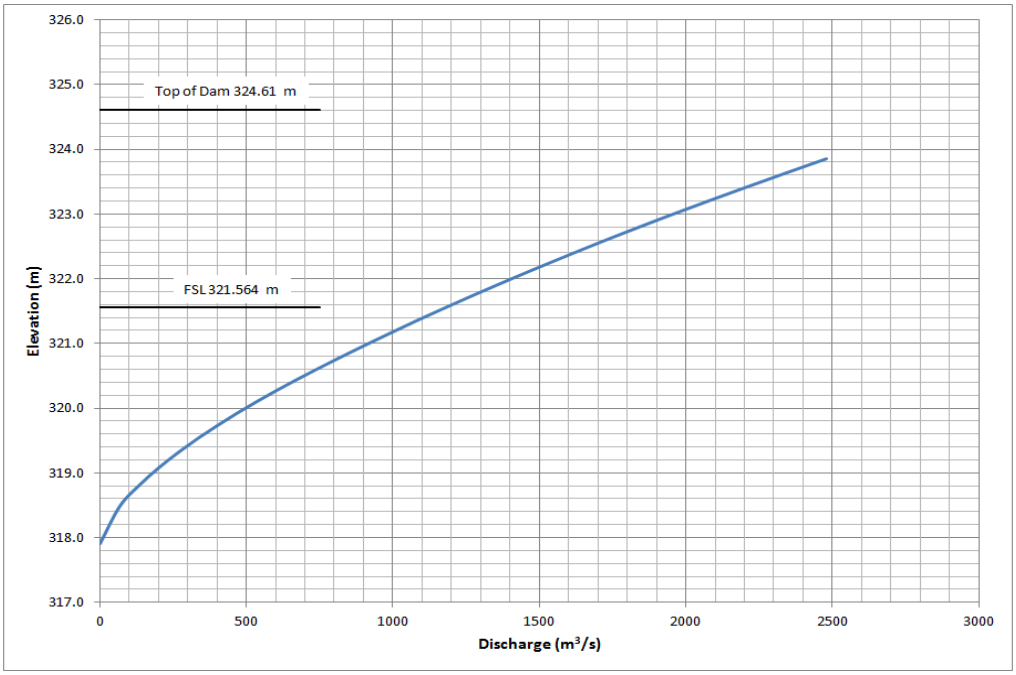
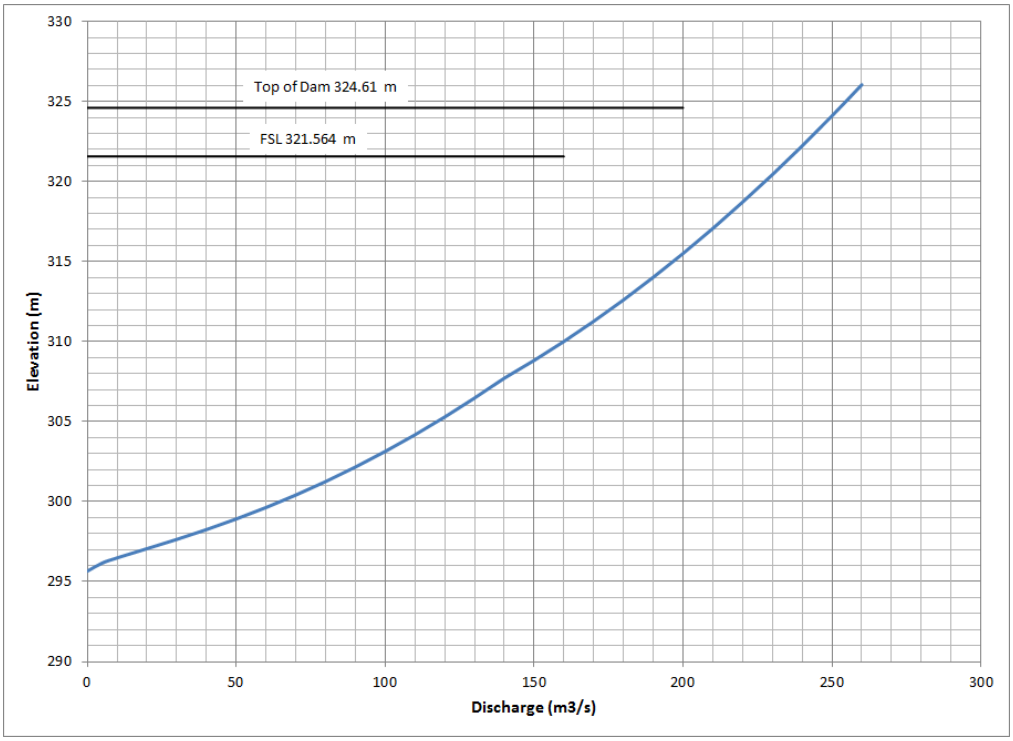


Figure 9.15
Low Level Conduit Rating Curve



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.3 Alexander Dam

9.5.3.1 Background

The preliminary design and cost estimates for the Alexander Dam were conducted by the Manitoba Water Resources Branch as a sector of the SNBB in 1969. The project was then re-visited by the SNBB and documented in their 1972 report (SNBB, 1972). The dam would be located on the Assiniboine River about 15 km (10 mi) upstream of Brandon, immediately downstream of the confluence of the Little Saskatchewan River (formerly known as the Minnedosa River) and the Assiniboine River.

At the proposed site, the Assiniboine River flows primarily eastward. The channel is well defined, about 60 m (200 ft) wide and 4.5 m (15 ft) deep. The flood plain is about 1,065 m (3,500 ft) wide at the proposed location of the dam. The elevation of the floodplain at the site is about EL 365 m (El 1,190 ft). The average slope of the river valley in the area is 0.5 m/km (1 ft/mi). The valley walls on either side rise about 53 m (175 ft) above the flood plain.

The reservoir would extend upstream for a distance of 105 km (65 mi) and inundate nearly 26,000 ha (64,150 ac) at its maximum water level. The flooded lands are used extensively for agriculture, and would also include a significant portion of the first nation community of Sioux Valley.

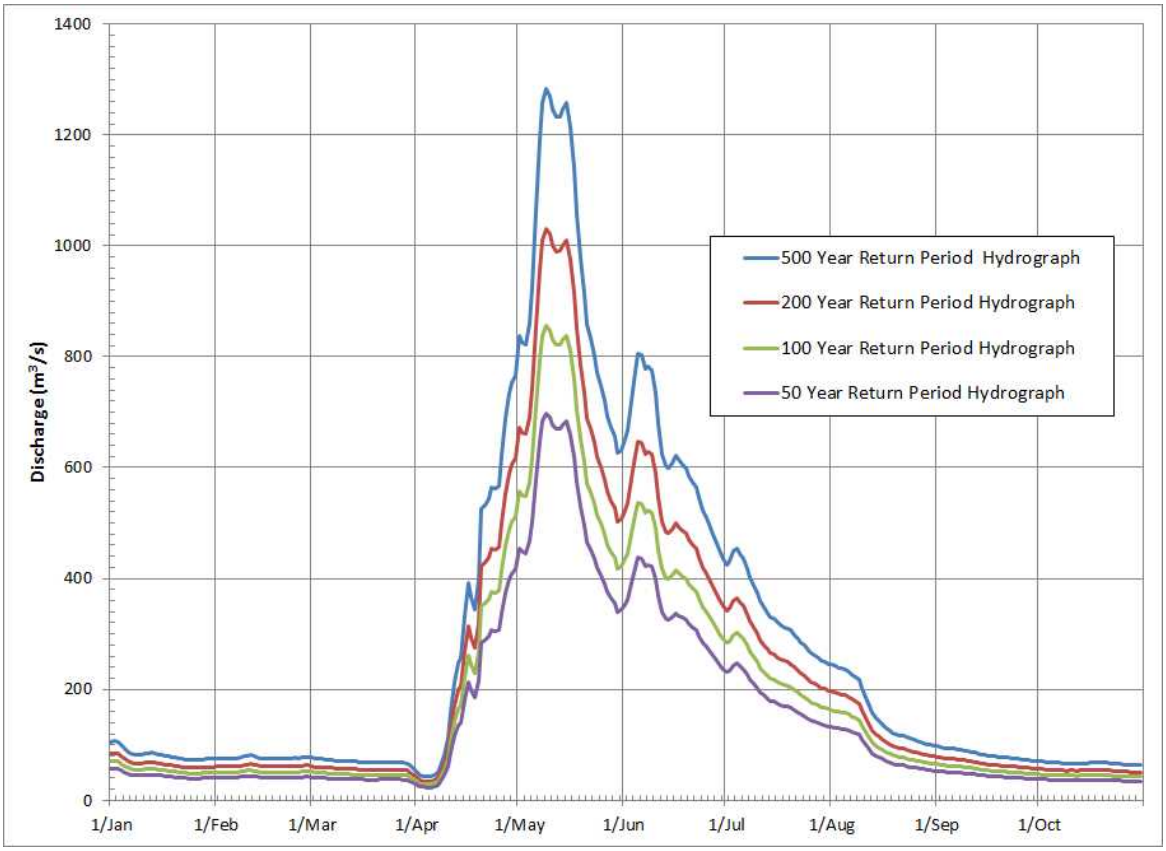
Two PTH highways, 83 and 21 cross the reservoir. PTH 83 would have to be raised and PTH 21 would have to be relocated. PR 259 would also have to be raised. Many other existing roads and crossings would be affected such as PR 254 and PR 250. These would have to be relocated or abandoned.

9.5.3.2 Hydrology

Due to the proximity of the dam site to Brandon, design hydrographs developed for Brandon would apply to the project. Design hydrographs for the Assiniboine River at Brandon have been computed based on discharge frequency curves for the Assiniboine River at Brandon provided by MIT and volume frequency curves prepared for this study, as documented in Section 5.4.

The design hydrographs for return periods from 50 years to 500 years are shown on Figure 9.16.

Figure 9.16
Design Hydrographs – Alexander Dam



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.3.3 Hydraulics

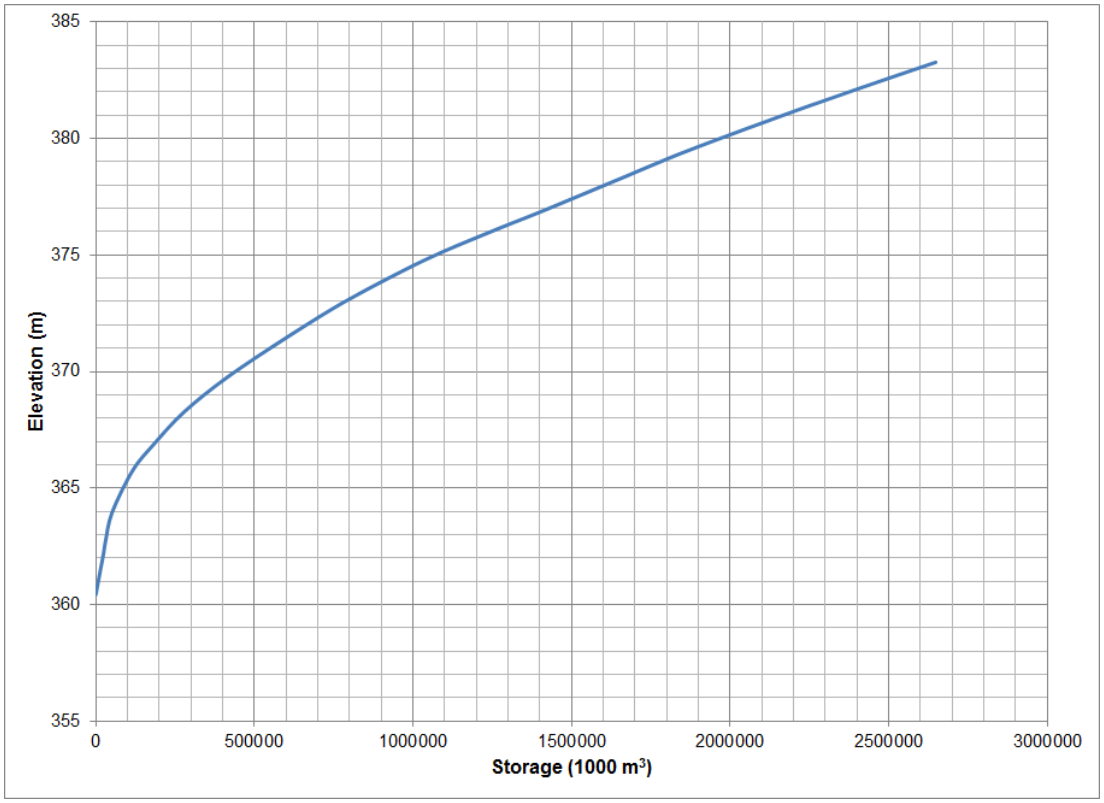
Three reservoir levels of the dam were examined with FSL at El 371.86 m (El 1,220 ft), El 374.90 m (1,230 ft), and El 377.95 m (El 1,240 ft). The full supply level of El 377.95 m (El 1,240 ft) was selected for flood control purposes. At this level, the reservoir would have 1,596,000,000 m^3 (1,310,000 ac-ft) of storage and flood over 20,200 ha (50,000 acres). At a maximum water level of El 382.22 m (El 1,254 ft), the flooded area would be nearly 26,000 ha (64,150 ac). The reservoir would extend upstream about 105 km (65 mi).

The earth embankment across the floodplain would average 21 m (70 ft) high. The top of the dam would be at elevation 385 m (1,262 ft) and would extend across the valley about 1,400 m (4,600 ft). A 7-bay, 91 m (300 ft) wide gated concrete chute spillway would be located in the embankment. That option was chosen because of poor foundation conditions in the abutments on either end of the embankment. The spillway design flood is the 1:500 year flood having a peak inflow of 1,282.75 m^3/s (45,300 cfs).

The riparian outlet would be a concrete horseshoe-shaped conduit, sized to pass 244 m^3/s (8,600 cfs) at the FSL, which is the approximate mean annual flood and bank full capacity downstream.

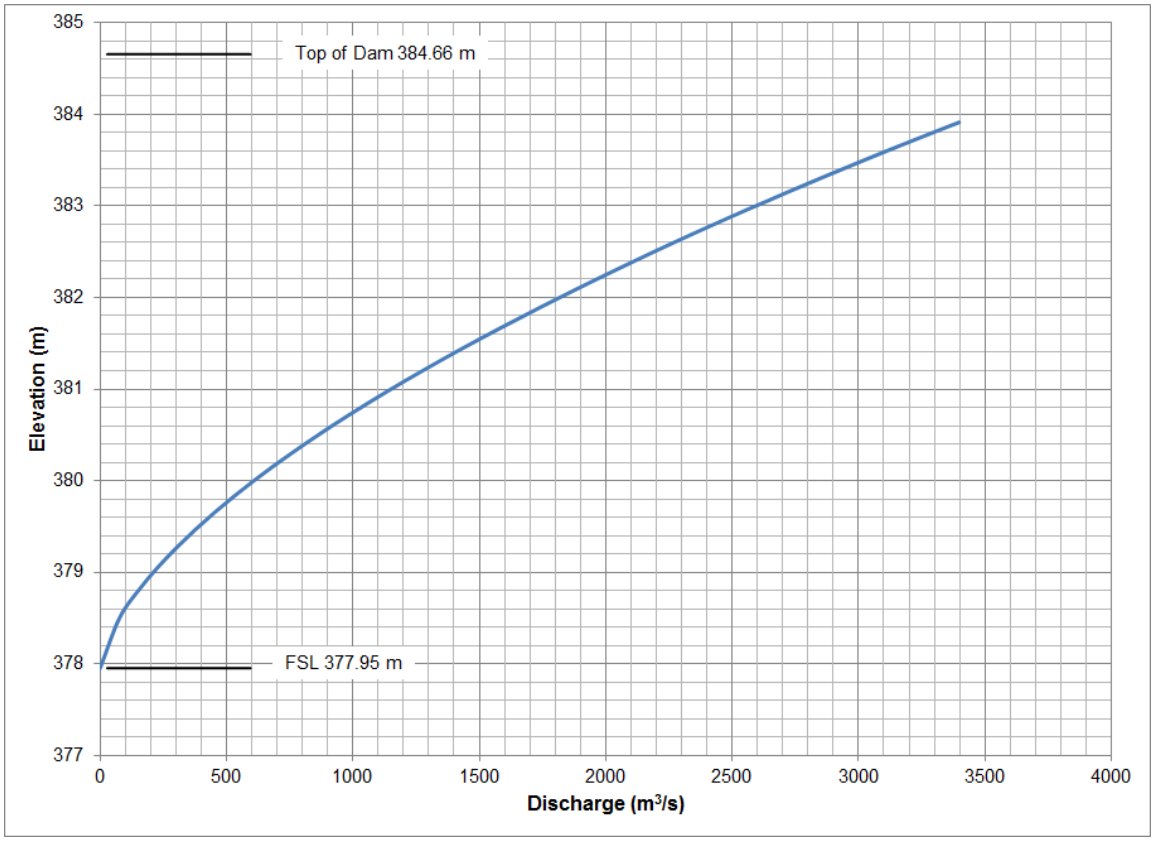
The reservoir stage-storage curve shown on Figure 9.17 was taken from the 1969 report by Manitoba Water Control and Conservation Branch, Department of Mines and Natural Resources (Manitoba, 1969).

Figure 9.17
Alexander Dam Stage-Storage Curve



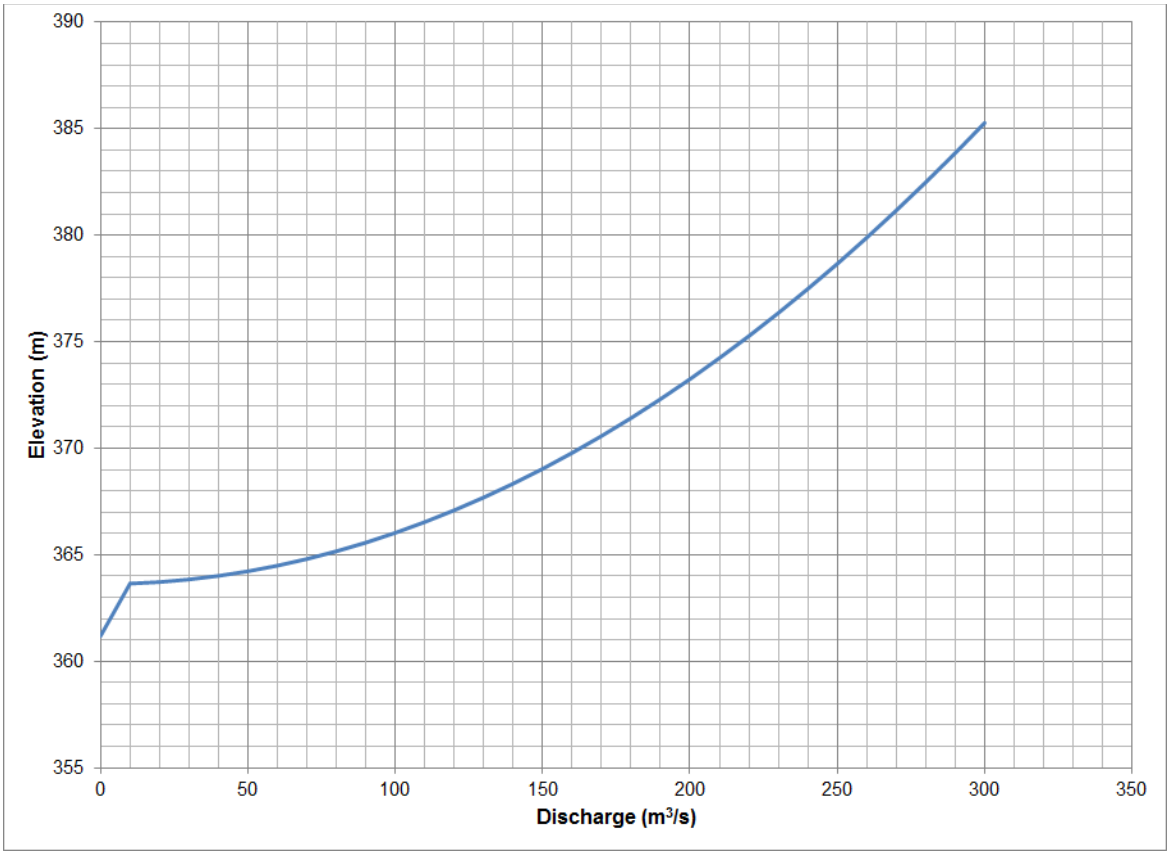
The stage-discharge rating curves for the overflow spillway and low level conduit are shown in Figure 9.18 and Figure 9.19, respectively.

Figure 9.18
Alexander Dam Overflow Spillway Stage-Discharge Curve



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 9.19
Alexander Dam Low Level Conduit Spillway Rating Curve



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.4 Zelena Dam

9.5.4.1 Background

In 1952, PFRA conducted a study to investigate the possibility of flood control and water supply along the Assiniboine River by storing flood water in the upper portion of the basin including tributaries. At that time, PFRA concluded that the Shell River was unsuitable for a dam because of its steep gradient (they cited eight feet per mile), and that the runoff from the Shell River contributed a relatively small percentage of the total runoff to the Assiniboine.

In 1970, the Manitoba Water Resources Branch, on behalf of the SNBB, revisited the Shell River idea with a view toward further regulating flows on the Assiniboine River. The results of this study were incorporated by SNBB in their 1972 report (SNBB, 1972), which has formed the

basis of the analysis for the current study. The selected site on the Shell River was about 20 km (12.4 mi) north of Roblin, Manitoba. Three reservoir sizes were investigated: 156,000,000 m^3 (125,000 ac-ft – the low dam), 199,000,000 m^3 (162,000 ac-ft) and 247,000,000 m^3 (200,000 ac-ft – the high dam). The high dam was selected for this study.

The Shell River Valley in the vicinity of the proposed dam is about 610 m (2,000 ft) wide, widening to about 790 m (2,592 ft) at the top, with fairly steep valley sides. Maximum relief between the stream bed and the upland is about 55 m (180.4 ft) on the north side of the valley and about 37 m (121.4 ft) on the south. Through the proposed site, the river channel is well-defined and about 15 m (49 ft) wide.

The reservoir would extend upstream about 24 km (14.9 mi). At the full supply level, the reservoir would flood about 1,800 ha (4,450 ac). At the time of the 1970 study, the flooded area for each reservoir was about an even split among cultivated acreage, hay and brush, and wooded areas.

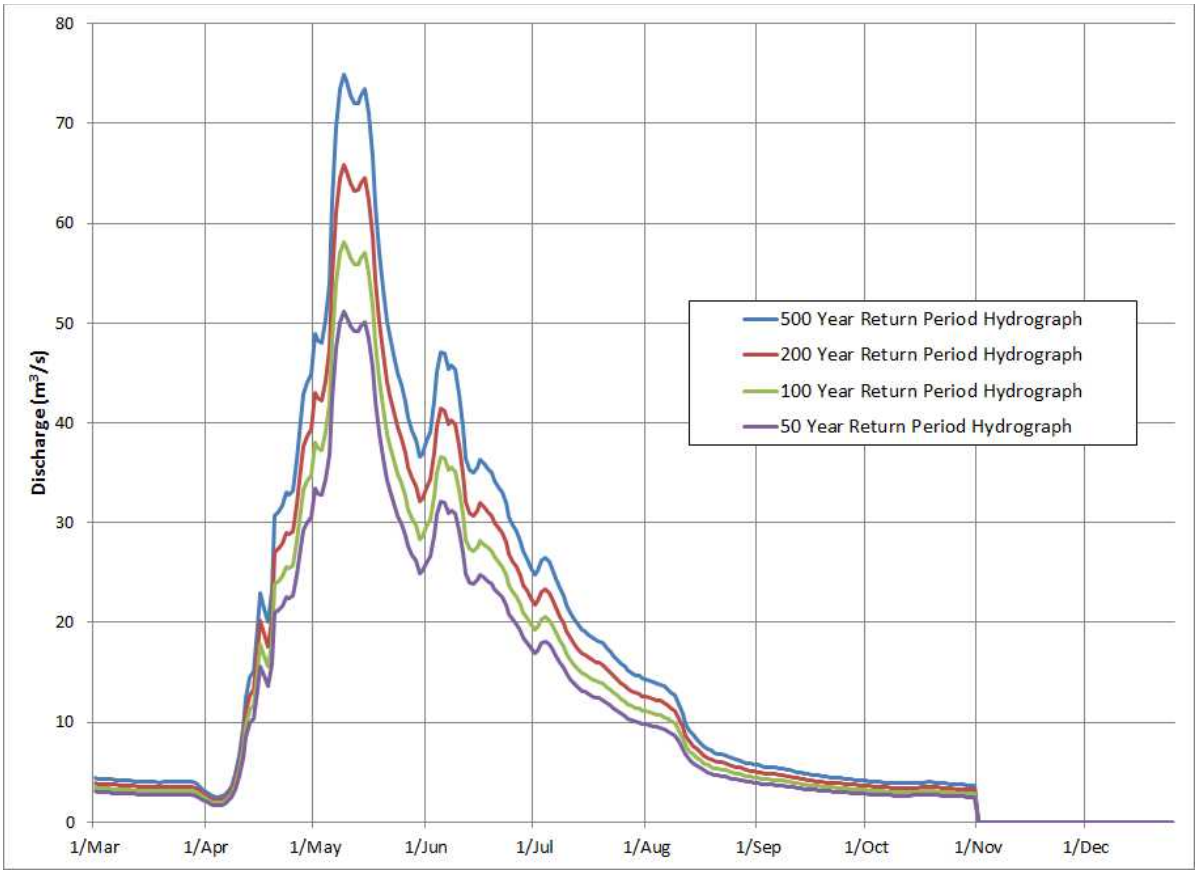
9.5.4.2 Hydrology

The Shell River rises in the Duck Mountain Provincial Forest and flows generally southward to empty into Lake of the Prairies (Shellmouth Reservoir) just upstream of the Shellmouth Dam. There are no major streams draining into the Shell River.

According to Water Survey of Canada, the drainage area above the proposed dam is about 1,300 km^2 (500 mi^2). The median annual runoff is 60,700,000 m^3 (49,000 ac-ft). The mean flow on the river as gauged at Station 05MD007, Shell River near Roblin (about 5 km (3.1 mi) south of the proposed dam) is about 1.7 m^3/s (60 cfs).

Design hydrographs for the Shell River at the Zelena Dam have been computed based on discharge frequency curves for the Shell River at Inglis provided by MIT. The design hydrographs at Brandon, discussed in Section 5.4, were used to define the shape of the hydrographs. The discharges were then scaled to the Zelena Dam site based on a frequency analysis of the Shell River flows at Inglis provided by MIT, and the ratio of the drainage areas at Zelena to the drainage area of the Shell River at Inglis. The design hydrographs for return periods from 50 years to 500 years are shown on Figure 9.20.

Figure 9.20
Design Hydrographs – Zelena Dam



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

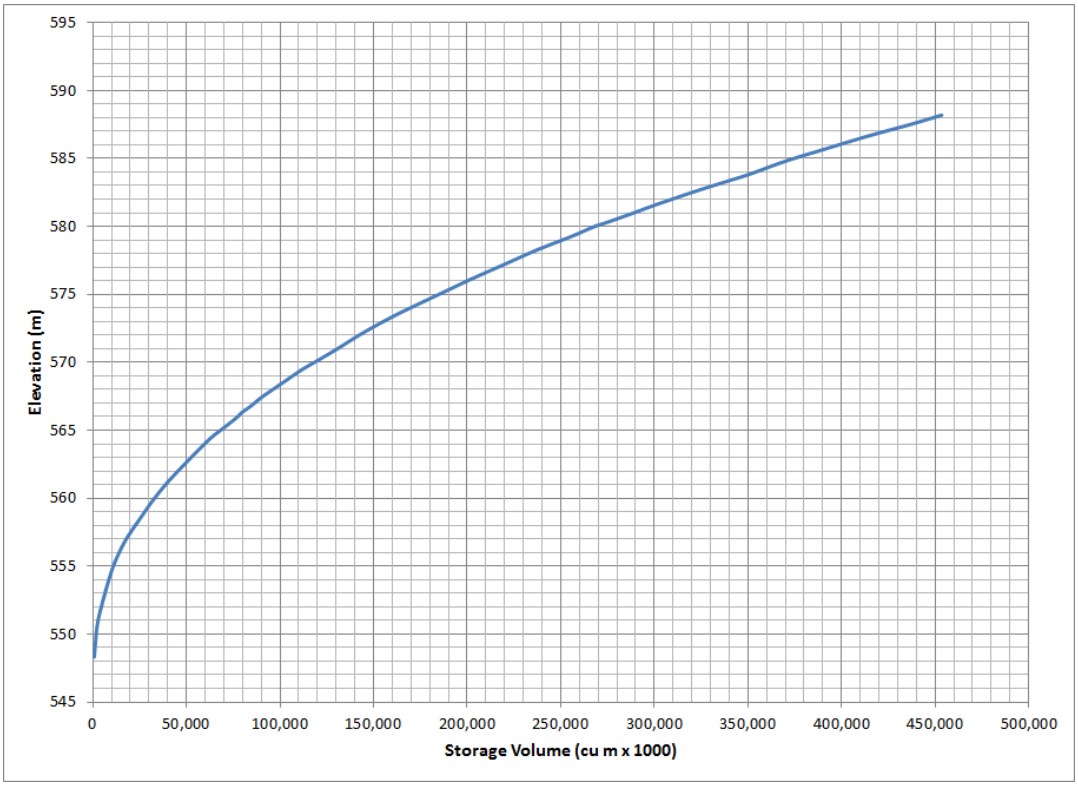
The method used to define the shape of the hydrograph for the Shell River assumes that the Shell River Basin and the Assiniboine River Basin upstream of Brandon have very similar drainage characteristics. The method was selected for its simplicity and to be consistent with the analyses of other reservoirs conducted for the study. It is understood that in reality the Shell River Basin is much smaller and that its typical hydrograph can vary from the shape of the Brandon hydrograph that was adopted for the analysis. However, since the peaks and volumes of the Shell River are very small compared to those of the Assiniboine River in Brandon and, in Holland where the hydraulic benefits of the storage reservoirs are being computed, a change to the hydrographs shown on Figure 9.20 would be negligible when evaluating the relative impact of a storage reservoir to downstream flows. The hydrographs shown on Figure 9.20 were, therefore, adopted for this conceptual level of analysis. A more detailed approach, such as developing a unit hydrograph based on historic flood events of the Shell River, could be undertaken for the Zelena Dam should it be selected to proceed to a subsequent stage of study.

9.5.4.3 Hydraulics

The embankment would be a zoned earthfill structure with a concrete spillway designed to pass a 1:500 year flood. Conduits would be designed to discharge 4.25 m^3/s (150 cfs) at the FSL of El 579 m (El 1,899 ft). The crest length for the dam would be 855 m (2,800 ft) with a maximum height of 37 metres (120 ft). The dam would provide a total storage capacity of 247,000,000 m^3 (200,000 ac-ft).

The reservoir stage-storage curve shown on Figure 9.21 was taken from the SNBB report (1972).

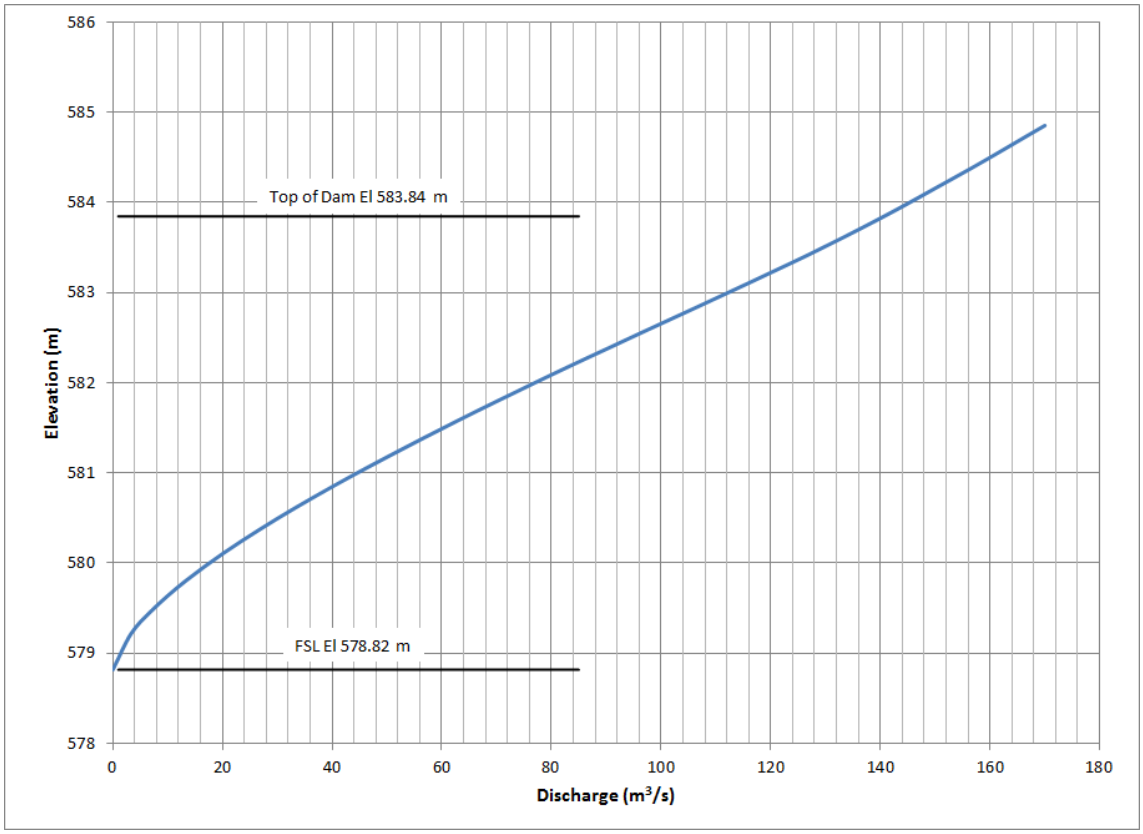
Figure 9.21
Zelena Dam Reservoir Storage Curve



The spillway would be comprised of a gated 7.62 m (25 ft) wide concrete overflow spillway and a low level riparian flow conduit having a diameter of 0.75 m (2.45 ft).

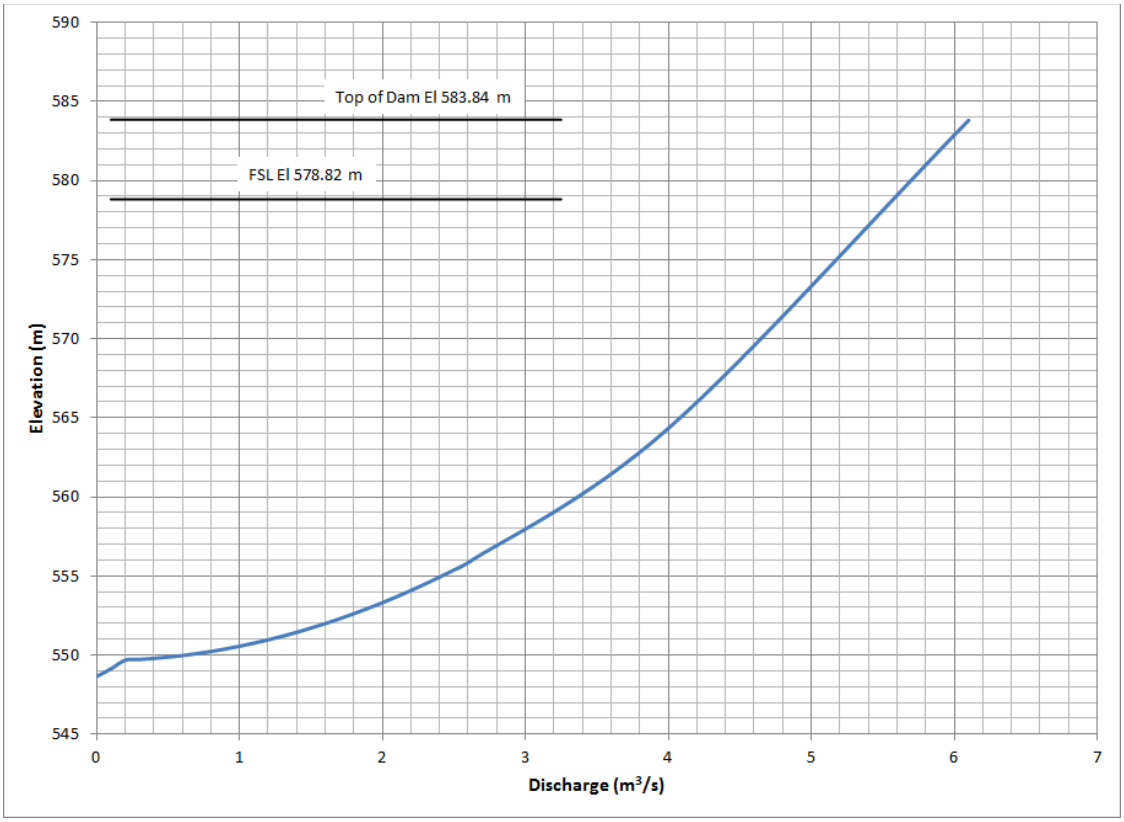
Figures 9.22 and 9.23 illustrate the rating curves for the overflow spillway and the low level riparian conduit.

Figure 9.22
Overflow Spillway Rating Curve – Zelena Dam



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 9.23
Low Level Riparian Conduit – Zelena Dam



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.5 Victor Dam

9.5.5.1 Background

As early as the 1950s, consideration was being given to constructing a major flood storage reservoir somewhere in the lower Qu'Appelle River Valley. In 1964, PFRA conducted a preliminary review of a possible site on the river about 8 km (5 mi) west of the Manitoba / Saskatchewan border, and west of the community of St. Lazare, Manitoba. In 1966, a study of the Victor Dam was included in a larger study of water use and supply in the Qu'Appelle and Assiniboine basins. Preliminary design for the dam was conducted by PFRA in 1970 as part of the SNBB studies. The results of this study were incorporated by SNBB in their 1972 report (SNBB, 1972), which has formed the basis of the analysis for the current study.

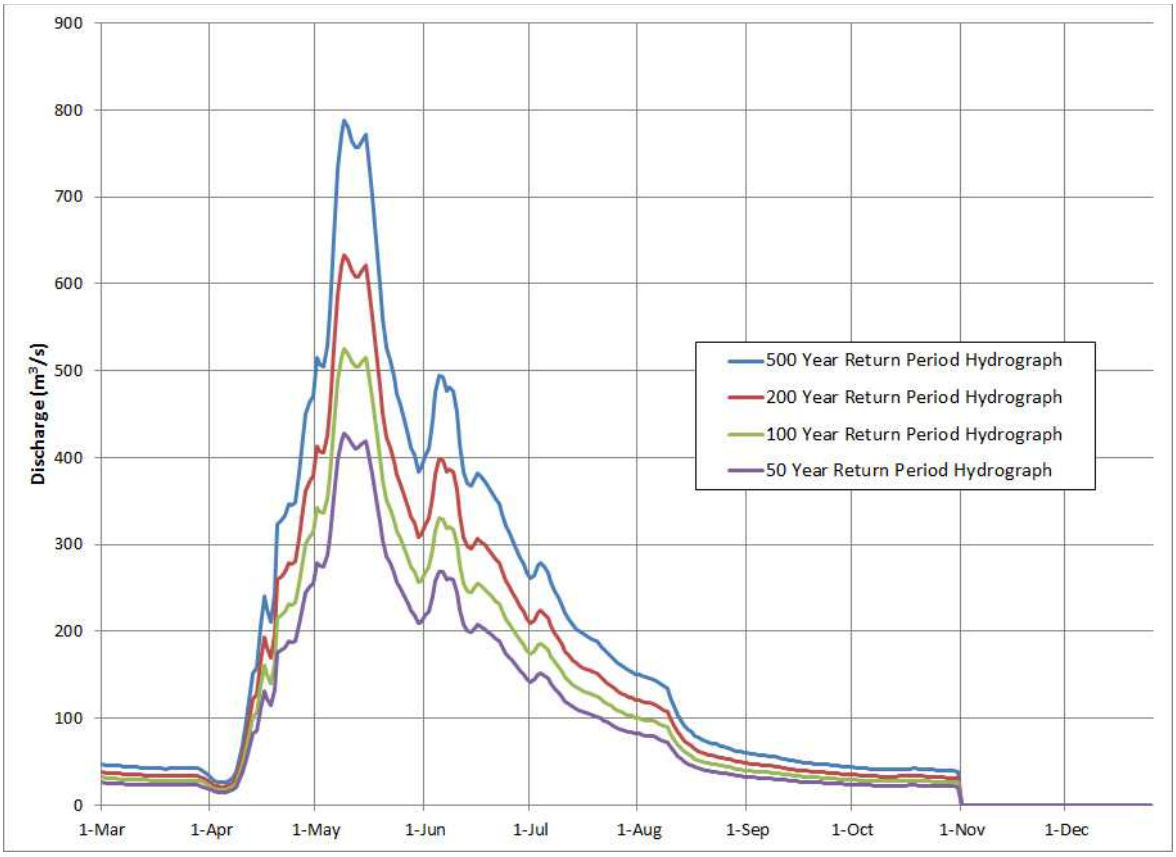
Further investigations that are beyond this conceptual phase analysis are necessary to determine whether or not the Victor Dam could be constructed in Manitoba versus in Saskatchewan and whether inter-jurisdictional issues would allow the project to proceed. Regardless of the exact location, the entire flooded area would be located in Saskatchewan. The reservoir would extend 24 km (15 mi) upstream and flood an area of approximately 2,000 ha (4,900 ac). As a result, the upstream community of Tantallon would require diking to protect against flooding. Highway 8, as well as municipal roads and a railway crossing, would have to be raised. However, the Victor Dam is being considered for this study as it is the only flood mitigation alternative that could potentially be located on the Qu'Appelle River in Manitoba to provide benefits downstream on the Assiniboine River.

9.5.5.2 Hydrology

The Qu'Appelle River watershed is almost entirely in Saskatchewan, except for a small portion between the Saskatchewan / Manitoba border and the point where the Qu'Appelle empties into the Assiniboine River at St. Lazare. At the proposed site, the valley is slightly under a kilometer wide with well-defined valley walls. The river is strongly meandering with many oxbows and meander cut-offs. The stream gradient is shallow.

Design hydrographs for tributaries and for main stem locations on the Assiniboine River were determined by prorating the design hydrographs for Brandon discussed in Section 5.4, to the drainage areas of the tributaries. The design hydrographs for the return periods from 50 years to 500 years for the Qu'Appelle River at the Victor Dam are shown on Figure 9.24.

Figure 9.24
Victor Dam – Design Discharge Hydrographs



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

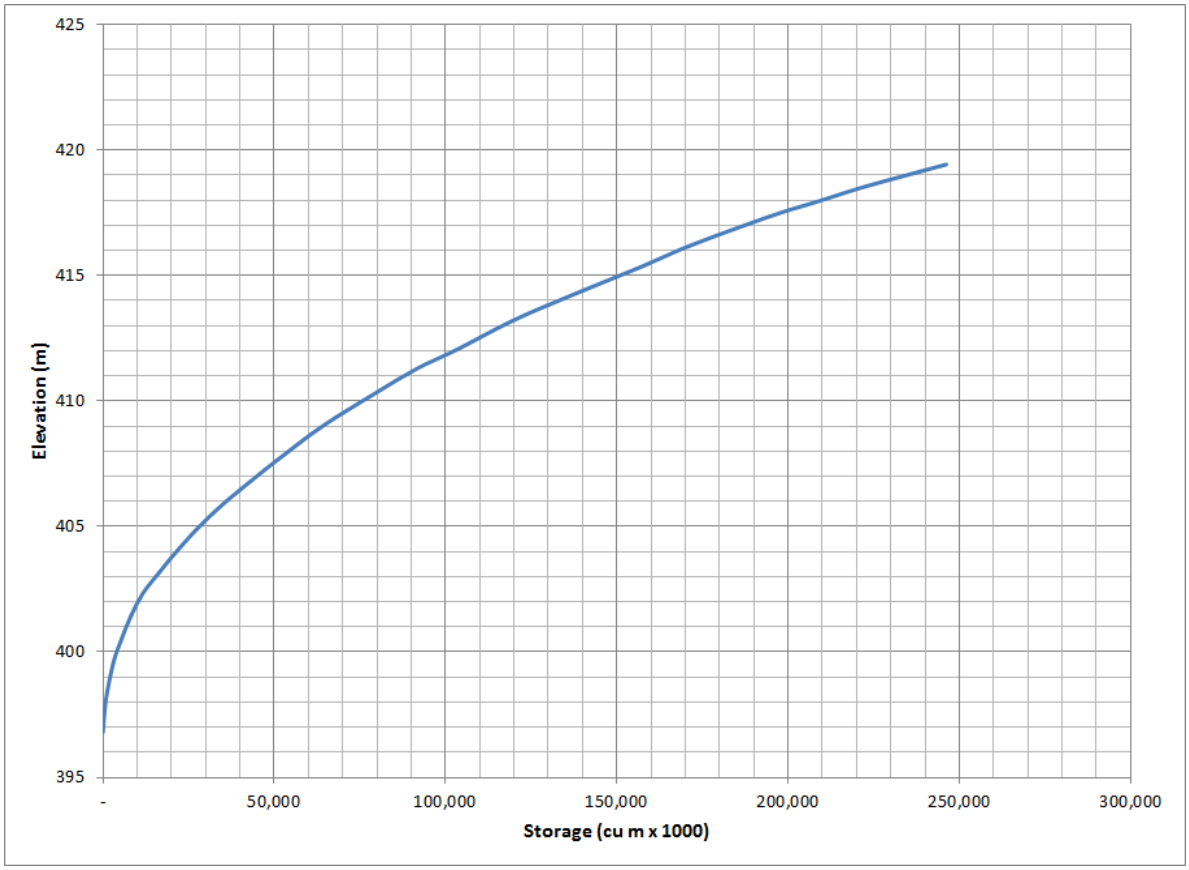
As discussed in Section 5.4, the method of prorating the hydrographs based on area assumes that inflows to the Assiniboine River are equally distributed across the entire watershed and that each unit of area contributes an equal volume of water to the Assiniboine River downstream. In reality, this is not the case on the Qu'Appelle River, and the hydrographs outlined in Figure 9.24 are conservative as the drainage area method over-estimates the flows on the Qu'Appelle River. For this reason, the hydraulic analysis for Victor Dam also considered hydrographs that were adjusted by a factor of 0.5. This is further discussed in Section 9.5.6.4.

9.5.5.3 Hydraulics

The earthfill embankment would have a crest length of about 1 km (0.6 mi) and a maximum height of 26 m (85.3 ft). The dam would store 168,900,000 m^3 (140,000 ac-ft) at a full supply

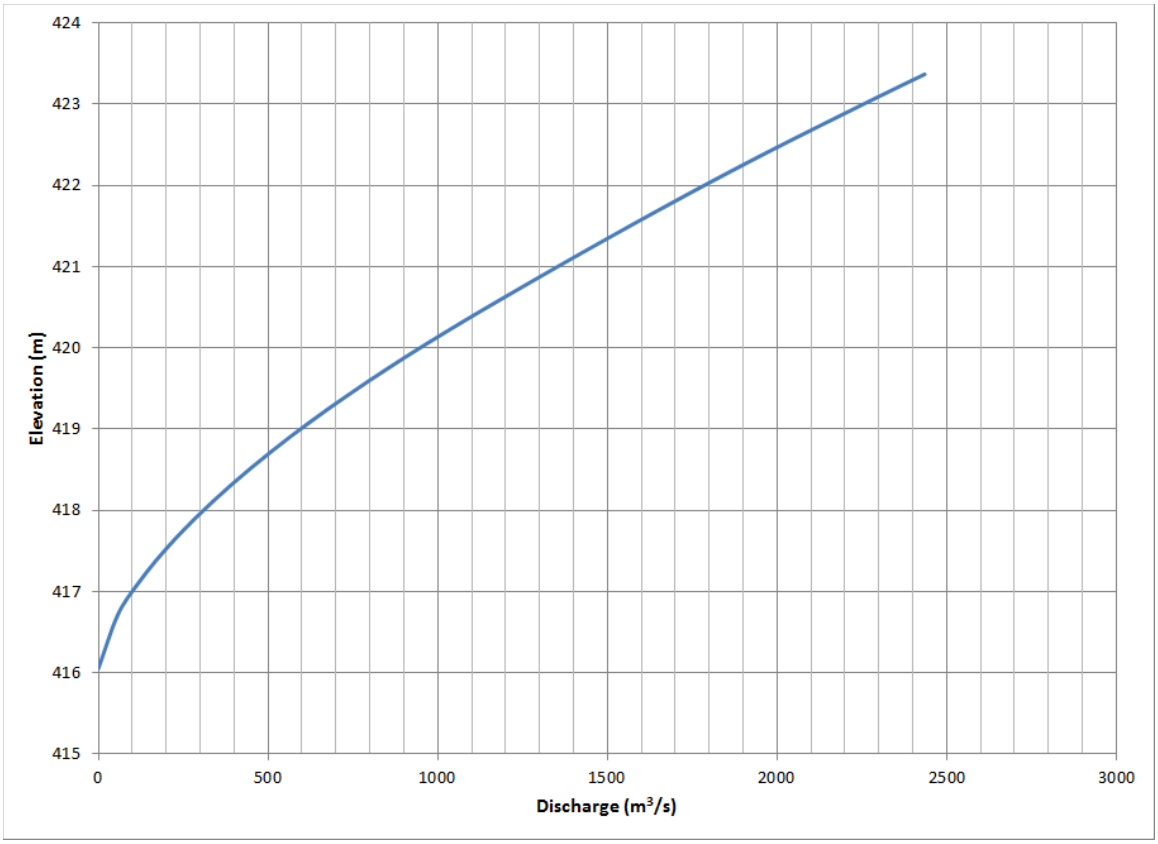
level of El 416.05 m (El 1,365 ft). The reservoir stage-storage curve shown on Figure 9.25 was taken from SNBB, 1972.

Figure 9.25
Victor Dam Storage Volume Curve



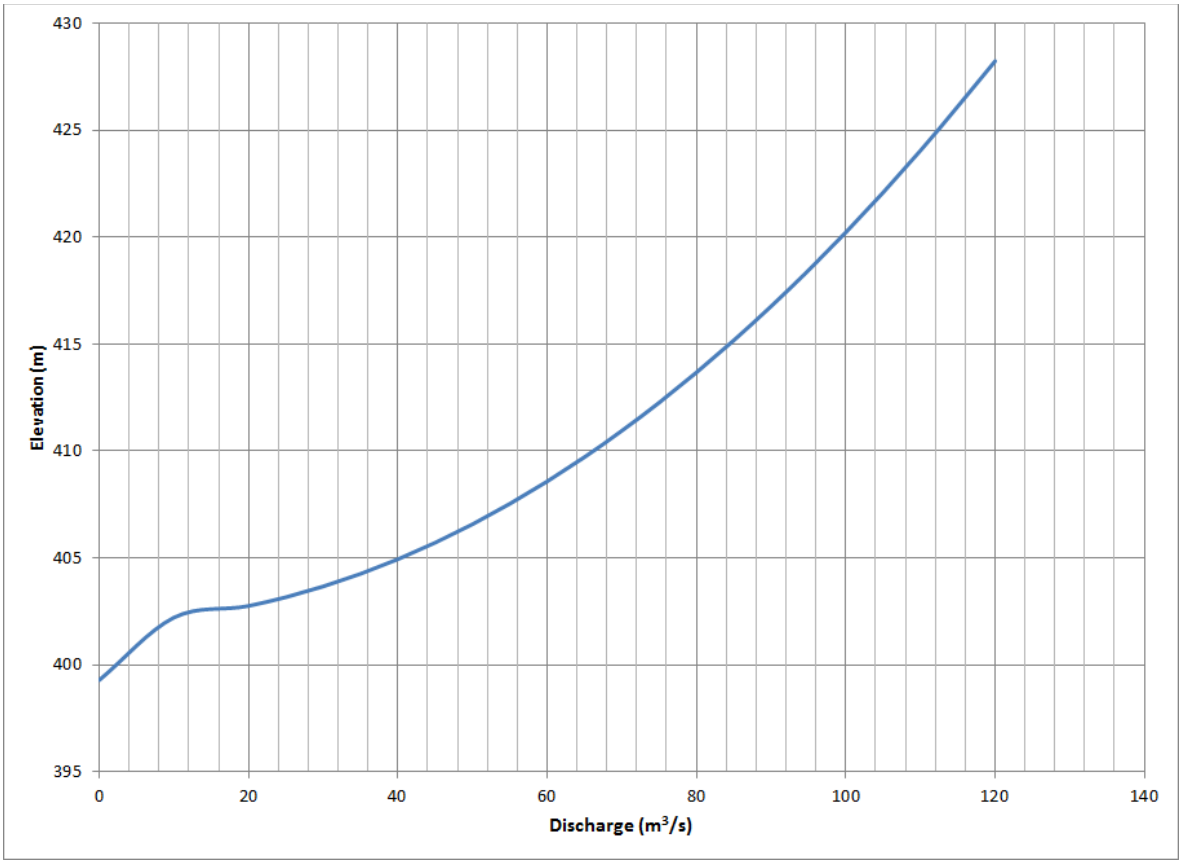
The Victor Dam spillway would consist of a 5-bay, 61 m (200 ft) wide, gated, concrete spillway, 61 m (200 ft) long, and would be used to pass the 953.7 m^3/s (33,680 cfs) inflow peak for the 500-year design flood. The Victor Dam stage discharge rating curve is shown on Figure 9.26.

Figure 9.26
Overflow Spillway Rating Curve – Victor Dam



The riparian outlet would be a concrete horseshoe-shaped conduit designed to pass flows of up to 87.8 m^3/s (3,100 cfs) at the FSL El 416.05 m (El 1,365 ft). The stage-discharge rating curve is shown on Figure 9.27.

Figure 9.27
Low Level Conduit Rating Curve – Victor Dam



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.6 Model Results

9.5.6.1 Holland Dam

Results at Holland Dam

The design hydrographs for the Assiniboine River at Holland, for return periods of 1:50 years to 1:500 years, were routed through the Holland Dam Reservoir. Reservoir routing was performed using a level pool routing program and applying the outlet rating curves and storage curves described in Section 9.5.2.3. Two modes of operation for the reservoir were implemented as described below:

1. The spillway vertical lift gates were opened fully.

2. The gates were operated so that the water level would rise to the maximum permissible with appropriate freeboard.

Contrary to the results of the analysis on the Impact of Operation of the Shellmouth Dam Gates on Major Floods (Section 9.2), adding gates to the Holland Dam has the ability to reduce peak flows for major floods. This is because the spillway capacity of the Holland Dam relative to the reservoir capacity is much larger, which provides a greater ability to control flows during large floods. Additionally, the Holland Dam for the current study was conceptualized as a flood reduction reservoir only, whereas the Shellmouth Reservoir is also used for water supply and recreational purposes. This limits the ability to use the full capacity of the Shellmouth Reservoir for flood reduction.

To implement the gate control operation on the Holland Dam, gates were operated daily, based on the design inflow hydrographs specified for this study. In reality, this operation assumes optimal conditions and requires an accurate forecast of inflows, which is often unavailable. Therefore, the results from this mode of operation are conservative from a project viability perspective and are the most optimistic estimations of the maximum benefit that the Holland Dam can provide.

Furthermore, to achieve the maximum benefits with gate control outlined above, it is necessary that the reservoir be completely empty at the beginning of the flood. It was assumed that sediment trapping and retention within the reservoir would not significantly affect live storage. Additionally, at this conceptual phase, optimization of the reservoir outflows and levels after the flood was not incorporated into the analysis. Careful consideration would be required to determine what operating conditions would be necessary to empty the reservoir without causing significant artificial flooding downstream during the summer, fall and winter months due to increased outflows from the dams. Based on experience with the Shellmouth Dam, it is evident that a strict set of operating rules would be necessary for the Holland Dam (and any other large storage reservoir), which would limit the maximum benefits of the dam. On this basis, it was estimated based on engineering judgment that 75% of the additional maximum peak flow reduction that could be achieved with the addition of gates to the reservoir would best represent the actual peak flow reduction capability of the reservoir. Further studies beyond this conceptual phase of analysis are necessary to determine if this 75% value is achievable.

The routings of the 500-year, 200-year, 100-year and 50-year floods, under both modes of operation of the gates, are shown in Figures 9.28, 9.29, 9.30 and 9.31.

Figure 9.28
500-Year Flood Event on the Assiniboine River Routed with and without Holland Dam Gate Control

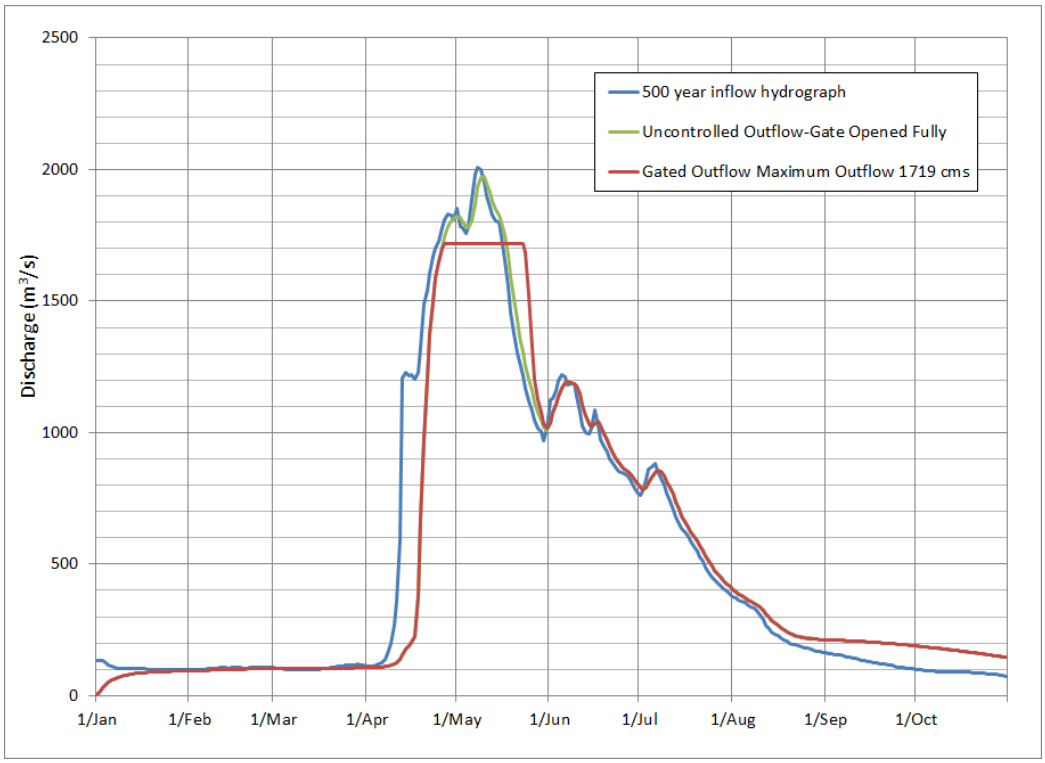


Figure 9.29
200-Year Flood Event on the Assiniboine River Routed with and without Holland Dam Gate Control

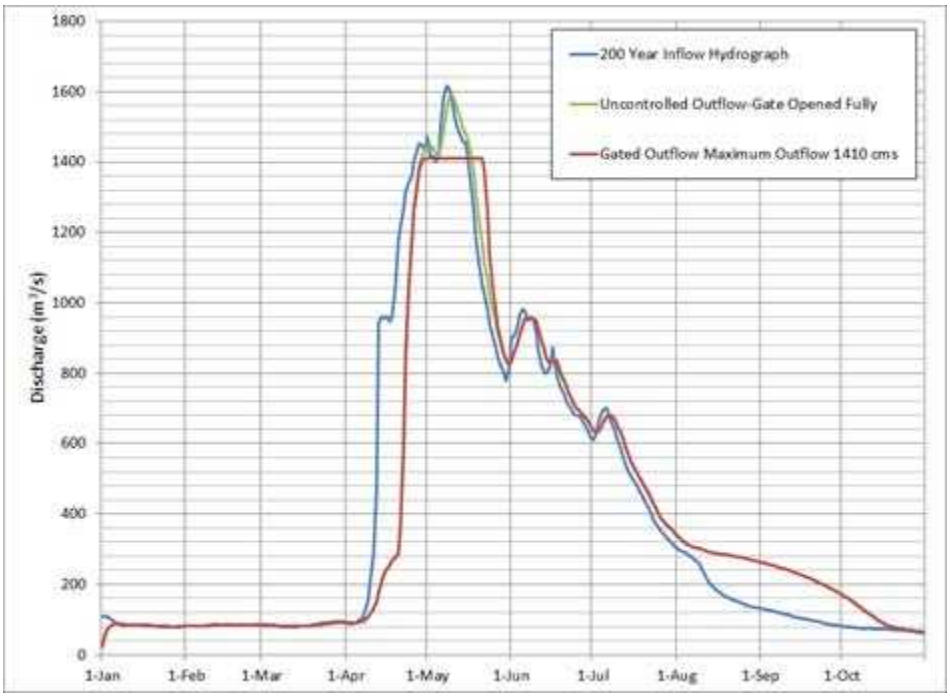


Figure 9.30
100-Year Flood Event on the Assiniboine River Routed with and without Holland Dam Gate Control

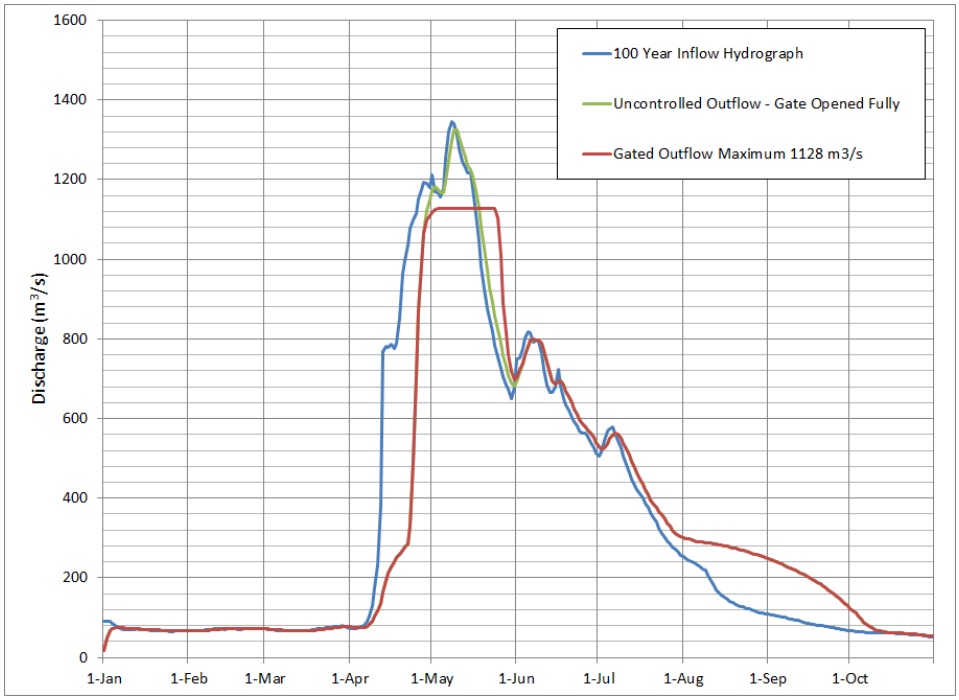
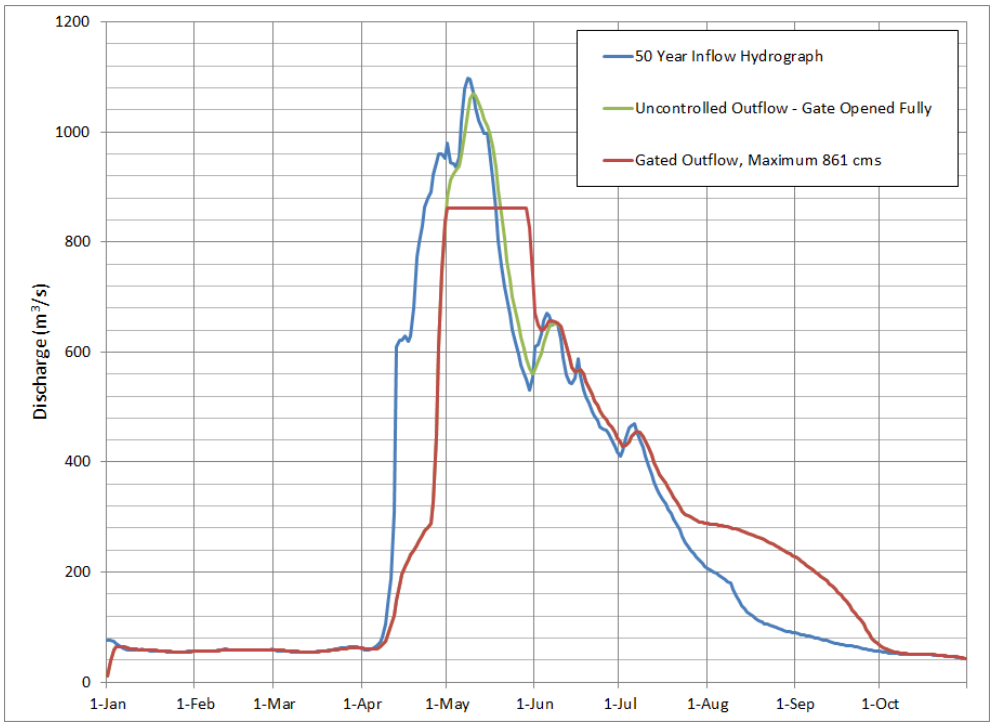


Figure 9.31
50-Year Flood Event on the Assiniboine River Routed with and without Holland Dam Gate Control



The routing of the 500-year flood through the reservoir, without outflow control, results in a 37 m^3/s (1,307 cfs) reduction of the peak outflow from the reservoir. Additional reduction in the outflow of 254 m^3/s (8,970 cfs) can be achieved if the spillway gates are operated to reduce the maximum outflow to 1,719 m^3/s (60,707 cfs), as shown in Figure 9.28.

Without gate control of outflow, the maximum water level on the reservoir would be El 322 m (El 1,056 ft) or 2.59 m (8.5 ft) below the top of the dam for the 500-year flood. With the gate control, the maximum level on the reservoir is at elevation 324 m (1,063 ft) or 0.46 m (1.50 ft) below the top of the dam. This maximum is less for smaller floods. The gate control operation utilizes the extra storage of the reservoir that was otherwise not used and, therefore, increases the reduction of inflow peak. Figures 9.32 to 9.35 illustrate the effects on the reservoir water levels of the controlled outflow for the different flood events.

Figure 9.32
Water Elevation of Holland Reservoir with and without Outflow Control, under 500-Year Inflow Event

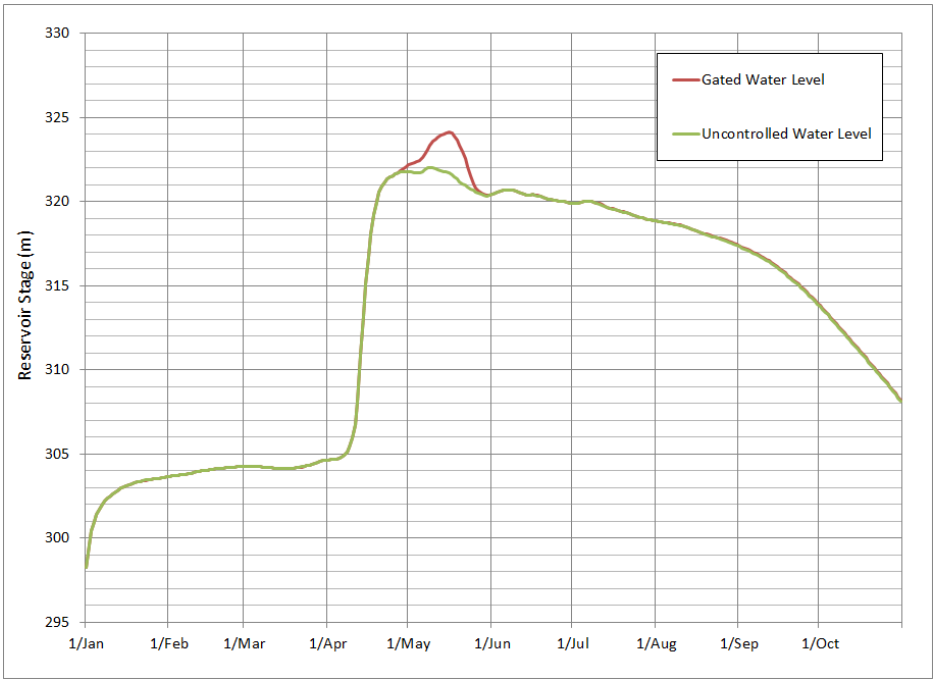


Figure 9.33
Water Elevation of Holland Reservoir with and without Outflow Control, under 200-Year Inflow Event

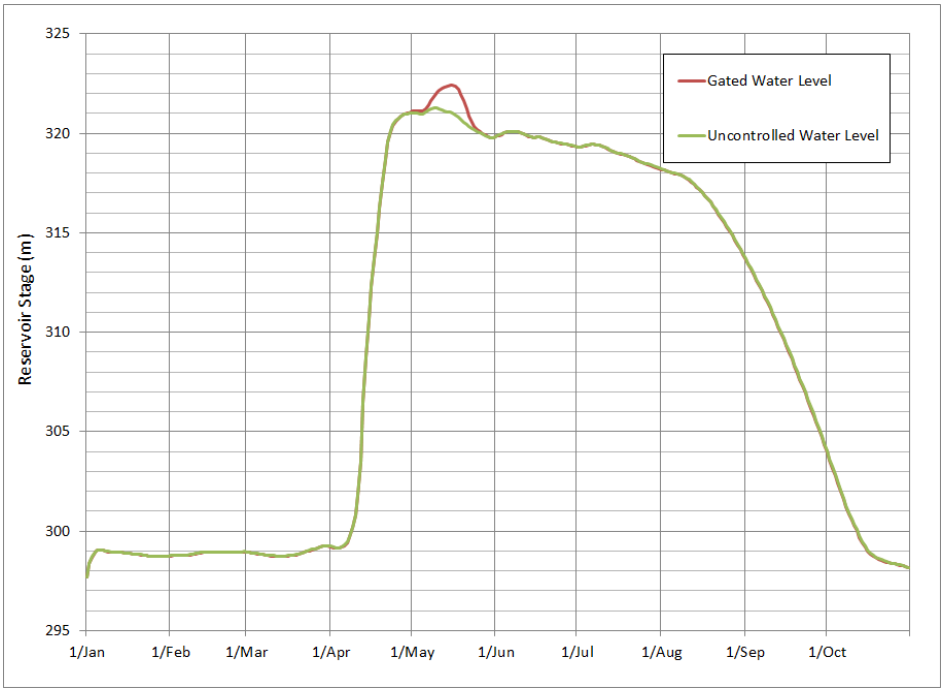


Figure 9.34
Water Elevation of Holland Reservoir with and without Outflow Control, under 100-Year Inflow Event

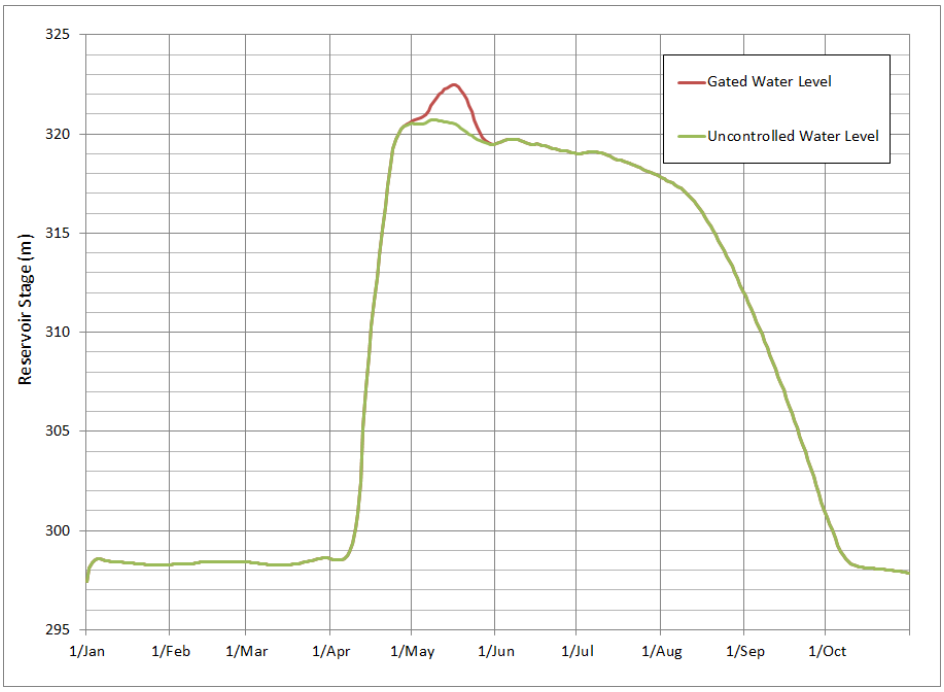
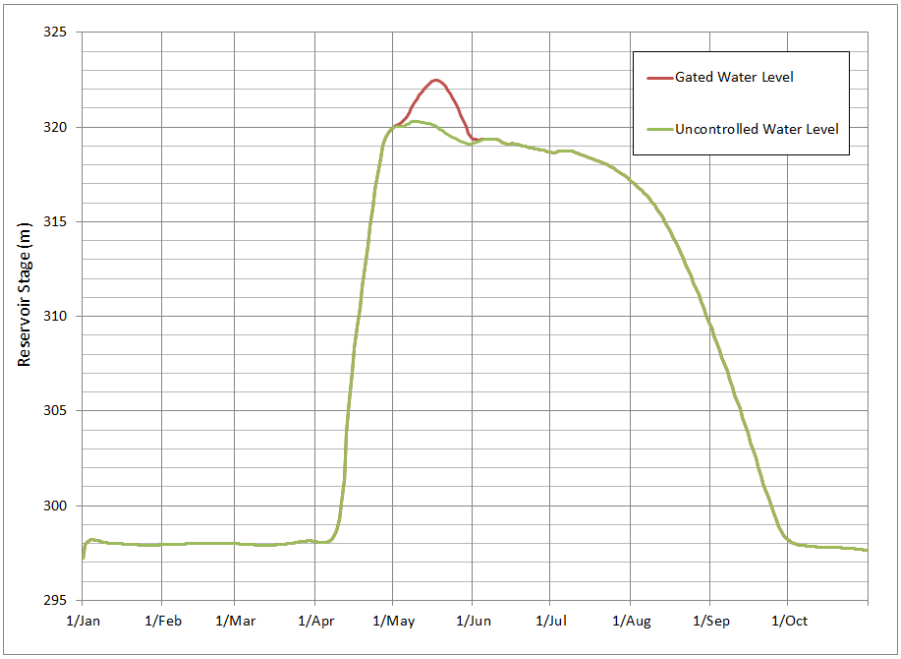


Figure 9.35
Water Elevation of Holland Reservoir with and without Outflow Control, under 50-Year Inflow Event



Results at Portage la Prairie

The location of the Holland Reservoir is in close proximity to Portage la Prairie, and there are no major inflows in the intervening distance. The discharge at Portage la Prairie was assumed to be equal to the outflow from the Holland Reservoir. No river routing was, therefore, conducted from the Holland Dam to Portage la Prairie.

Based on the results of the routings with and without the condition of gate control for the 1:50 to 1:500 year events, and considering the uncertainties that were discussed previously with being able to operate the gates to achieve the maximum benefits, the peak flow reduction capacity of the Holland Dam was estimated. The results are provided Table 9.7.

Table 9.7
Peak Flow Reduction Capacity of Holland Dam

Return Period (year)	Reduction in Peak Due to Holland Dam (m^3/s)	% Reduction in Peak Due to Holland Dam (%)
	Portage	Portage
50	185	17
100	168	12
200	169	10
500	228	11

Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.6.2 Alexander Dam

Results at the Alexander Dam

The design hydrographs for the Assiniboine River at Holland for return periods from 1:50 years to 1:500 years were routed through the Alexander Dam Reservoir. Reservoir routing was performed using a level pool routing program and applying the outlet rating curves and storage curves described in Section 9.5.3.3. Similar to that of the Holland Dam, two modes of operations were implemented (i.e., operations with and without gate control). The operation with gate control resulted in the reservoir level rising to the maximum permissible with appropriate freeboard. The freeboard used was 3.66 m (12 ft) for the 500-year flood. Figures 9.36 to 9.39

illustrate the effects of controlled gate operation on the level of the reservoir for different return periods. The routing of the 500-year, 200-year, 100-year and 50-year flood through the reservoir is shown in Figures 9.40 to 9.43.

Figure 9.36
Water Elevation of Alexander Reservoir with and without Outflow Control, under 500-Year Inflow Event

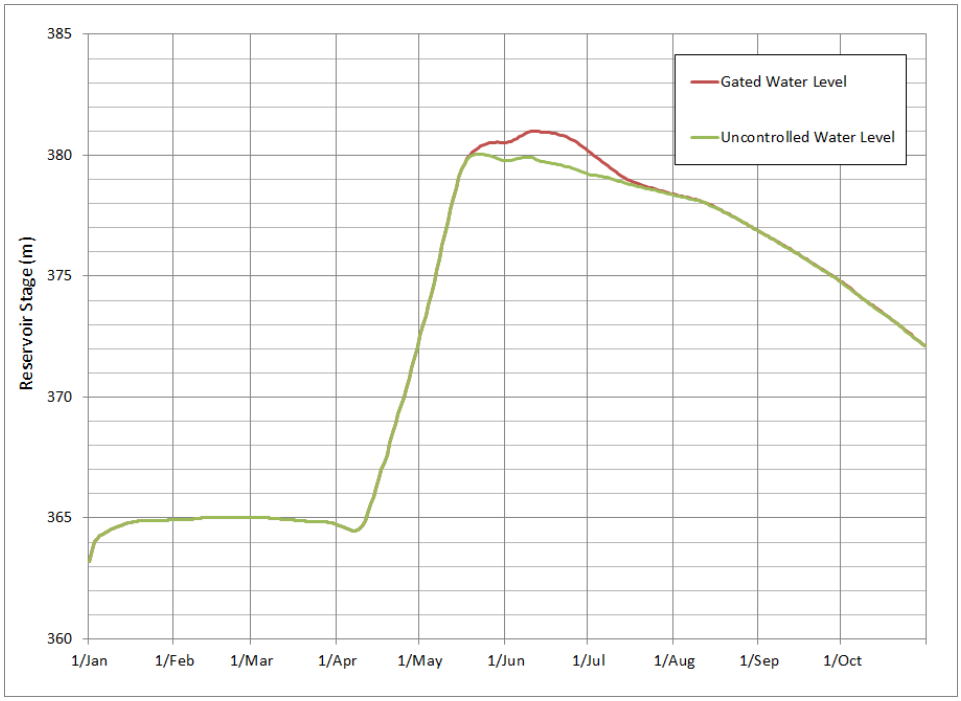


Figure 9.37
Water Elevation of Alexander Reservoir with and without Outflow Control, under 200-Year Inflow Event

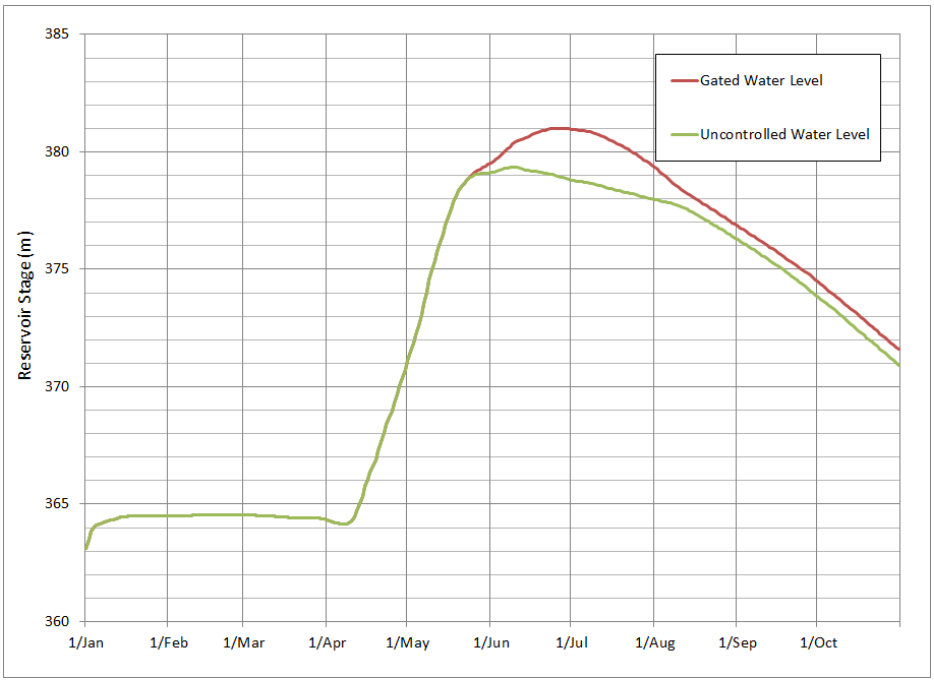


Figure 9.38
Water Elevation of Alexander Reservoir with and without Outflow Control, under 100-Year Inflow Event

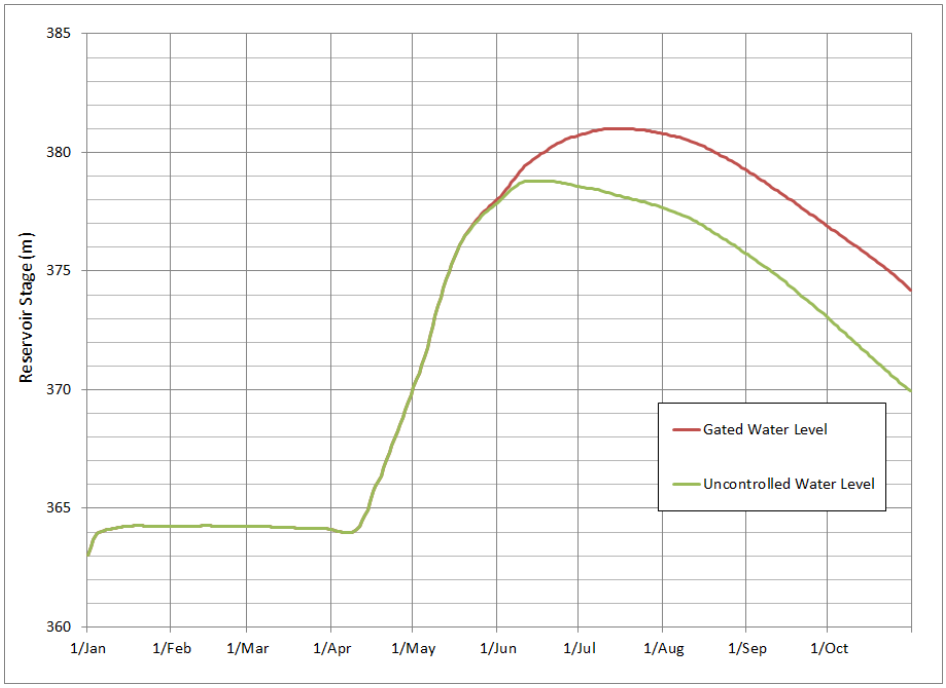


Figure 9.39
Water Elevation of Alexander Reservoir with and without Outflow Control, under 50-Year Inflow Event

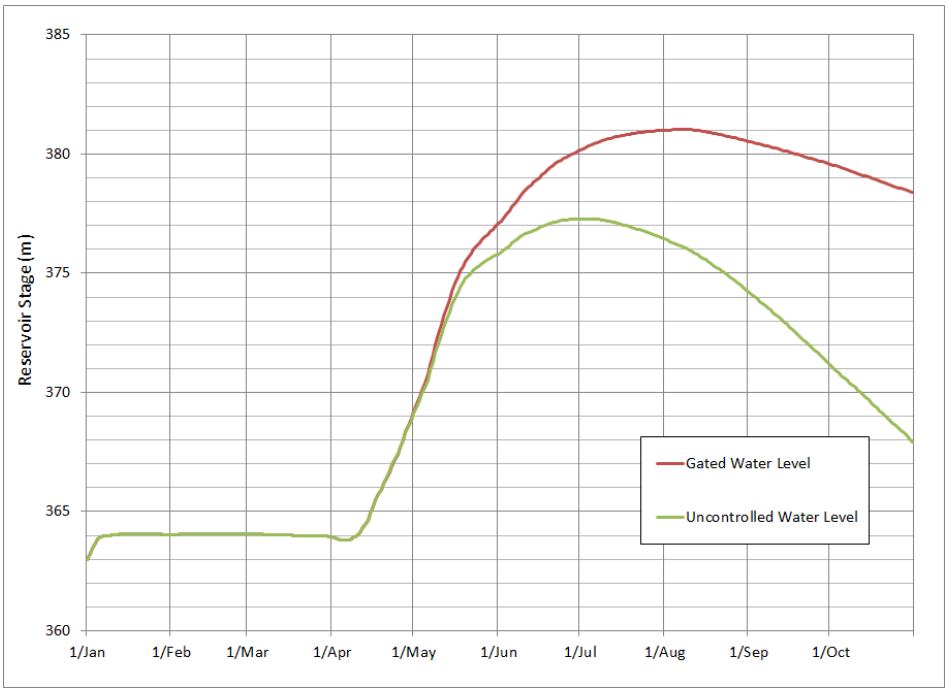


Figure 9.40
500-Year Flood Event on the Assiniboine River Routed with and without Alexander Dam Gate Control

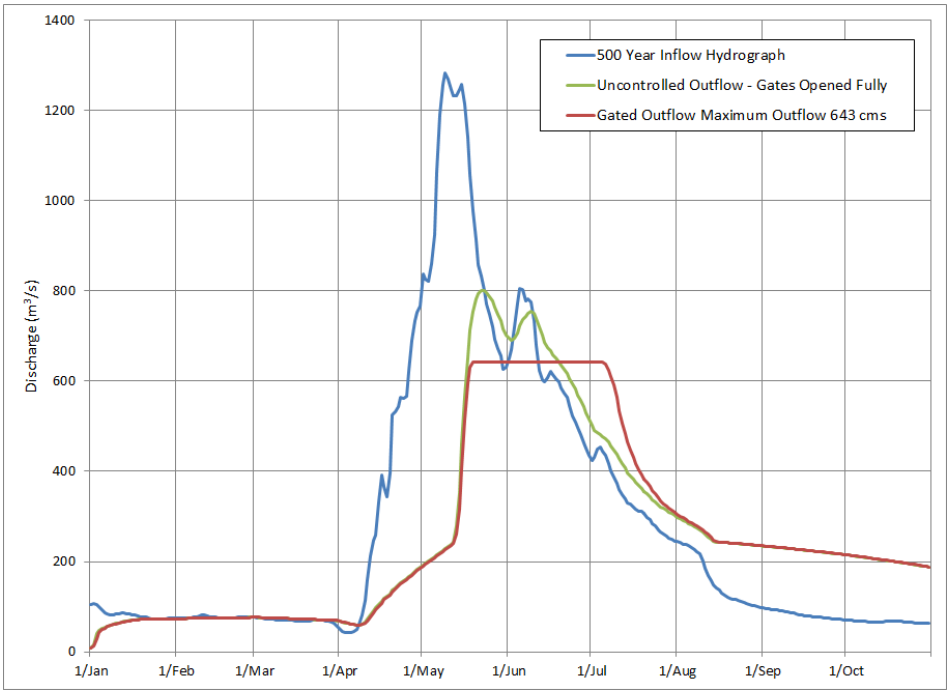


Figure 9.41
200-Year Flood Event on the Assiniboine River Routed with and without Alexander Dam Gate Control

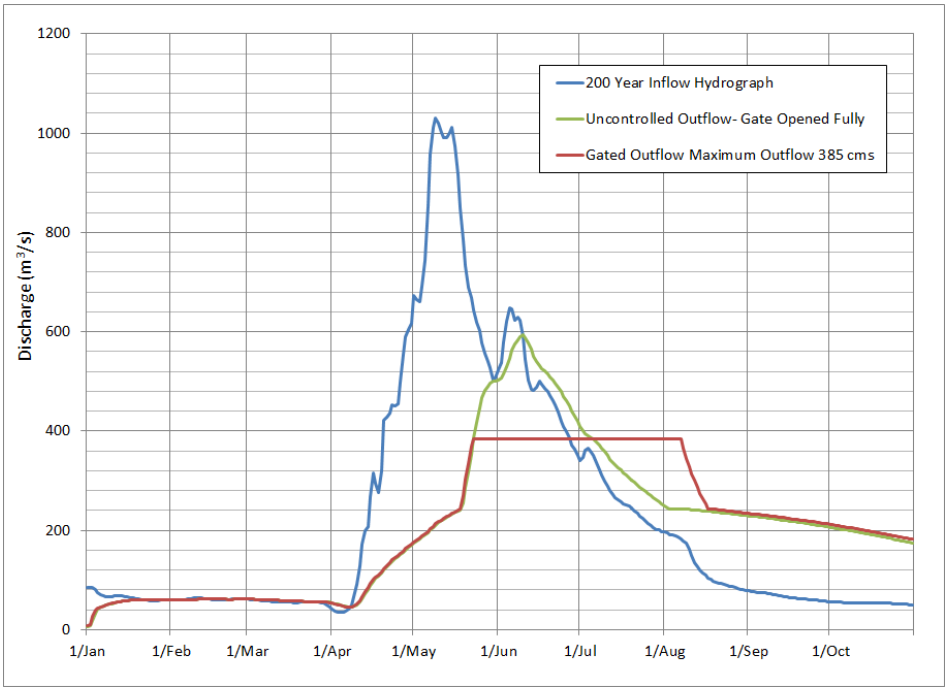


Figure 9.42
100-Year Flood Event on the Assiniboine River Routed with and without Alexander Dam Gate Control

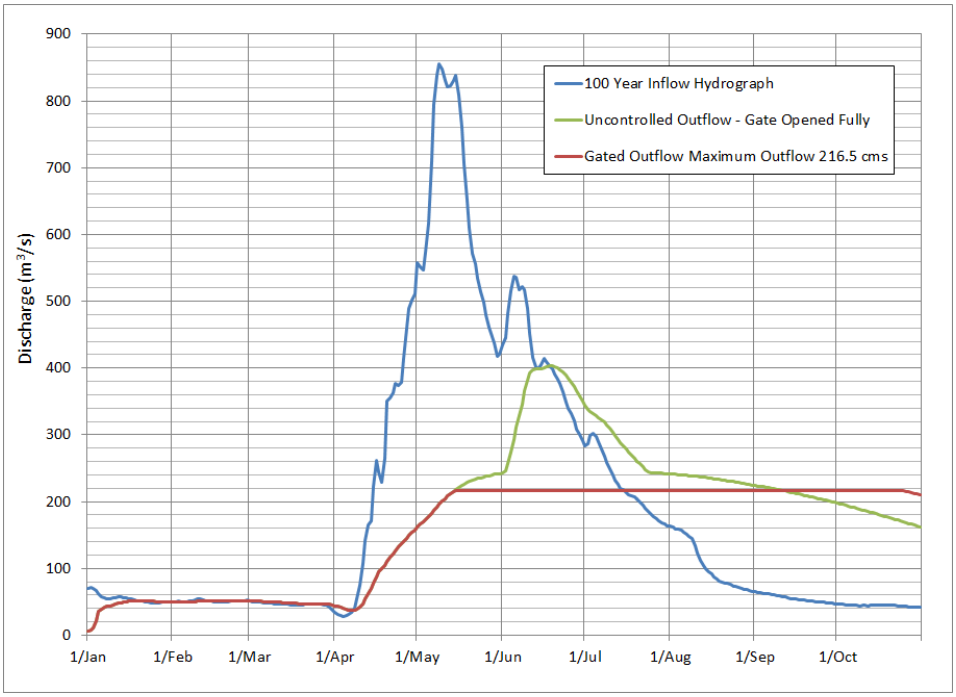
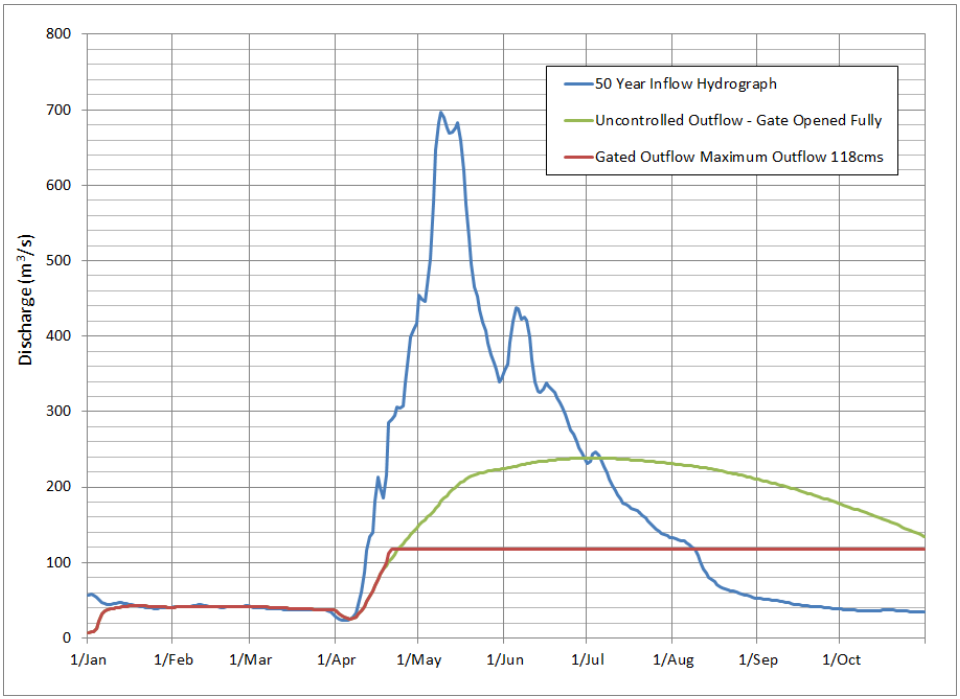


Figure 9.43
50-Year Flood Event on the Assiniboine River Routed with and without Alexander Dam Gate Control



The 500-year flood peak discharge of 1,283 m^3/s (45,309 cfs), when routed through the Alexander Dam Reservoir, results in a maximum outflow of 800 m^3/s (28,252 cfs) at the dam. If the outflow is controlled by operating the gates, the outflow can then be further reduced to a maximum of 643 m^3/s (22,707 cfs), or a reduction of 47% compared to the condition without the dam. Due to the proximity of the Alexander Dam Reservoir to Brandon, the discharge at Brandon is also reduced to 643 m^3/s (22,708 cfs) with the dam compared to 1,283 m^3/s (45,310 cfs) without the dam.

Effect at Brandon and Portage la Prairie

The outflow from the dam was routed to Portage la Prairie using the Muskingum River routing model documented in Section 8.2.3 of this report. Figures 9.44 to 9.47 illustrate the resulting hydrograph at Portage with and without the effect of the Alexander Dam for the various flood events.

Figure 9.44
Muskingum routing results of a 500-Year Event at Portage with and without Alexander Dam

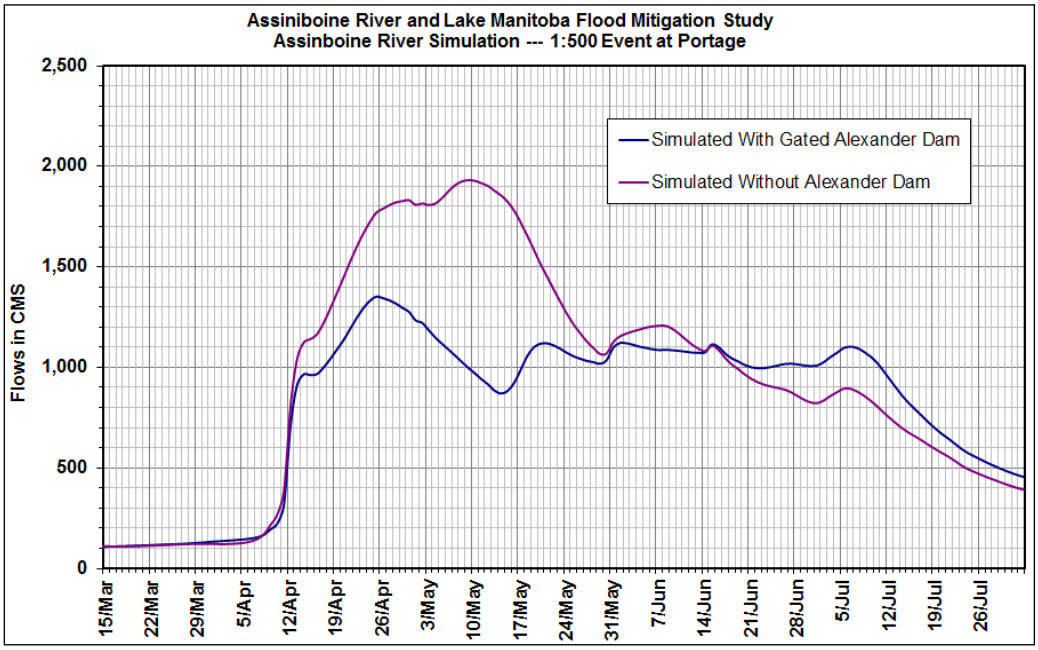


Figure 9.45
Muskingum routing results of a 200-Year Event at Portage with and without Alexander Dam

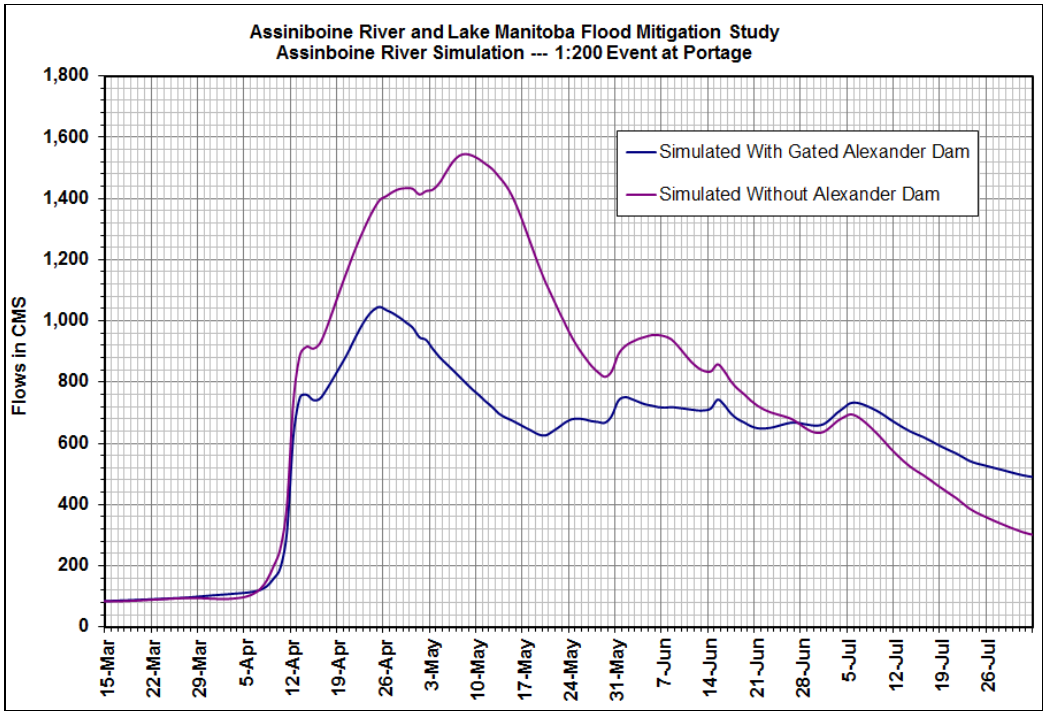


Figure 9.46
Muskingum routing results of a 100-Year Event at Portage with and without Alexander Dam

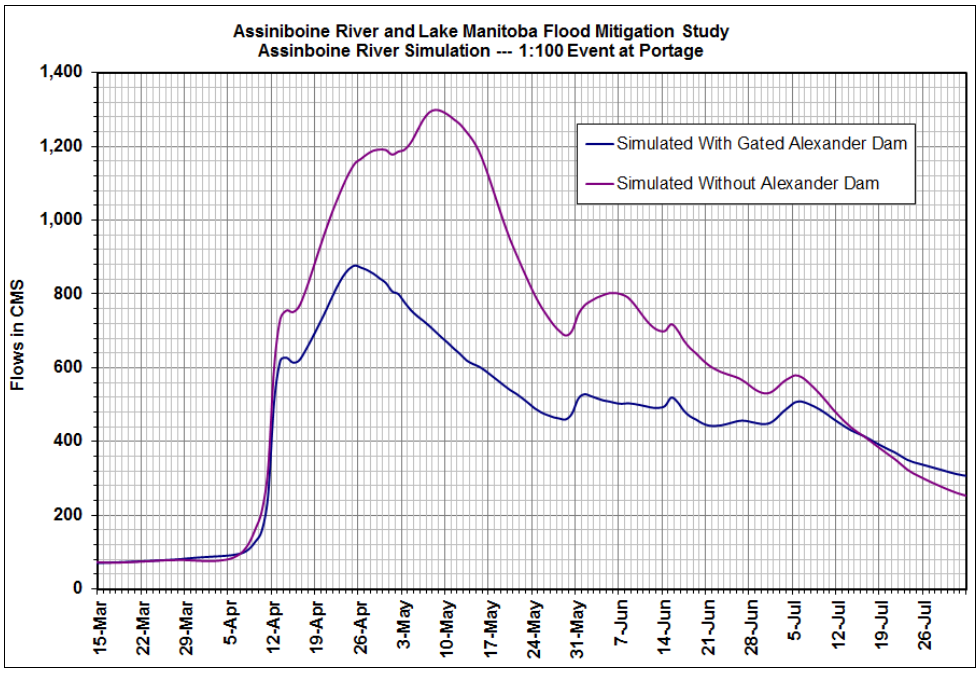
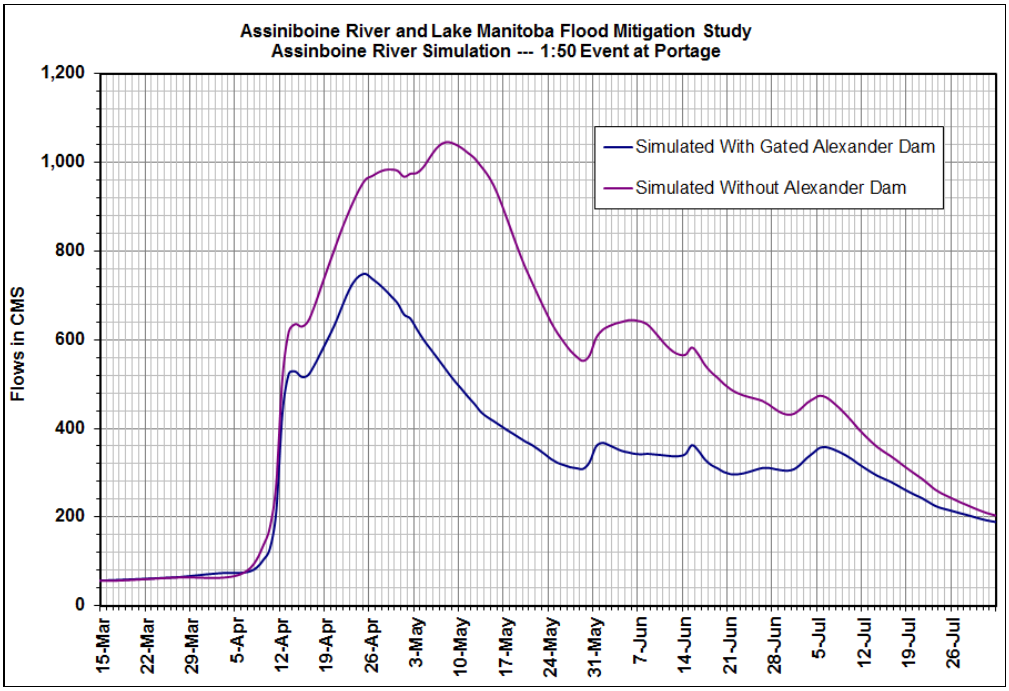


Figure 9.47
Muskingum routing results of a 50-Year Event at Portage with and without Alexander Dam



As shown on Figure 9.44, the peak discharge on the Assiniboine River at Portage la Prairie is 1,931 m^3/s (68,193 cfs). The corresponding peak discharge with gate control operation at Alexander Dam is 1,352 m^3/s (47,745 cfs). At Brandon, the results of the routing at the dam apply due to the proximity of the Alexander Dam to Brandon. Based on the results of the routings with and without the condition of gate control for the 1:50 to 1:500 year events, and considering the uncertainties with being able to operate the gates to achieve the maximum benefits as discussed in Section 9.5.6.2, the peak flow reduction capacity of the Alexander Dam was estimated. The results at Brandon and Portage are provided in Table 9.8.

Table 9.8
Peak Flow Reduction Capacity of Alexander Dam

Return Period (year)	Reduction in Peak Due to Alexander Dam (m^3/s)		% Reduction in Peak Due to Alexander Dam (%)	
	Brandon	Portage	Brandon	Portage
50	532	297	78	28
100	572	418	69	32
200	569	500	57	32
500	555	579	45	30

Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.6.3 Zelena Dam

Results at Zelena Dam

The design hydrographs of the Shell River at Zelena for return periods of 1:50 years to 1:500 years were routed through the Zelena Dam Reservoir. Reservoir routing was performed using a level pool routing program and applying the outlet rating curves and storage curves described in Section 9.5.4.3. Unlike other reservoirs, only the operation without gate control was implemented, due to the large reservoir size relative to inflow volumes. Under these conditions, the reservoir level would rise to a level that is less than the maximum permissible with appropriate freeboard. Figures 9.48 to 9.51 illustrate the reservoir levels for different return periods. The routing of the 1:500, 1:200, 1:100 and 1:50 year floods through the reservoir are shown in Figures 9.52 to 9.55.

Figure 9.48
Water Elevation of Zelena Reservoir under 500-Year Inflow Event

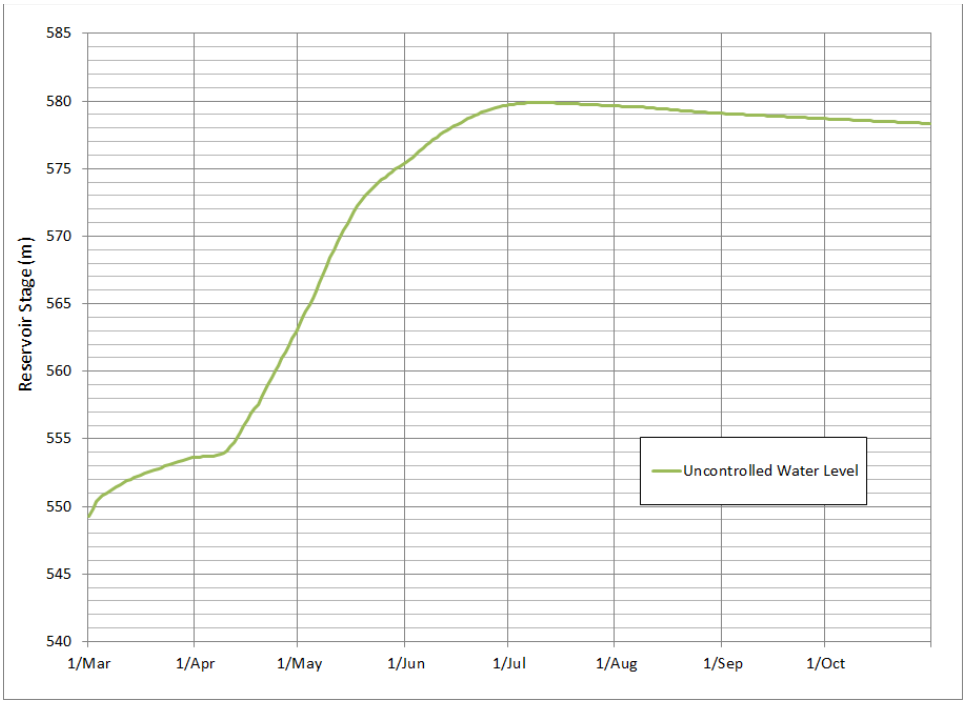


Figure 9.49
Water Elevation of Zelena Reservoir under 200-Year Inflow Event

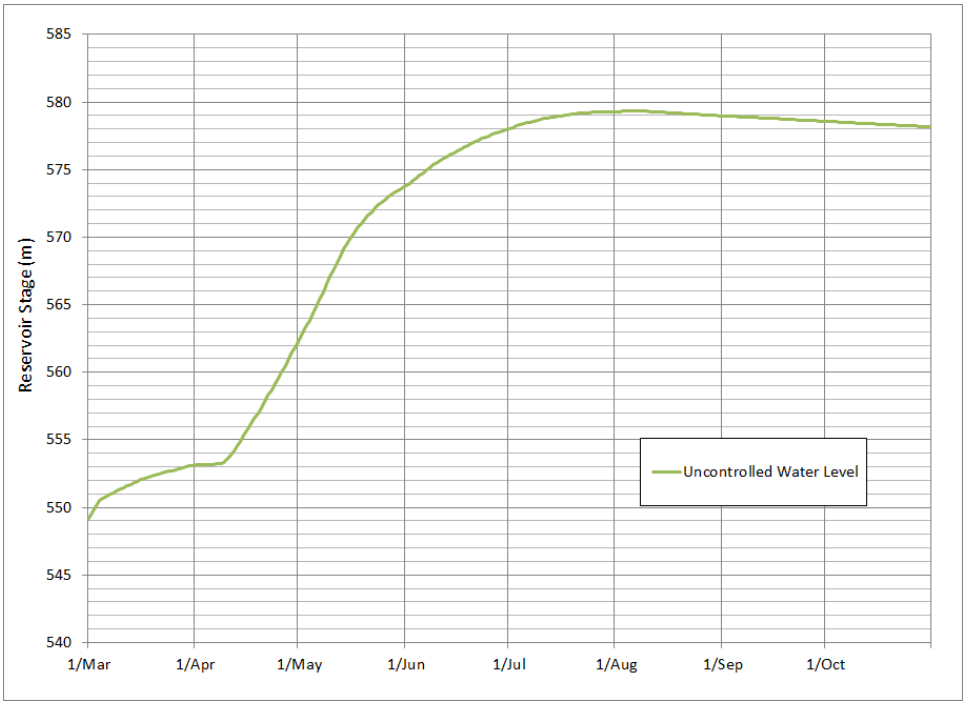


Figure 9.50
Water Elevation of Zelena Reservoir under 100-Year Inflow Event

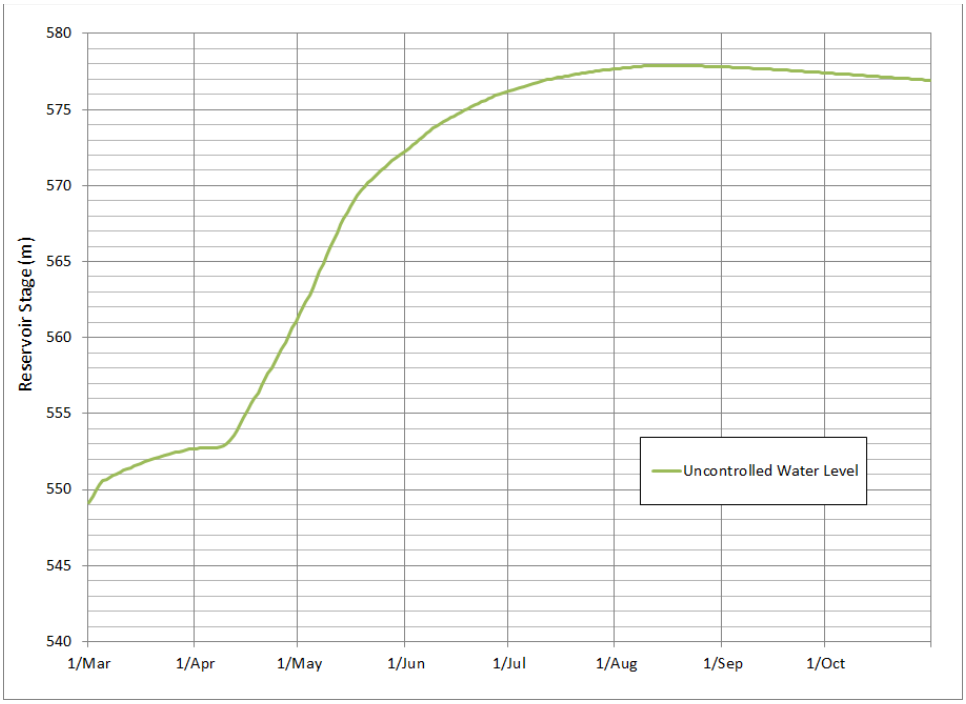


Figure 9.51
Water Elevation of Zelena Reservoir under 50-Year Inflow Event

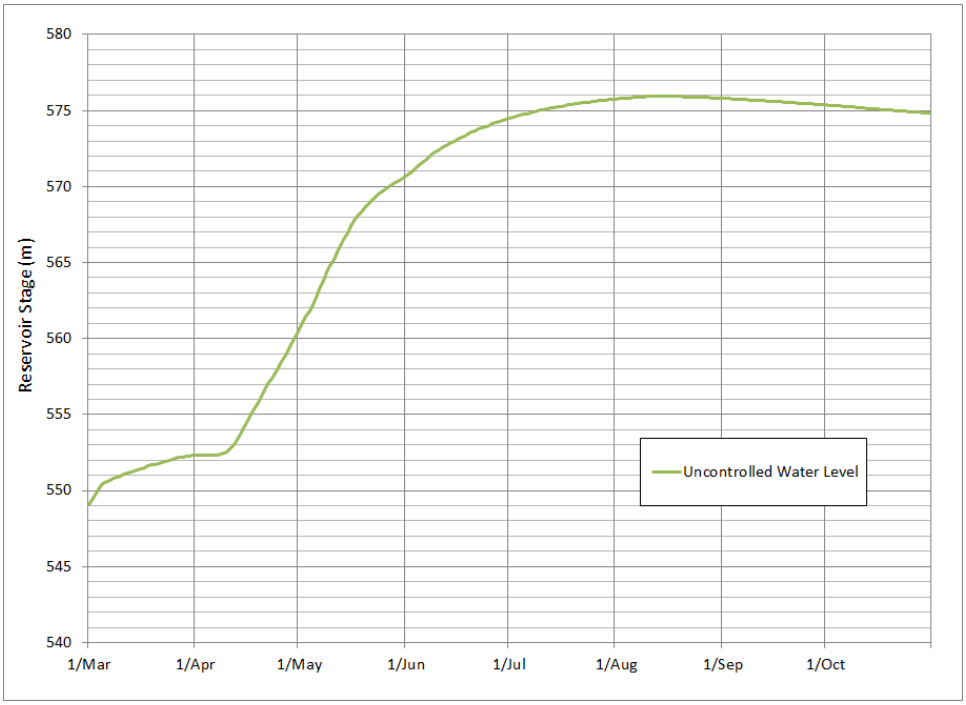


Figure 9.52
500-Year Flood Event on Shell River Routed through Zelena Dam

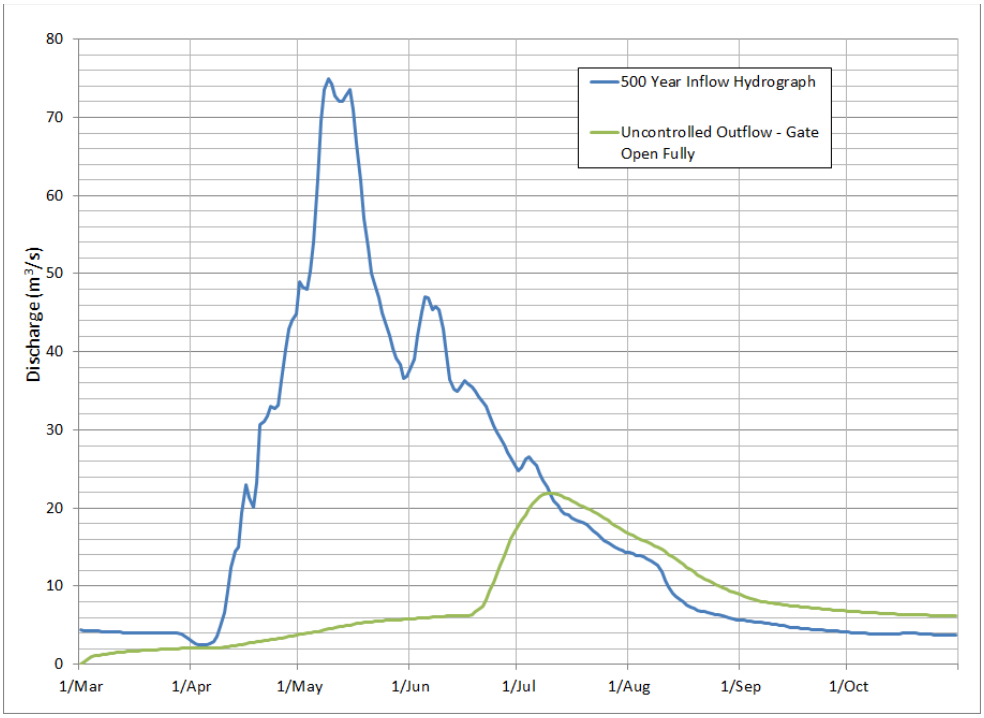


Figure 9.53
200-Year Flood Event on Shell River Routed through Zelena Dam

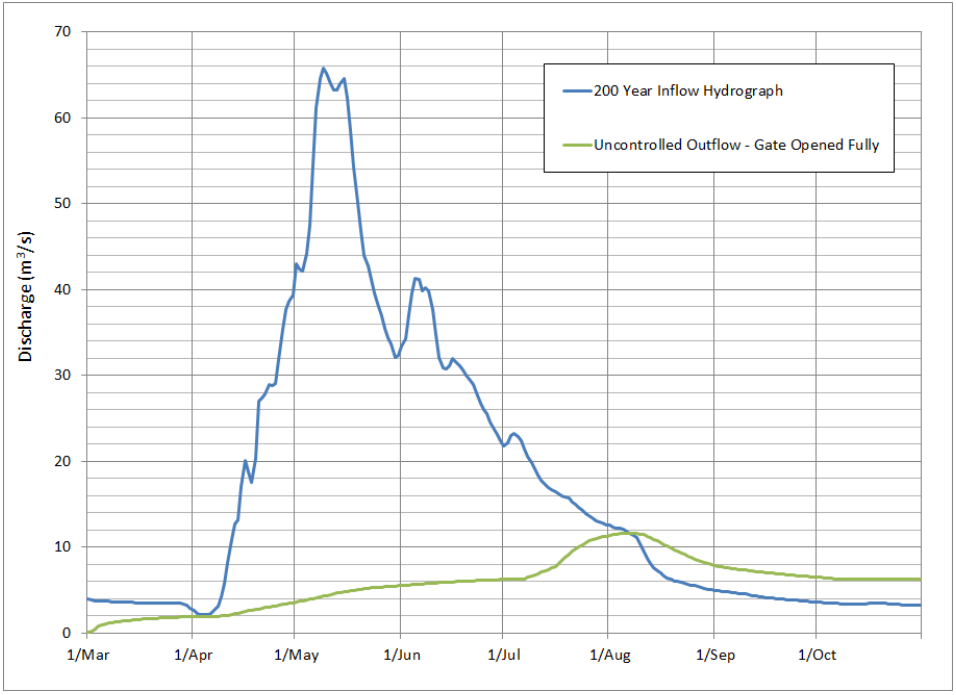


Figure 9.54
100-Year Flood Event on Shell River Routed through Zelena Dam

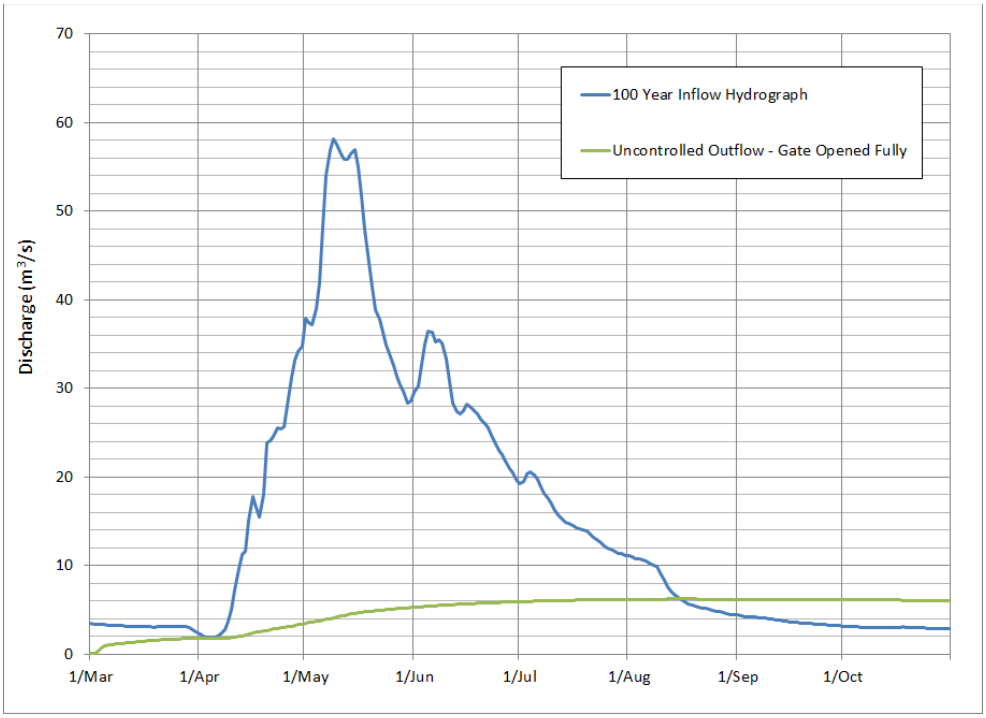
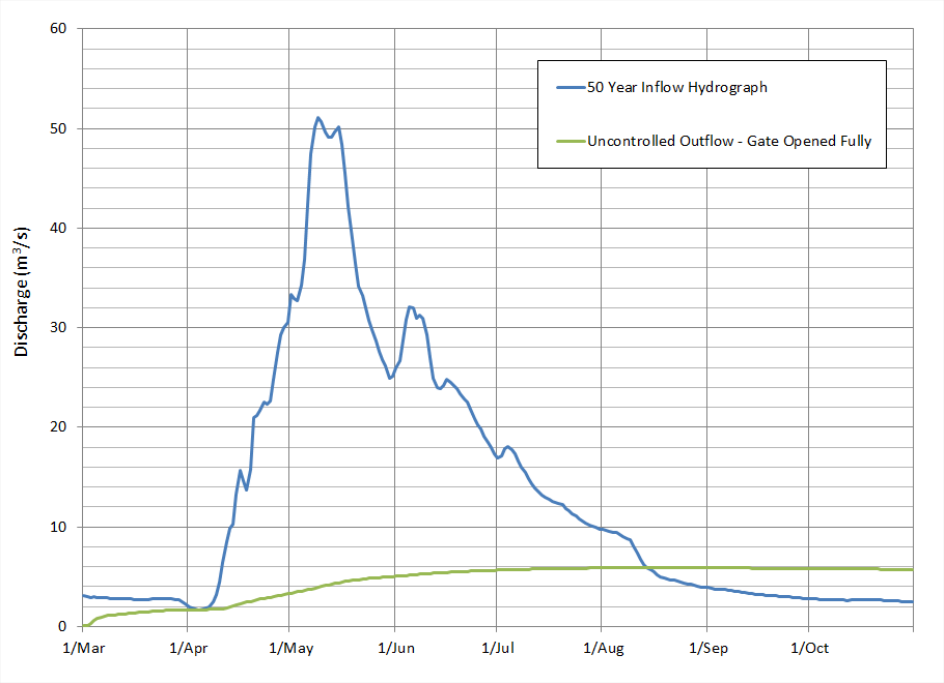


Figure 9.55
50-Year Flood Event on Shell River Routed through Zelena Dam



The 1:500 year flood peak discharge of 75 m^3/s (2,649 cfs), when routed through the Zelena Dam Reservoir, results in a maximum outflow of 22 m^3/s (777 cfs) at the dam. Gate control was not used for the Zelena Dam since maximum reduction was achieved without the use of gates.

Results at Brandon and Portage la Prairie

The hydrographs at the dam shown in Figures 9.52 to 9.55 were routed to Portage la Prairie using the Muskingum River Routing model documented in Section 8.2.3. The reduction in the outflow was applied directly to the Shellmouth Dam outflow without routing the flow through the Shellmouth Reservoir. This approximation was used since it was assumed that the peak reduction on the Shell River would occur coincidentally with the peak inflow on the Assiniboine River when the reservoir was full and inflow was equal to outflow. This approximation is considered to be valid for extreme floods but would not be completely valid for lower flood magnitudes. Given that the peak discharge of the Shell River is very small compared to the Assiniboine River at downstream locations, the error in the approximation would be insignificant in the context of this study.

Figures 9.56 to 9.63 illustrate the resulting hydrograph at Brandon and at Portage / Holland, with and without the effect of the reservoir at Zelena Dam for the various flood events.

Figure 9.56
Muskingum Routing Results at Brandon with and without Zelena Dam

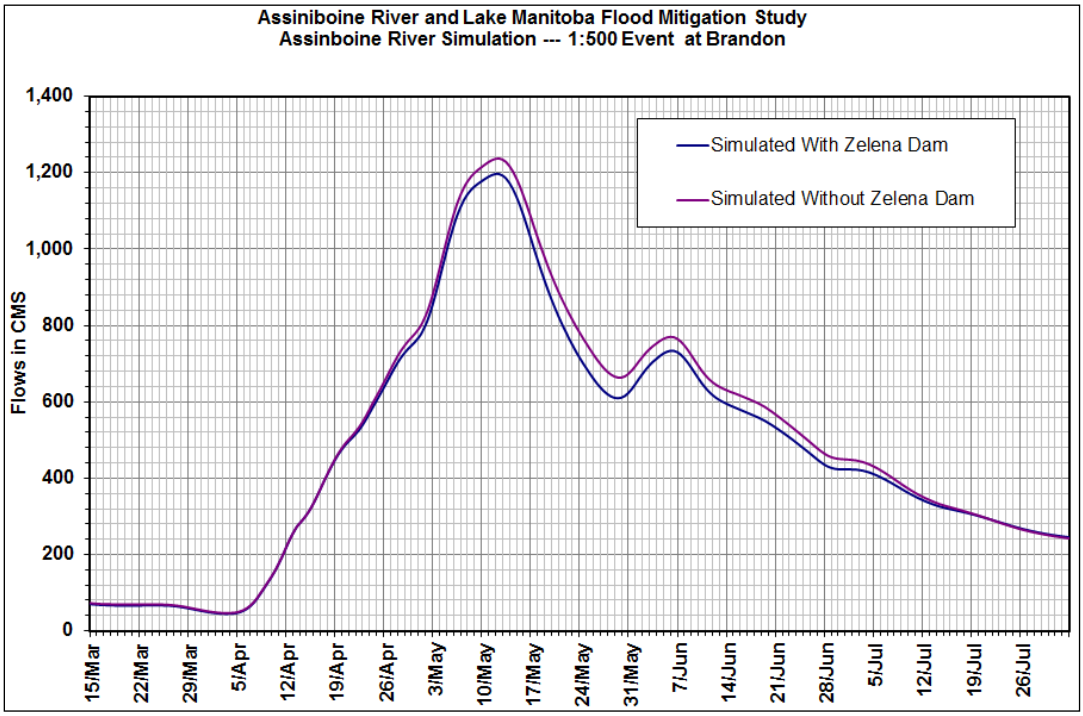


Figure 9.57
Muskingum Routing Results at Portage with and without Zelena Dam

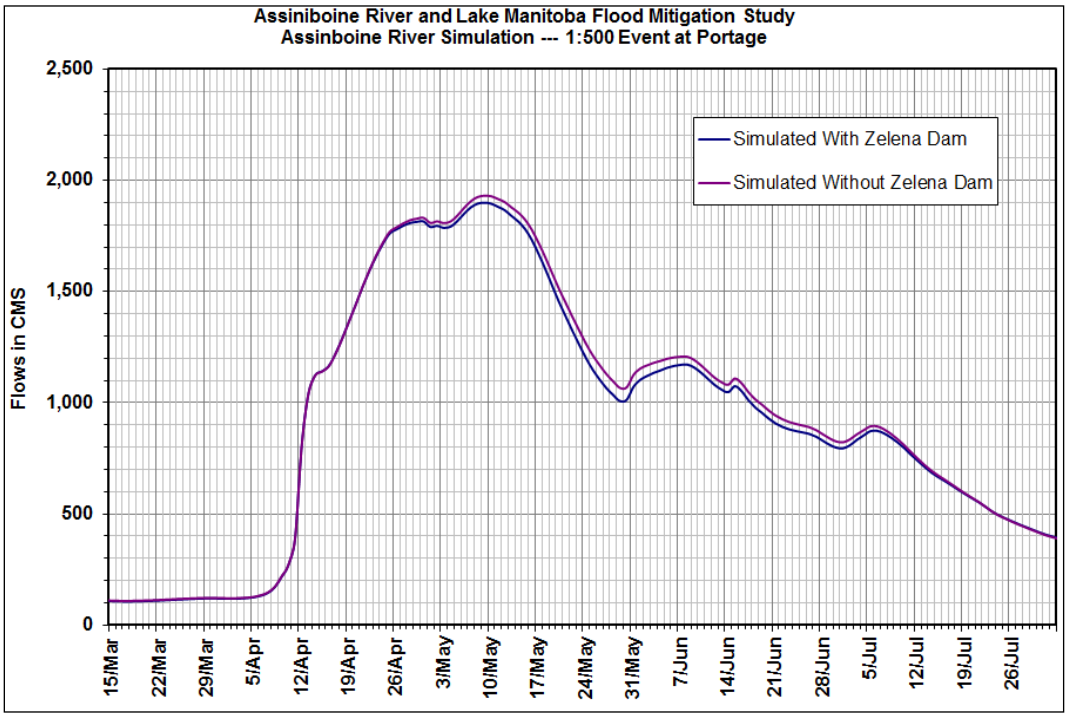


Figure 9.58
Muskingum Routing Results of a 200-Year Event at Brandon
with and without Zelena Dam

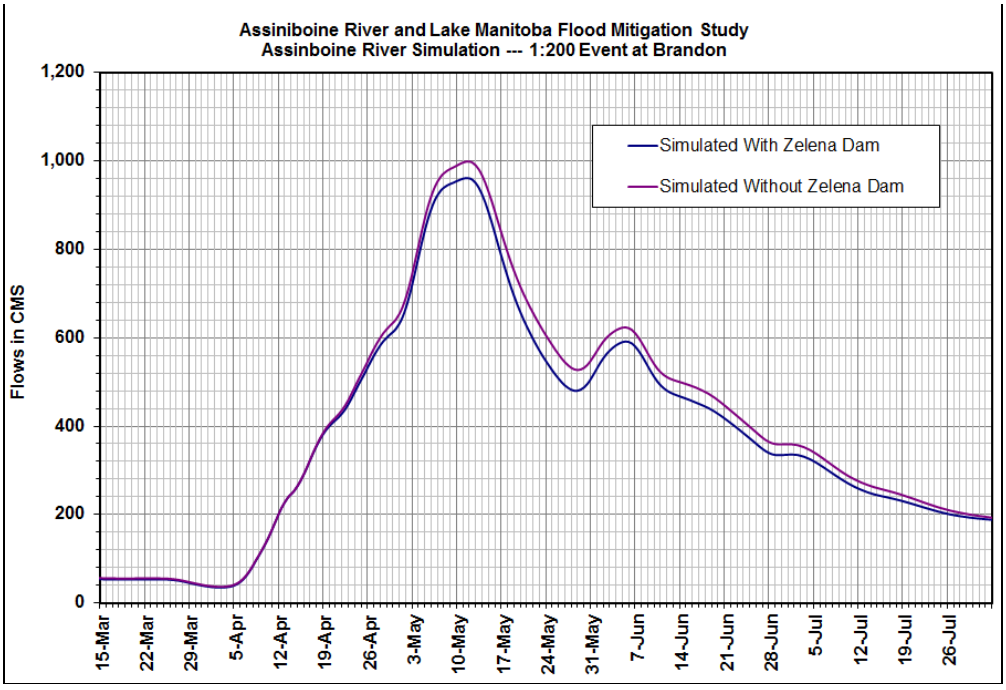


Figure 9.59
Muskingum Routing Results of a 200-Year Event at Portage
with and without Zelena Dam

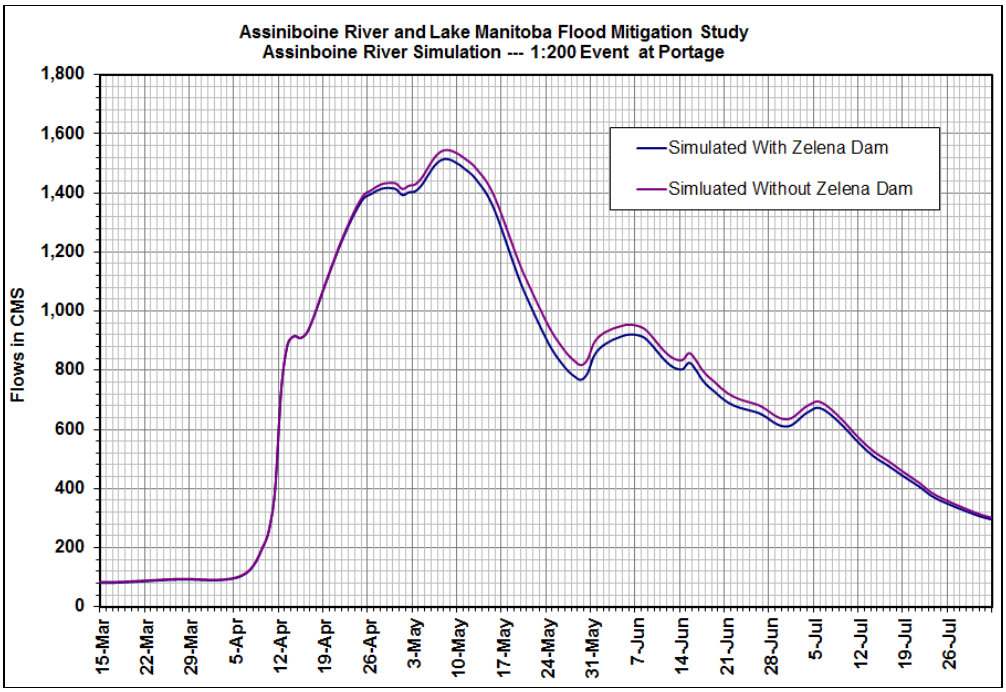


Figure 9.60
Muskingum Routing Results of a 100-Year Event at Brandon
with and without Zelena Dam

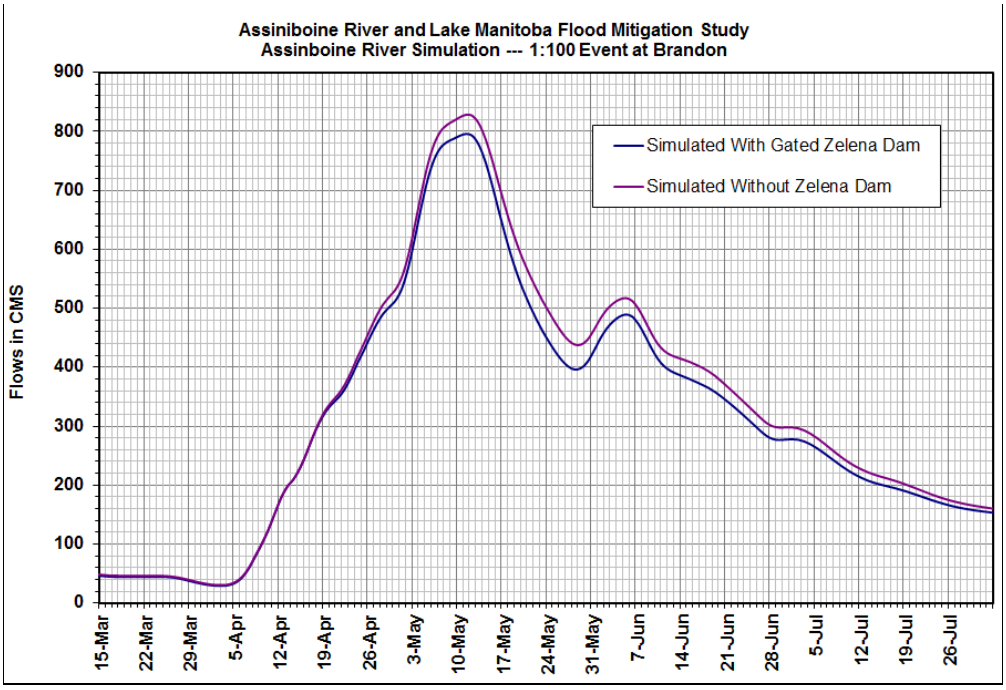


Figure 9.61
Muskingum Routing Results of a 100-Year Event at Portage
with and without Zelena Dam

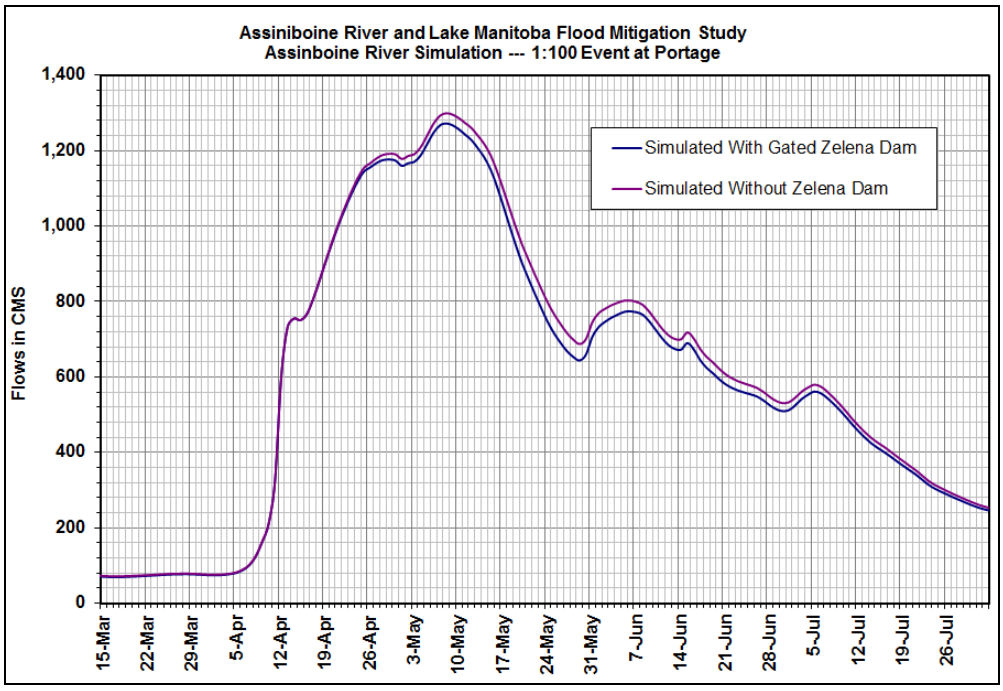


Figure 9.62
Muskingum Routing Results of a 50-Year Event at Brandon
with and without Zelena Dam

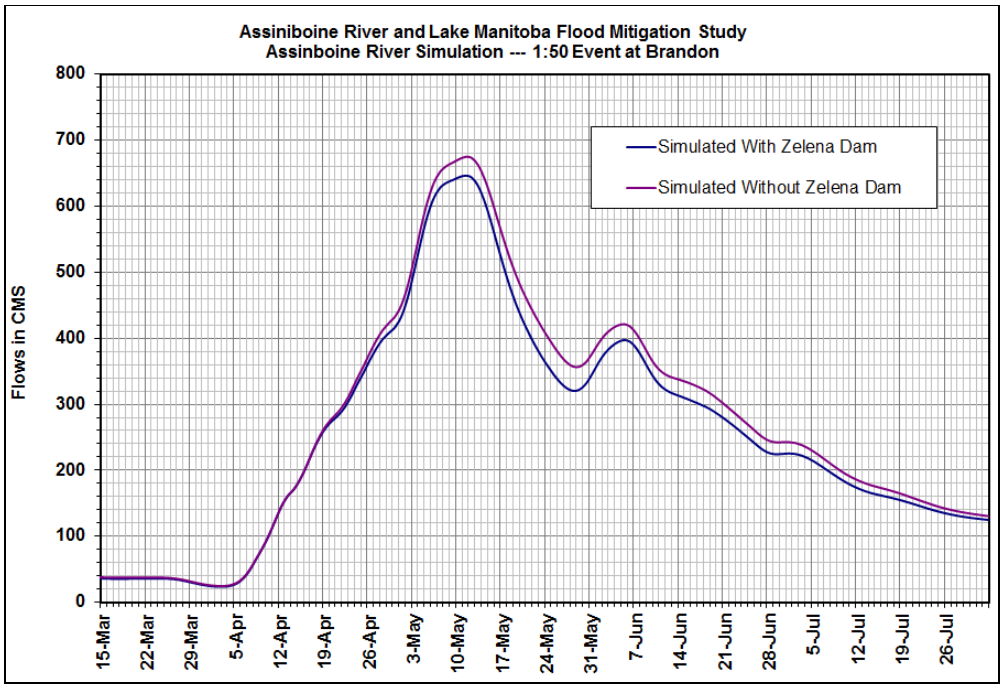
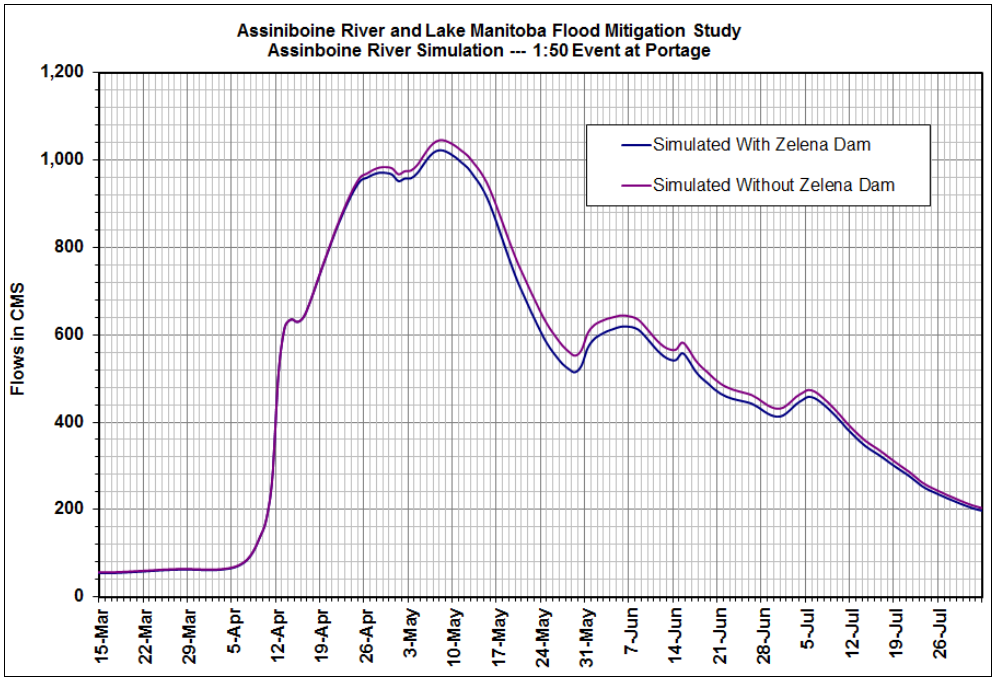


Figure 9.63
Muskingum Routing Results of a 50-Year Event at Portage
with and without Zelena Dam



The peak discharges on the Assiniboine River at Brandon and Portage la Prairie are 1,222 m^3/s (43,155 cfs) and 1,931 m^3/s (68,193 cfs), respectively. The corresponding peak discharges, at Brandon and Portage la Prairie, with the Zelena Dam are 1,195 m^3/s (42,201 cfs) and 1,897 m^3/s (66,992 cfs). Table 9.9 summarizes the results at Brandon and Portage for different return periods.

Table 9.9
Peak Flow Reduction Capacity of Zelena Dam

Return Period (year)	Reduction in Peak Due to Zelena Dam (m^3/s)		% Reduction in Peak Due to Zelena Dam (%)	
	Brandon	Portage	Brandon	Portage
50	28	23	4	2
100	32	23	4	2
200	37	35	4	2
500	27	34	2	2

Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.6.4 Victor Dam

Results at Victor Dam

The design hydrographs for the Qu'Appelle River at Victor Dam for return periods from 1:50 years to 1:500 years were routed through the Victor Dam Reservoir. Reservoir routing was performed using a level pool routing program and applying the outlet rating curves and storage curves described in Section 9.5.5.3. Similar to that of Holland Dam, two modes of operations were implemented (i.e., operation with and without gate control). The operation with gate control resulted in the reservoir level rising to the maximum permissible with appropriate freeboard. The freeboard used was 0.5 m (1.6 ft) for the 1:500 year flood. The routing of the 1:500, 1:200, 1:100 and 1:50 year floods through the reservoir is shown in Figures 9.64 to 9.67. Figures 9.68 to 9.71 illustrate the reservoir levels for different return periods.

Figure 9.64
500-Year Flood Event on the Assiniboine River Routed with and without Victor Dam Gate Control

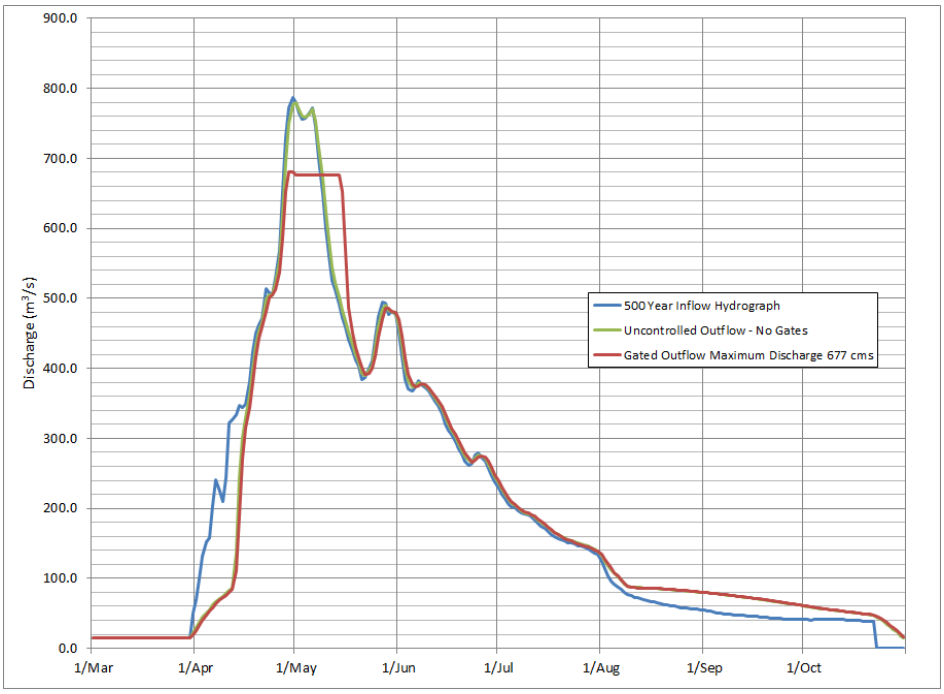


Figure 9.65
200-Year Flood Event on the Assiniboine River Routed with and without Victor Dam Gate Control

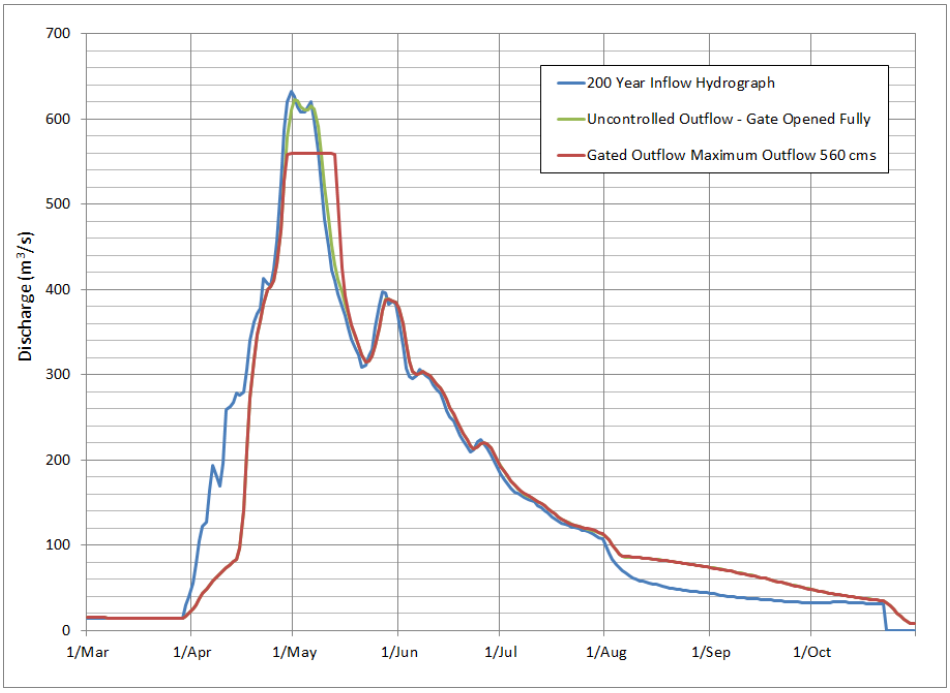


Figure 9.66
100-Year Flood Event on the Assiniboine River Routed with and without Victor Dam Gate Control

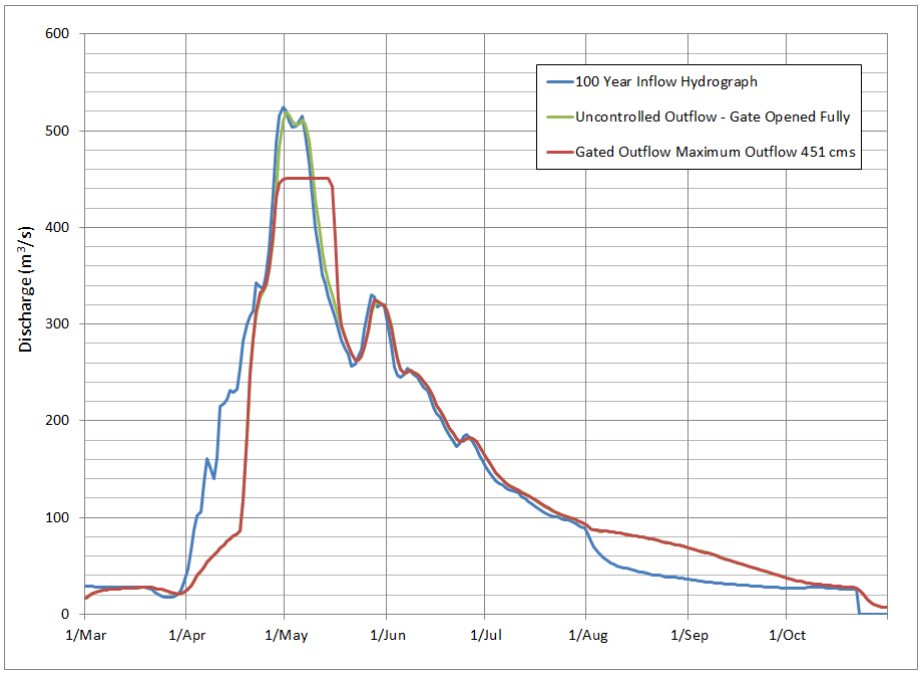


Figure 9.67
50-Year Flood Event on the Assiniboine River Routed with and without Victor Dam Gate Control

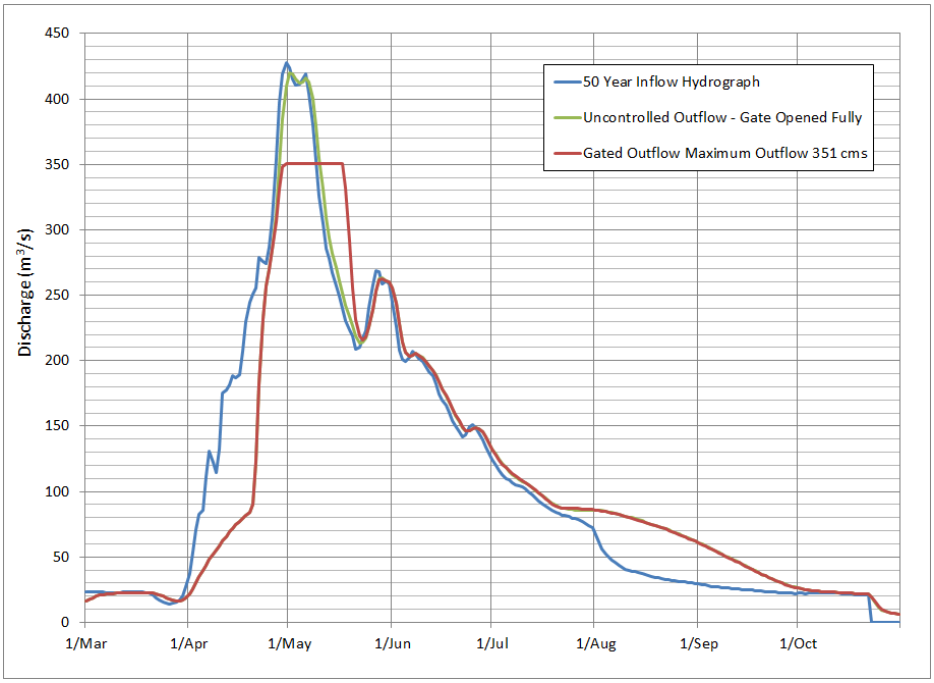


Figure 9.68
Water elevation of Victor Reservoir with and without Outflow Control, under 500-Year Inflow Event

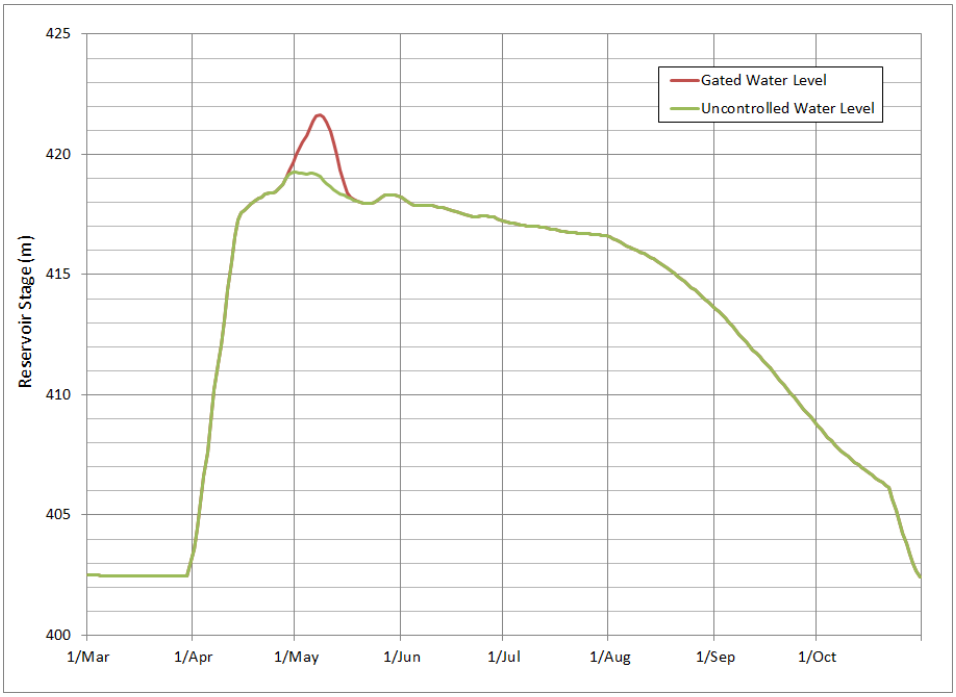


Figure 9.69
Water elevation of Victor Reservoir with and without Outflow Control, under 200-Year Inflow Event

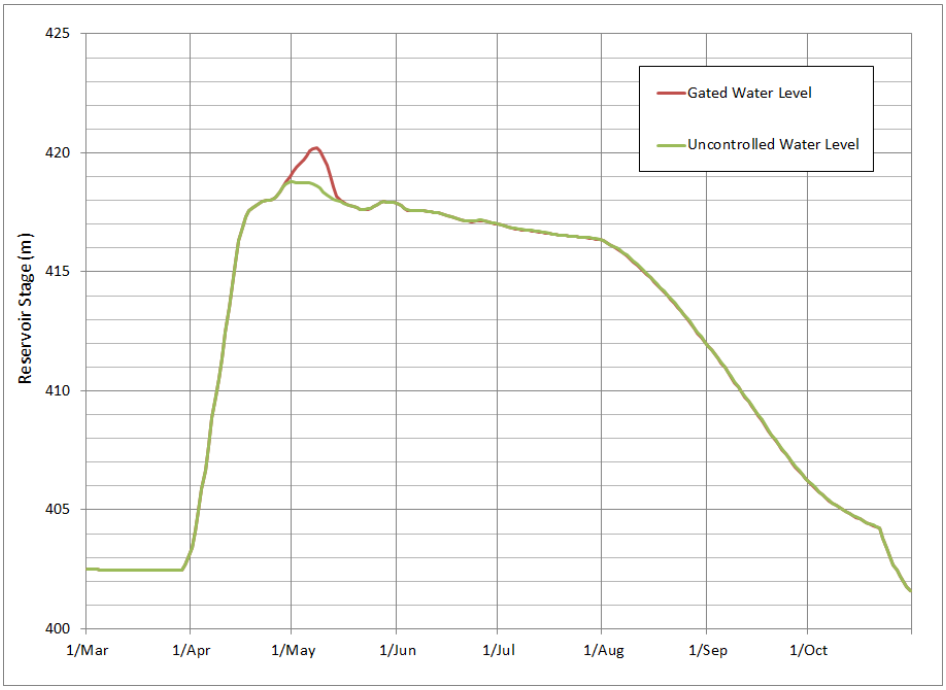


Figure 9.70
Water elevation of Victor Reservoir with and without Outflow Control, under 100-Year Inflow Event

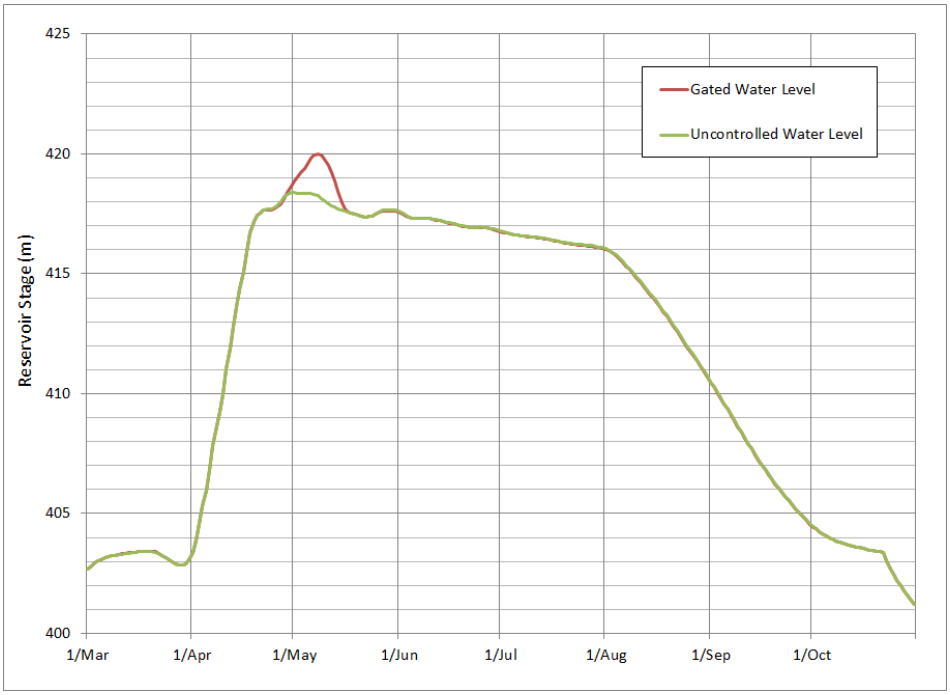
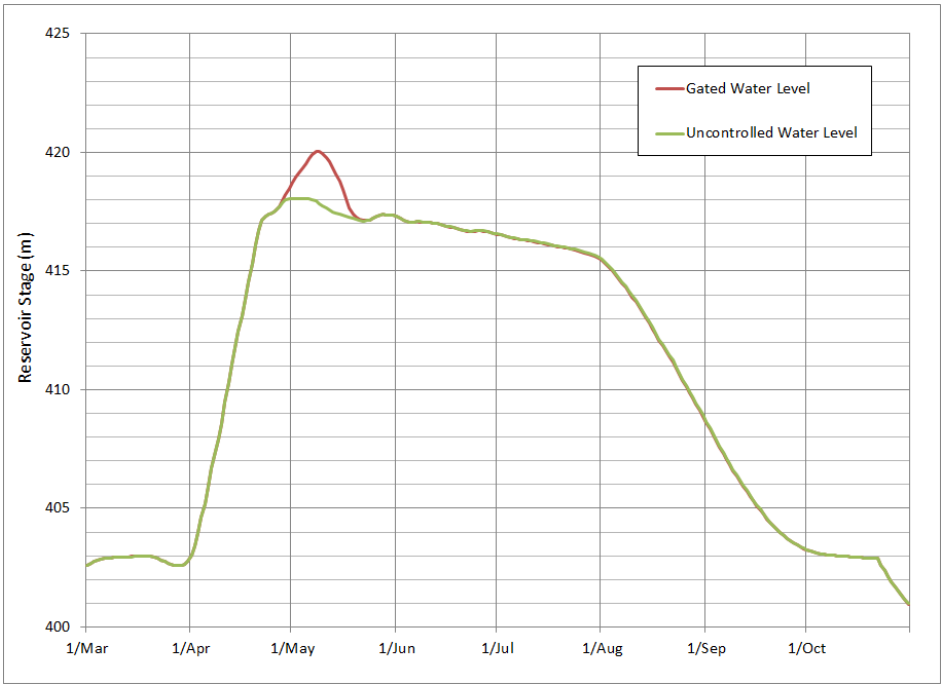


Figure 9.71
Water elevation of Victor Reservoir with and without Outflow Control, under 50-Year Inflow Event



The 1:500 year flood peak discharge of 790 m^3/s (27,899 cfs), when routed through the Victor Dam Reservoir, results in a maximum outflow of 780 m^3/s (27,545 cfs) at the dam. If the outflow is controlled by operating the gates, the outflow can be further reduced to a maximum of 677 m^3/s (23,908 cfs), or a reduction of 14% compared to the condition without the dam.

As discussed in Section 9.5.5.2, the design hydrographs for Victor Dam are conservative because the drainage area method used to develop the hydrographs overestimates the flows on the Qu'Appelle River. To determine the effects of this conservative assumption, peak flow reductions through the dam were also calculated using hydrographs that were adjusted by a factor of 0.5. It was determined that the results of the analysis at the dam would be lower using smaller hydrographs. For the 500 year flood, the peak reduction was 82 m^3/s (2,896 cfs), which was 28 m^3/s (989 cfs) less than the base case results. Once routed downstream, the difference in peak flow at Portage la Prairie became negligible. On this basis, the results of the analysis using the hydrographs developed from the drainage area method were adopted for this study.

Results at Brandon and Portage la Prairie

The results at the dam illustrated in Figures 9.64 to 9.67 were routed to Portage la Prairie using the Muskingum River routing model documented in Section 8.2.3. Figures 9.72 to 9.79 illustrate the resulting hydrographs at Brandon and at Portage, with and without the effect of the Victor Dam for the various flood events.

Figure 9.72
Muskingum Routing Results at Brandon with and without Victor Dam

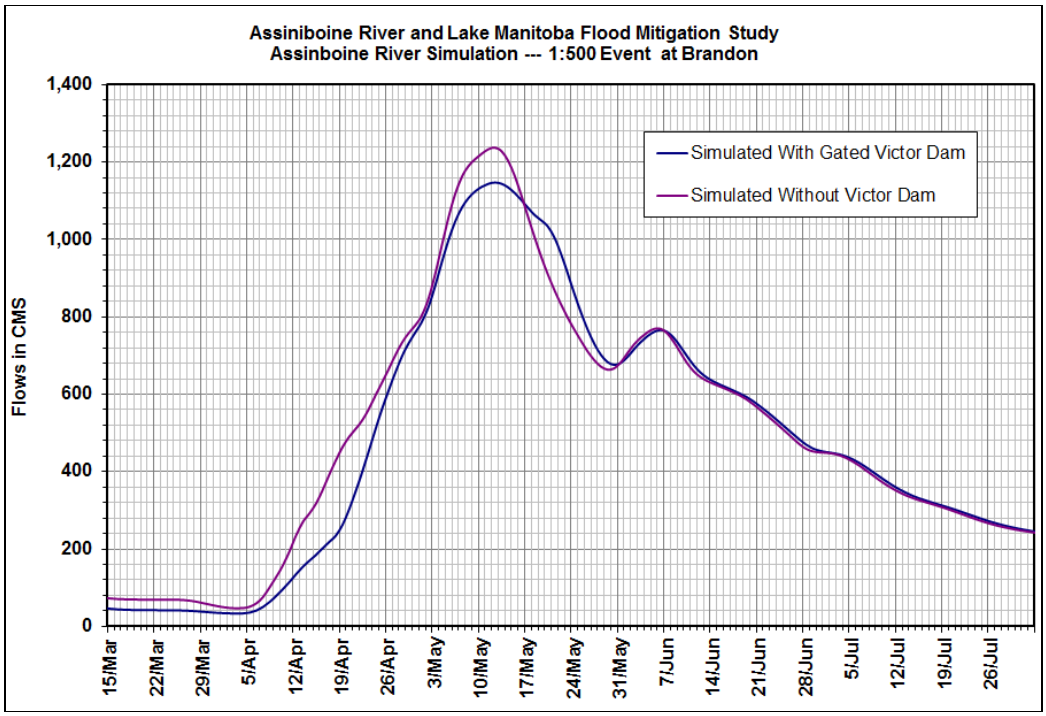


Figure 9.73
Muskingum Routing Results at Portage with and without Victor Dam

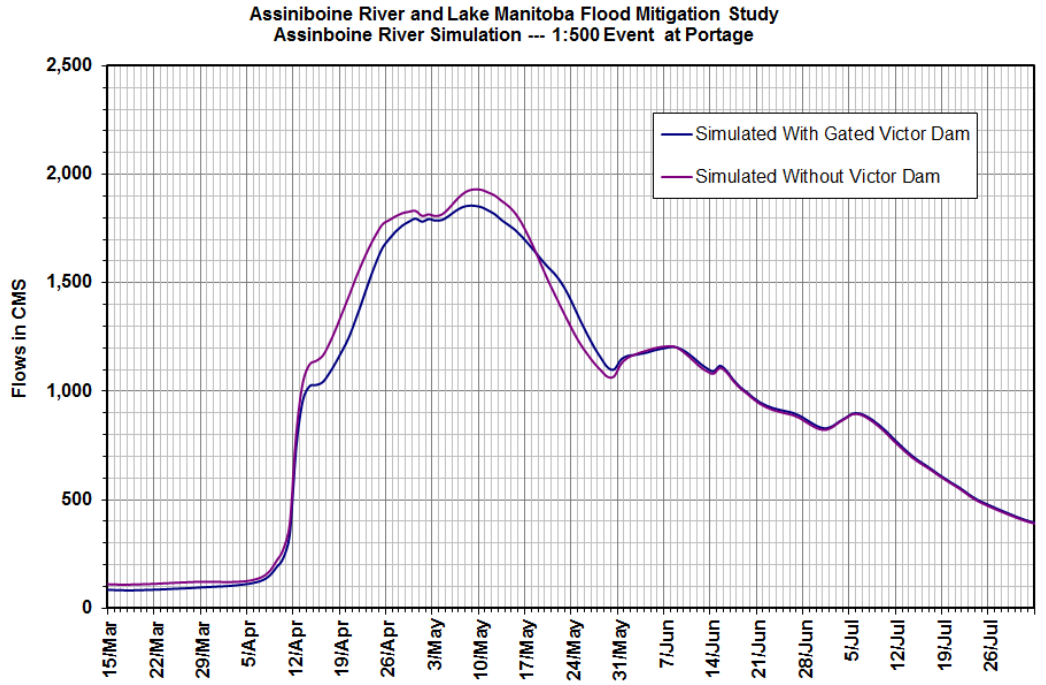


Figure 9.74
Muskingum Routing Results of a 200-Year Event at Brandon with and without Victor Dam

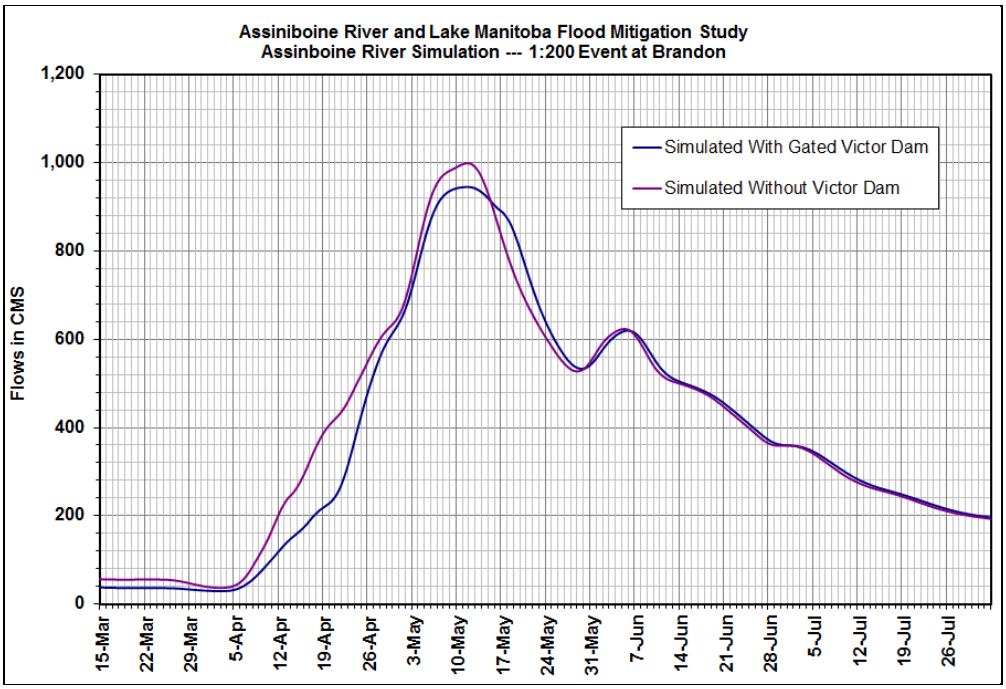


Figure 9.75
Muskingum Routing Results of a 200-Year Event at Portage with and without Victor Dam

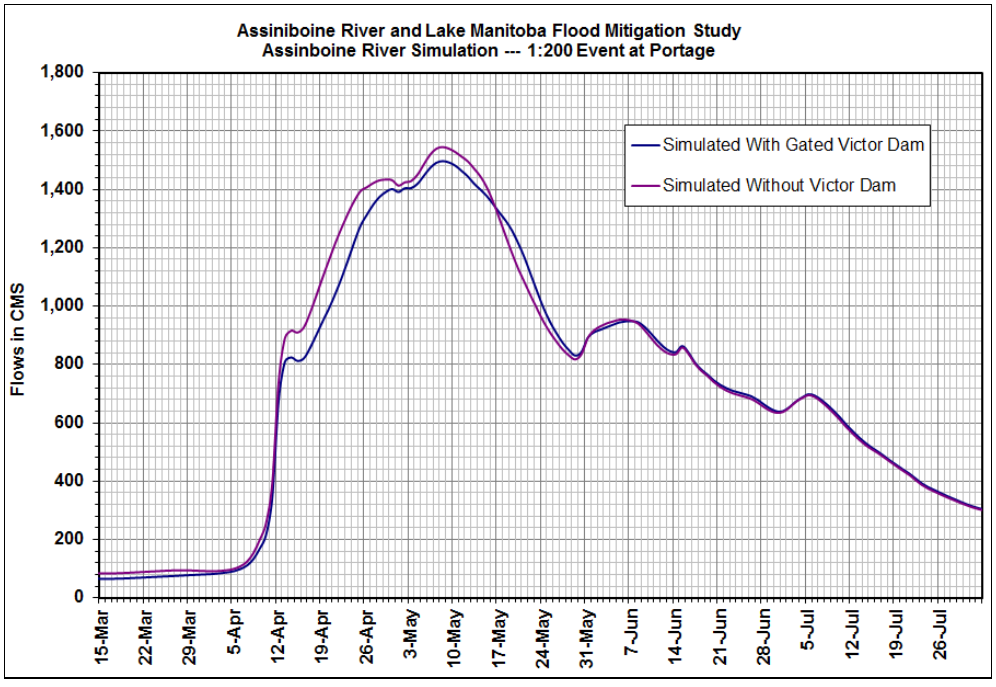


Figure 9.76
Muskingum Routing Results of a 100-Year Event at Brandon
with and without Victor Dam

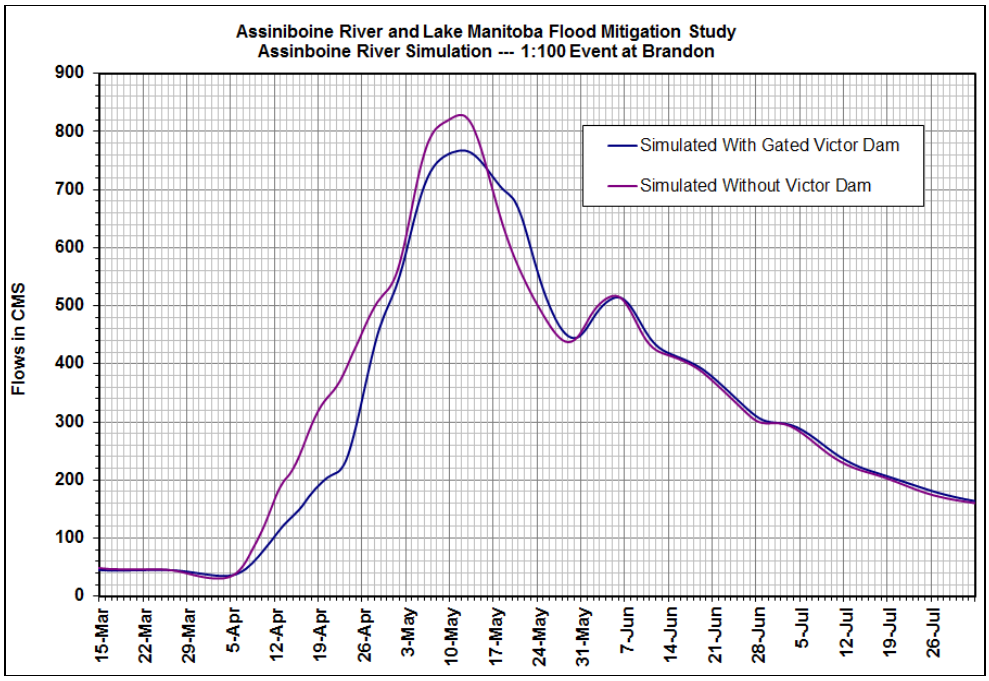


Figure 9.77
Muskingum Routing Results of a 100-Year Event at Portage
with and without Victor Dam

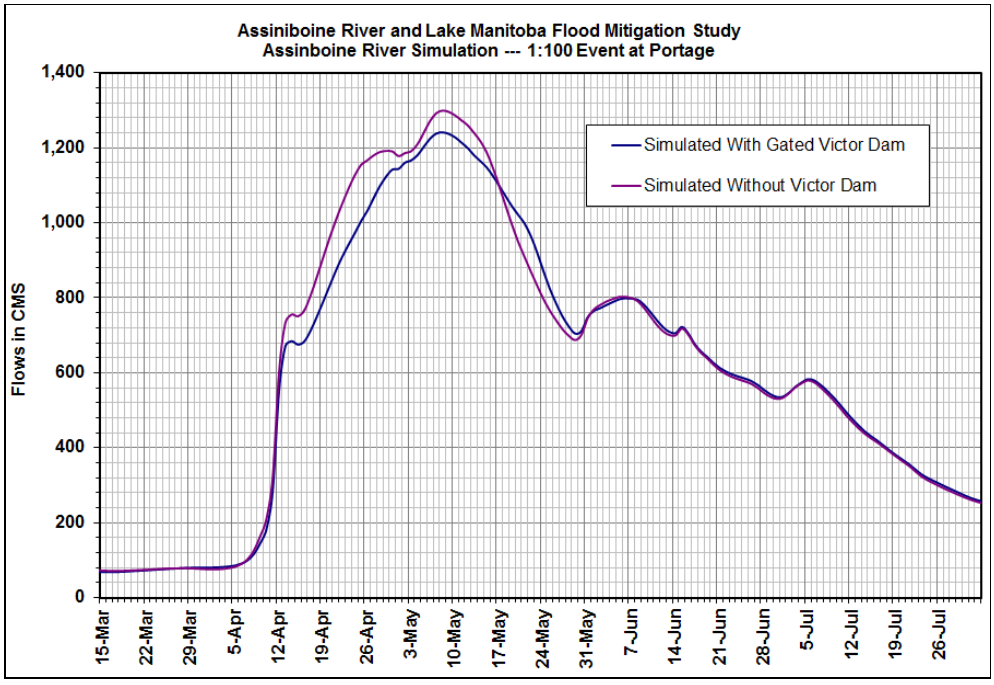


Figure 9.78
Muskingum Routing Results of a 50-Year Event at Brandon with and without Victor Dam

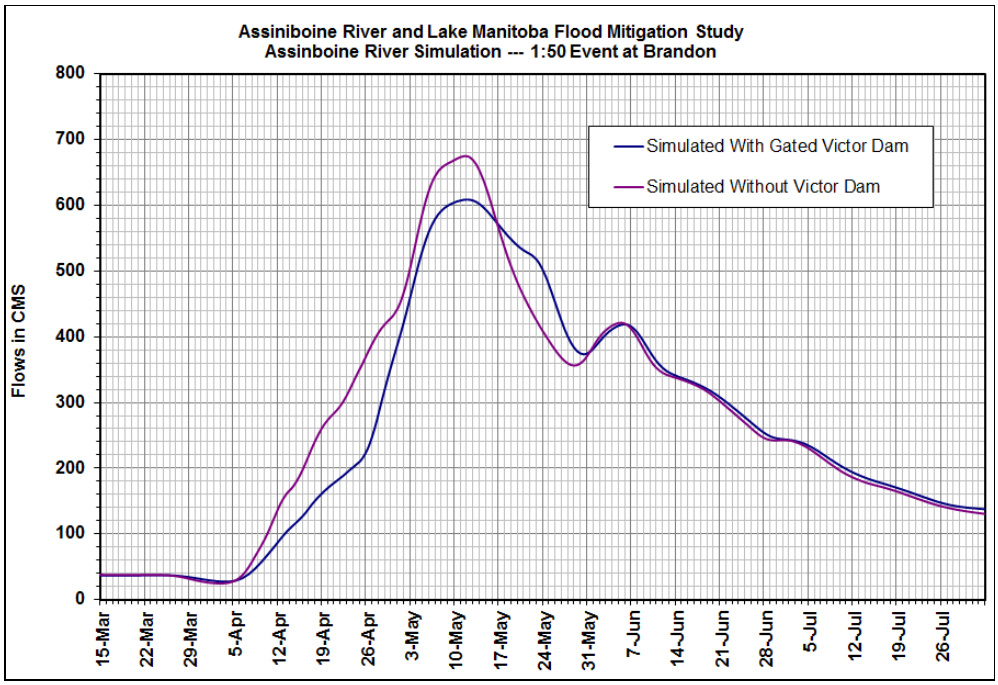
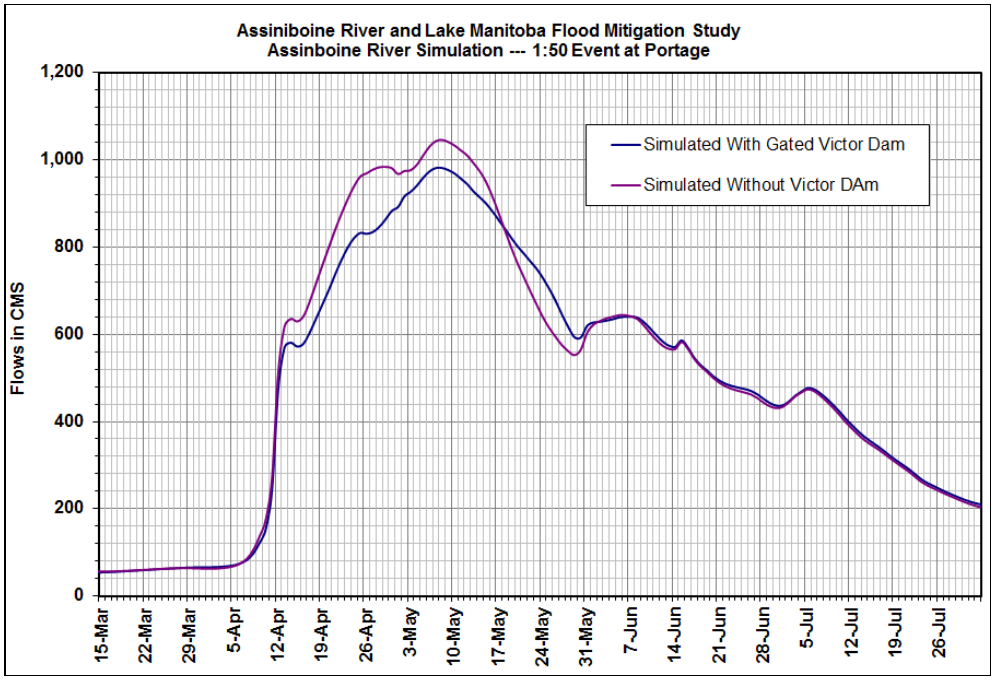


Figure 9.79
Muskingum Routing Results of a 50-Year Event at Portage with and without Victor Dam



The peak discharges on the Assiniboine River at Brandon and Portage la Prairie are 1,222 m^3/s (43,155 cfs) and 1,931 m^3/s (68,193 cfs), respectively. The corresponding peak discharges, at Brandon and Portage la Prairie, with gate control operation of Victor Dam, are 1,147 m^3/s (40,506 cfs) and 1,855 m^3/s (65,509 cfs). Based on the results of the routings with and without the condition of gate control for the 1:50 to 1:500 year events, and considering the uncertainties with being able to operate the gates to achieve the maximum benefits as discussed in Section 9.5.6.1, the peak flow reduction capacity of the Victor Dam was estimated. The results at Brandon and Portage are provided in Table 9.10.

Table 9.10
Peak Flow Reduction Capacity of Victor Dam

Return Period (year)	Reduction in Peak Due to Victor Dam (m^3/s)		% Reduction in Peak Due to Victor Dam (%)	
	Brandon	Portage	Brandon	Portage
50	50	53	8	5
100	46	47	5	3
200	41	41	4	3
500	57	63	5	3

Note: Flows can be converted to cfs by multiplying metric values by 35.3.

9.5.7 Estimated Costs

A cost estimate was developed for each of the four large storage reservoir projects documented in this report. Project costs were based on SNBB, 1972. At the time, costs were itemized for each project and included main embankment, spillway outlet works, reservoir damages, clearing and care of river costs during construction. The costs were previously reported in 1971 dollars.

Table 9.11 lists the project costs for the four reservoir storage alternatives. The 1971 costs were escalated to 1997 dollars using the Canadian Electric Utility Construction Price Index (CEUCPI) for Dams, Waterways and Reservoirs. The 1997 costs were further escalated to 2014$ using an assumed escalation rate of 5% per year based on Consumer Price Index data from Statistics

Canada, as well as experience with other similar, recent KGS Group projects. The 1997 cost index value was the last year that the CEUCPI was available.

The flood control benefits determined in this study required vertical lift gates to be operated to maximize storage benefits. The costs that were determined by SNBB studies (SNBB, 1972) did not include gates at the Alexander Dam and at the Victor Dam. The costs of the gates at those two sites were determined using a unit rate of $25,000/m^2 determined based on engineering experience with the recent Pointe du Bois Project. For the Holland Dam, it was assumed that the difference in costs between the radial gates that were originally considered in the SNBB studies (SNBB, 1972) and the vertical lift gates considered in the current study were negligible. Therefore the Holland Dam did not require an additional cost for gates. Also, as discussed in Section 9.5.2 and 9.5.5, the width of the spillway at the Holland Dam and the Victor Dam were increased for this study. The cost for the additional spillway width was determined on the basis of the original cost per unit width, escalated to 2014$. The corresponding costs to include the additional gates and increase the spillway width are included in Table 9.11.

Table 9.11
Project Cost Estimates

Components	Alexander Dam	Holland Dam	Victor Dam	Zelena Dam
1971 Components				
Land	$ 11,250,000	$ 1,191,000	$ 777,000	$ 398,000
Reservoir Clearing	$ 2,100,000	$ 1,900,000	$ 270,000	$ 621,000
Reservoir Damages	$ 5,485,000	$ 784,000	$ 2,102,000	$ 193,000
Care of Water	$ 400,000	$ 500,000	$ 300,000	$ 400,000
Earth Dam	$ 3,950,000	$ 5,663,000	$ 1,644,000	$ 1,846,000
Spillway	$ 1,900,000	$ 1,657,000	$ 828,000	$ 1,016,000
Conduit	$ 900,000	$ 1,085,000	$ 527,000	$ 314,000
Sub Total Direct Cost (1971$)	$ 25,985,000	$ 12,780,000	$ 6,448,000	$ 4,788,000
Sub Total Direct Cost (2014$)	$ 292,019,000	$ 143,622,000	$ 72,463,000	$ 53,808,000
Additional Components				
Gates	$ 13,935,000		$ 9,954,000	
Spillway		$ 14,897,000	$ 1,642,000	

Components	Alexander Dam	Holland Dam	Victor Dam	Zelena Dam
Allowance for Uncertainties (10%)	$ 30,595,000	$ 15,852,000	$ 8,406,000	$ 5,381,000
Total Direct Cost (2014$)	$ 336,549,000	$ 174,371,000	$ 92,465,000	$ 59,189,000
Environmental Mitigation (15%)	$ 50,482,000	$ 26,156,000	$ 13,870,000	$ 8,878,000
Indirect Cost (Owner's costs, Engineering, Administration, Approvals) (20%)	$ 67,310,000	$ 34,874,000	$ 18,493,000	$ 11,838,000
Contingencies (30%)	$ 136,302,000	$ 70,620,000	$ 37,448,000	$ 23,972,000
Total Cost in 2014 Dollars	**$ 590,643,000**	**$ 306,021,000**	**$ 162,276,000**	**$ 103,877,000**

Land acquisition was included in the cost estimates from the 1972 studies, and considered whether the land was privately owned or crown owned and also the type of land use. The unit rates used at the time, once escalated to 2014$ as discussed previously, fall within the range that was established for the current study of $1,000/ha (for non-arable land) to $15,000/ha (for prime arable land), as reported in Section 7.0 of the report. On this basis, adjusting 1972 land acquisition estimates was not warranted for the current study. However, limited detail was available in the SNBB reports (SNBB, 1972) to determine the exact distribution between private and crown land, as well as land use. Should any of the large reservoir projects proceed to a subsequent stage of study, further analysis will be necessary to determine what area of land needs to be purchased and current land use.

An allowance for "reservoir damages" was considered in the 1972 studies and included costs for raising and relocation of roads, transmission line relocation, relocation of farms and demolition of abandoned buildings. For example, the Holland Dam Project included a cost for the relocation of several farmsteads, the raising of PTH 5 and the protection of the road for wave action. Detailed information on the items considered was not always available for each project, neither was it clear whether other items, such as the relocation of properties in the First Nation Community of Sioux Valley for the Alexander Dam Project, were considered. Further investigations beyond the scope of the current study are necessary to determine the exact amount and items considered in the previous studies, and whether or not these values are significantly different today. To account for this high level of uncertainty, an additional allowance of 10% of the total direct cost was included in the estimation.

In addition to the costs outlined above, consideration must be given to the costs for easements and compensation, environmental assessment and licensing, and mitigation of environmental impacts. Due to the potential significant environmental impacts (discussed in Section 9.5.8) associated with the proposed dam projects, major environmental mitigation costs are expected; therefore, a cost allowance equivalent to 15% of the total direct cost was added to the estimate. As per the economic guidelines set out for the study and discussed in Section 7.0 of the report, an allowance of 20% of the total direct cost was also added to account for indirect costs such as engineering, approvals, administration and owner's costs, and a contingency allowance of 30% of the direct and indirect cost was added for uncertainty.

It should be noted that it is possible that the assumed proportion of public (crown) and private lands and the resulting land acquisition costs presented in this study (which were based largely on the SNBB, 1972 report values) could potentially be less than actual farmland market costs due to potential changes in land ownership proportions since 1972. Consequently, the actual project costs for land acquisition could be greater than estimated. Higher land costs would make the proposed storage project(s) even more costly and more difficult to justify on the basis of flood control benefits alone.

9.5.8 Environmental and Socio-Economic Impacts

Construction of the proposed dams and reservoirs will have substantial physical, biological and socio-economic environmental effects on the surrounding area that may make these options unfeasible. The reservoirs considered were previously studied in the late 1960s and early 1970s; and, as such, the environmental issues resulting from the construction and then operation of the proposed reservoirs were not addressed in detail at that time. For this reason, and at this conceptual level phase of analysis, to identify key environmental issues associated with the construction of these reservoirs, the information presented, as well as the public responses noted during: (a) the recent technical workshop (2013), (b) the recent Ste. Agathe Detention Structure evaluated as part of the Flood Protection for Winnipeg (2000) (KGS Group, 2000b), and (c) the public open houses performed as part of the Treherne Dam Pre-Feasibility Study (July 2002 and in 2009) (KGS Group and PMA, 2002) were used for the current study. The environmental screening and public comments noted during these studies were similar and, therefore, were deemed generally representative of the proposed reservoirs in the prairie region. Consequently, the environmental screening along with the public comments and issues

identified in these studies were considered applicable to the reservoirs assessed for the current study.

The impoundment of the reservoirs will inundate large areas of land; regulated release of flow will change the flow regime downstream potentially affecting fish and fish habitat; and the dams would block fish passage and the navigability of the rivers. As such, prior to construction, approvals would be required under the *Canadian Environmental Assessment Act* (CEAA, 2012), *Fisheries Act*, *Navigation Protection Act*, and the *Manitoba Environment Act*. The project would also need to conform to many other federal and provincial acts and regulations including but not limited to the *Environmental Protection Act*, *Wildlife Act*, *Endangered Species and Ecosystems Act*, *Heritage Resources Act*, and *Water Protection Act*.

While a comprehensive environmental impact assessment would be required to obtain the necessary project approvals, for the purpose of this study, a high level environmental scoping was conducted to identify the most probable issues and effects. The more substantial potential environmental issues and effects associated with construction of the studied dams and reservoirs include:

- Loss of substantial terrestrial habitat, agriculture land and First Nation reserve land:
  - Construction of the Holland, Alexander, Zelena and Victor reservoirs is expected to inundate approximately 7,500, 26,000, 1,800, and 2,000 ha (18,500, 65,150, 4,450, 4,900 ac) of land, respectively.
  - The majority of the land is under agricultural production and would require substantial land acquisition and relocation of buildings and farmsteads. During the public open houses in 2002 and again in 2009 for the proposed Boyne River Dam located at Treherne, the inundated reservoir area was only 384 ha (950 ac) (which is many times less than any of the proposed reservoirs) but several existing land owners responded in writing that they would do everything possible legally and politically to ensure that such a project was never implemented. Similarly, there was very strong opposition voiced during the technical workshop, with the attendees questioning why these dams and reservoirs were even being considered as an option for flood protection. Continuing to consider this option as part of any future EIA would substantially delay or stop the approval process.
  - Construction of the Alexander Dam would inundate a portion of the Sioux Valley Dakota Nation, which would result in strong opposition and require a special agreement that could substantially delay or stop the approval process.
- Alteration of aquatic life and habitat along the rivers:

- Construction of the dam will obstruct the movement of fish and reduce or eliminate access to spawning or wintering grounds. Based on the recent licensing of the replacement of the 1st Avenue Dam on the Assiniboine River at Brandon, Department of Fisheries and Oceans (DFO) appears very focused on limiting any further blockage of fish movement and passage on the Assiniboine River.

- Formation of the reservoir will change the riverine habitat into a lake habitat, which will change the habitats and natural productive capacity of different fish species, result in increased sediment and debris accumulation upstream of the dam and deprivation of sediment downstream of the dam.

- The reservoir and dams will reduce spring flows downstream of the structures, which can adversely affect channel maintenance and fish spawning habitat.

- Impede navigation along the rivers:

  - While only the Assiniboine River is on the list of scheduled waters under the *Navigation Protection Act*, the other rivers are used for recreational purposes.

- Quality of water both in and downstream of the new reservoir:

  - Impoundment and flooding of terrestrial habitat can increase mercury levels in the water and bioaccumulate in fish.

  - Impoundment and flooding of agricultural land will increase nutrient levels in the water as they leach from the agricultural land. Retention of the water in the reservoir combined with higher nutrient loads can cause algal blooms during summer.

  - Oxygen depletion can occur due to degradation of organic material (dead algae) and water releases from the depth of a reservoir.

- Ground water levels in the immediate vicinity of the impoundment:

  - Reservoir water may seep into groundwater supplies and over time raise the groundwater level affecting yields, salinity and drainage on surrounding agricultural fields.

9.5.9 Conclusions

Based on the results of the analysis documented in this report, the following conclusions have been made:

- Four large dams and reservoirs have been investigated in this study having project costs (2014$) of $591 million for the Alexander Dam, $306 million for the Holland Dam, $162 million for the Victor Dam and $104 million for the Zelena Dam.

- Outflow control using gates at the dam is shown to be effective in attenuating the peak outflows at all dams, although it is an optimistic estimation, which represents the maximum benefits of the reservoir. For this conceptual phase of analysis, it was estimated based on judgment that 75% of the additional maximum peak flow reduction

that can be achieved with the addition of gates to the reservoir would best represent the actual peak flow reduction capability of the reservoir. Further studies that are beyond this conceptual phase of analysis are necessary to determine if this 75% value is achievable.

- The Alexander Dam provides the largest peak flow reduction capacity on the Assiniboine River in Brandon (45% to 78%), followed by the Victor Dam (4% to 8%) and the Zelena Dam (2% to 4%). On the Assiniboine River near Holland, the Alexander Dam also provides the largest peak flow reduction (28% to 32%), followed by the Holland Dam (10% to 17%), the Victor Dam (3% to 5%) and the Zelena Dam (2%).
- Construction of the proposed dams and reservoirs is expected to be very challenging to license and to legally obtain the flooded areas. Consequently, these reservoirs are not considered a feasible option for flood protection, as they will have substantial environmental effects on the surrounding area and will receive very strong public opposition. The most substantial environmental issues and effects associated with construction of the proposed dams and reservoirs include loss of agricultural land, loss of First Nation reserve land, blocking fish passages, altering fish habitat associated with the change in flow regime and impeding navigation of the rivers. The loss of agricultural and First Nation land is expected to receive substantial opposition that would delay or stop the approval process.

The Victor Dam is the least likely to proceed due to inter-jurisdictional issues. The Victor Dam and Zelena Dam also have the highest cost per m^3/s of peak flow reduction.

Peak flow reductions at Brandon have limited benefits because the City of Brandon is already implementing a 1,076 m^3/s (38,000 cfs) flood protection via dike upgrades, as described in Section 9.6.

The environmental effects on the surrounding area together with the relatively high costs of the projects would likely render the storage reservoir alternatives difficult to complete and have, therefore, not been identified as preferred alternatives for the current study. These conclusions are consistent with comments received at the technical workshop undertaken at the beginning of this study, when a majority of experts and stakeholders present at the workshop suggested that most of the large storage reservoir projects should not be considered for this study due to environmental concerns and relatively high costs, which would render these types of projects unfeasible.

9.6 DIKING AT BRANDON

9.6.1 Introduction

The City of Brandon, located in southwestern Manitoba, is the second largest city in the Province with a population of approximately 40,000. The Assiniboine River flows easterly through the north edge of the city. The majority of the developed areas of the city are located on higher elevations outside of the Assiniboine River Valley floodplain. However, industrial, commercial, residential and recreational properties / facilities exist within the floodplain, most notably the Brandon Flats area located south of the river between 1st and 26th Streets.

The flooding experienced in the spring and summer of 2011 was unprecedented with respect to flood extent, flood duration and scale. Flooding during the spring of 2011 was anticipated to occur within southern Manitoba due to wet antecedent conditions during the summer and fall of 2010 combined with above average winter snow pack. However, it was the extended flooding due to heavy rainfall events within the Assiniboine River watershed, which created the problematic conditions in Brandon.

It was anticipated that the existing dikes protecting the flood prone areas would be insufficient in height to protect against the forecasted 2011 Flood water levels. The forecasted peak, however, was surpassed resulting in much higher water levels and hastily raised dikes increasing concerns with respect to the integrity of these earthen structures. Similarly, a significant flood event slightly less in magnitude occurred in the spring of 2014 straining mitigation efforts once again in the Assiniboine River Valley and the City of Brandon. Following the 2011 Flood, it was apparent that a greater level of flood protection was required to protect the City of Brandon in the future. AECOM was commissioned to provide designs for raising and reconstructing the existing dikes and to design the additional dikes required to protect additional flood prone areas. Background information and the description of proposed flood protection described in this section are based on the AECOM Report (AECOM, 2013).

Flood protection criteria for the City of Brandon were developed in consultation with the project "Flood Committee" with representatives from MIT and the City of Brandon. This group agreed that a level of protection of 1,076 m^3/s (38,000 cfs), equivalent to the peak of the 2011 Flood (the largest flood on record), should be used for design of the proposed flood protection system.

The design work and construction on various components of the dike works is currently underway, and construction of the 18th Street dike has already been completed.

This section of the report summarizes the hydraulic basis for the design, as well as the dike and infrastructure requirements to meet the 1,076 m^3/s (38,000 cfs) level of protection defined by the Flood Committee. In addition to the committed design for the largest flood on record as described above, the requirements, as well as the incremental costs at a conceptual level, have been defined to increase the level of protection to a flood event with a return period of 1:300 years.

9.6.2 Hydrology

In June 2006, Manitoba Water Stewardship (MWS) completed a detailed review and updating of flood hydrology (MWS, 2006) for the Assiniboine River at Brandon. Based on their results, the peak discharge for the 2011 Flood estimated at 1076 m^3/s (38,000 cfs) was originally approximated to be equal to an event with a 1:300-year return period.

However, updated hydrologic analyses conducted by MIT show that the frequency curve in Brandon has changed since 2006, once the larger floods of 2011 and 2014 were incorporated into the analysis. As shown in Section 5.2, the current and updated frequency curve indicates that the 2011 Flood peak of 1,076 m^3/s (38,000 cfs) now has a return period of 1:140 years. The peak flow for a 1:300-year return period event is now 1,345 m^3/s (47,500 cfs). Table 9.12 summarizes the return period of different Assiniboine River flood events in Brandon.

Table 9.12
Assiniboine River in Brandon Frequency Curve

Percent Change of Exceedance	Return Period in Years	Flow	
		m^3/s	cfs
0.20%	500	1563	55,200
0.33%	300	1345	47,500
0.50%	200	1198	42,300
1.00%	100	968	34,200
2.00%	50	773	27,300

Percent Change of Exceedance	Return Period in Years	Flow	
		m^3/s	cfs
3.00%	33	668	23,600
5.00%	20	549	19,400
10.00%	10	405	14,300
20.00%	5	278	9,800
30.00%	3.3	211	7,460
50.00%	2	134	4,720
May 10, 2011 Peak* (0.7%)	140	1076	38,000

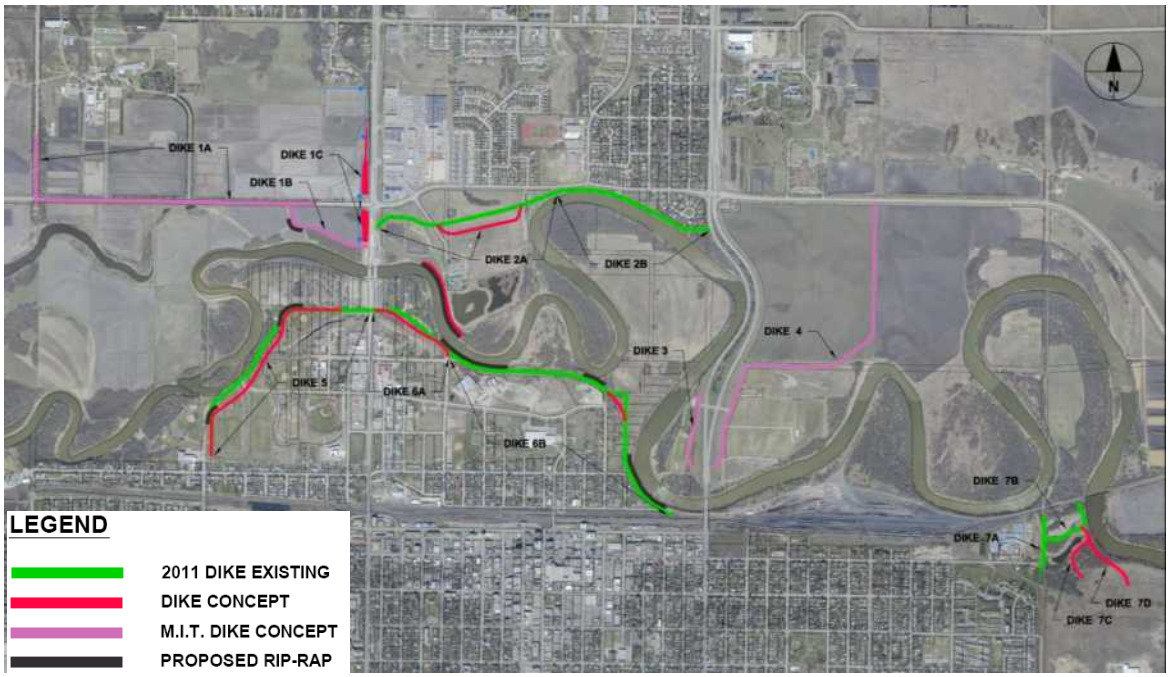
*Estimated discharge at 1st Street Bridge based on meterings.

9.6.3 Proposed 1076 m^3/s Flood Protection Scheme

9.6.3.1 General

As part of the proposed dike renewal scheme, existing dikes will require upgrading for general realignment and also to meet the required flood elevation heights for the design flood event of 1,076 m^3/s (38,000 cfs). Secondly, the scheme will consider the construction of new dikes. The proposed design flood protection scheme was taken from the AECOM Report (AECOM, 2013). The alignment of the existing dikes and of the dikes investigated by AECOM for the renewal project are shown on Figure 9.80 (AECOM, 2013).

Figure 9.80
Alignments of Dikes Investigated for a 1,076 m^3/s Flood Protection



Upgrades to the existing dikes include the following options:

- South Existing Dikes – 26th Street North to 5th Street (dikes 5, 6A and 6B).
- North Existing Dikes – Kasiurak Bay / 1st Street to 18th Street North (dikes 2A and 2B).

Upgrades are required along the existing dike for the following reasons:

- Increase in dike height to predicted flood water elevations for a 1076 m^3/s (38,000 cfs) flood.
- Improvements to dike slope sections constructed under emergency conditions.
- To provide continued and future vehicle access along the top of the dikes.
- To reduce the cross sectional area of dike exposed to flood waters, and thus reduce erosion.

Proposed construction of the new dikes includes the following options:

- 17th Street East Dikes (dikes 7A, 7B, 7C and 7D).
- Various locations along the South Existing Dike (dikes 5, 6A, and 6B).

- 1st Street Dikes (dikes 3 and 4).
- 18th Street North Dikes (dikes 1A, 1B and 1C).

The proposed construction of the 1st and 18th Street north dikes was initially based on design concepts by Manitoba Infrastructure and Transportation (MIT), which have since been incorporated into the City of Brandon's proposed flood protection scheme. The recommended option presented by AECOM in their report (AECOM, 2013), which formed the basis of the cost estimate summarized in Section 9.6.4.4 consisted of conducting upgrades and construction of new dikes along alignments 1C, 2A, 2B, 5, 6A, 6B and 7A shown on Figure 9.80.

9.6.3.2 New Dike Construction

Construction of flood protection dikes along 1st and 18th Street are proposed to be completed with preliminary design side slopes of between 4.0(H):1.0(V) and 5.0(H):1.0(V). The design of the dikes will be completed to ensure that the design factor of safety is reached (i.e., greater than 1.5 in the permanent condition). As ground conditions across the flood plain are highly variable spatially, the dikes will be constructed on a location specific basis, as the requirements for dike height and crest width vary with the intended end use (i.e., vehicle access requirements and final flood elevations).

9.6.3.3 Upgrades to Existing Dikes

Upgrades to the existing dikes are considered necessary to meet the required flood elevations for a 1,076 m^3/s (38,000 cfs) flood event. Several areas of the existing dikes were observed to be deficient in regards to stability and seepage control. Correction of these deficiencies, as well as overall improvements and upgrades to the existing dikes, is required to eliminate these potential problem areas and to provide appropriate reliability with regard to the updated protection scheme.

9.6.3.4 North Dikes – Impacts to Existing LDS

The Land Drainage System (LDS) on the north side of the Assiniboine River was evaluated by AECOM for potential upgrades and enhancements to minimize the number of outfalls through the existing dikes. This would reduce the risk of the river backing up at the existing outfalls and

flooding areas upstream of the dike. At the conceptual design level, it was proposed that the five existing land drainage sewer outfalls to the north of the Assiniboine River be removed. This work will include the upgrading of the existing system by intercepting the existing outfalls where possible and reducing the number of outfalls required. A control structure and retention pond will also be incorporated at the new outfall to protect the existing infrastructure from surcharge should a flood of up to 1,076 m^3/s (38,000 cfs) occur. The retention pond would be utilized as temporary storage to minimize the size of the outfall piping required, as well as acting as a pumping chamber during a flood event.

9.6.3.5 Existing Infrastructure and Utilities

Upgrades to the existing dike will include changing the side slopes; and this will affect the footprint of the dikes and the existing infrastructure such as LDS, WWS, street lighting, MTS and Hydro.

Due to the variability of the utility information in the areas and poor record information acquired from the various utility companies, calculating an actual construction cost for the removal and rerouting of the existing utilities was not completed at this stage of the work. An allowance equal to five percent (5%) of the overall construction cost was assumed to be required to abandon and/or reroute the existing utilities.

9.6.3.6 Hydraulic Impact Associated with Dike System Upgrades

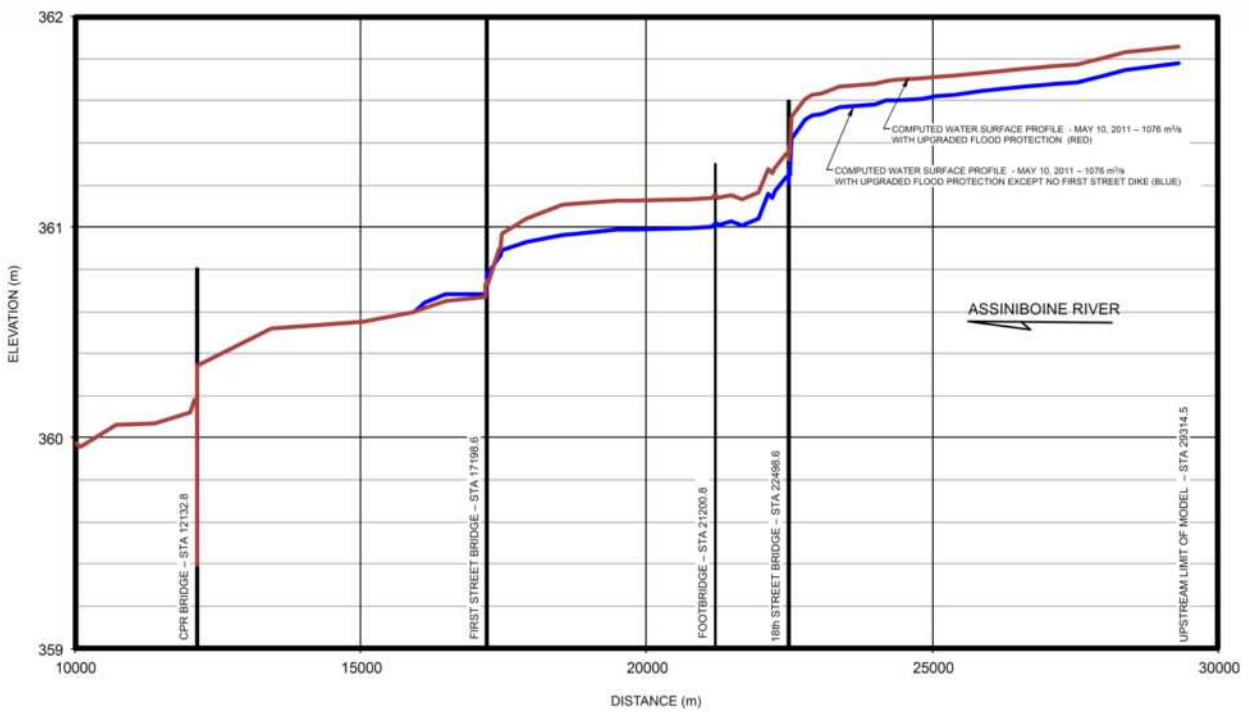
The proposed realignment of the Brandon dikes and the addition of new dikes along Grand Valley Road and at 1st Street, as shown on Figure 9.80, will influence the water surface profile on the Assiniboine River. The proposed dike realignment and the addition of the Grand Valley Road and 1st Street dikes have been incorporated into the backwater model to enable the estimation of the water surface profile with these changes included.

The 1st Street dikes, although protecting a large area, are for the most part outside of the effective flow area of the river and, therefore, do not alter the water surface profile directly due to encroachment. However, the dikes would prevent overtopping of 1st Street, which could occur for flows greater than a 2% event (1:50 year return period) if permanent or temporary diking measures are not taken. The temporary diking of 1st Street to prevent overtopping during these

larger events restricts the river flow up to the 1st Street Bridge opening. This flow constriction would result in greater losses through that section of river and higher water levels upstream of 1st Street.

The 1st Street dikes provide flood protection and maintain traffic on 1st Street. A sensitivity analysis was undertaken by AECOM (2013) to assess the water level reduction if the 1st Street dikes were not constructed, permitting flow overtop of 1st Street during an extreme flood event. The water surface profile, with no 1st Street dikes versus conditions with the proposed dikes, is presented in Figure 9.81. Water levels during a flood event equivalent to the 2011 Flood are decreased by approximately 0.13 m (0.43 ft) between the 1st Street Bridge and the 18th Street Bridge and approximately 0.10 m (0.33 ft) upstream of the 18th Street Bridge if the 1st Street dikes were not constructed.

Figure 9.81
Assiniboine River Water Surface Profile with and without 1st St Dikes



9.6.3.7 Cost Estimate

Parameters and Criteria

The cost estimate was based on the AECOM report (2013). Although not totally consistent with the Economic Guidelines presented in Section 7.0, it was believed to be the most appropriate estimate to use given that it is on the public record. The cost estimate for the upgrades to the existing dike network was prepared by AECOM and considered the following information:

- Material quantities for the upgrades to the dikes were based on a typical dike cross section as indicated on the drawings, with typical side slopes of 5:1 using suitable material as per the geotechnical section of this report.
- Riprap quantities were based on the assumption of Class 350 rock for bank protection.
- Five percent overall construction costs were assumed to be required to move or re-route existing utilities infrastructure.
- Fifteen percent allowance for the compaction of dikes.
- Five percent allowance for unsuitable material within the existing dikes.
- No landscaping or aesthetic costs other than seeding were included in this estimate.
- The abandonment of existing walkways and light standards were to be dealt with directly by the City of Brandon.
- No upgrades were currently required to the existing wastewater sewer systems within the City of Brandon.
- The land drainage work on the north dike was conceptual only. The size and location of the proposed new system would be defined during the detailed design, and the cost estimates would be updated at that stage.
- All previous information prepared by government departments was accurate.
- Total costs in the cost estimate were rounded off to the nearest $100.
- The unit prices listed in the cost estimate were obtained from a review of prices received on similar projects.
- Actual unit prices would be dependent upon tendering.
- Retaining wall structure options at Kasiurak Bay were assumed to be required for the full 400 m (1,312 ft) length to protect existing properties from flooding.

Administration and contingency line items were included per Manitoba Water Services Board (MWSB) standard requirements. These line items were intended to cover the following typical project expenditures:

- Administration costs.
- Construction delays.
- Design changes due to onsite conditions.
- Adverse weather conditions, etc.
- Interim interest charges.
- Possible survey and legal fees.

The cost estimate developed by AECOM (2013) was developed at a conceptual level. The quantities were roughly estimated based on limited survey information. The unit rates were based on discussions with local contractors. Total costs of the recommended option, including contingencies were estimated to be $24 million. A summary of the costs by AECOM (2013) has been provided in Table 9.13.

Table 9.13
Summary of Estimated Costs
for 1076 m^3/s Flood Protection in Brandon

Section	Proposed Work	Estimated Cost of Recommended Option
	North Dike	
1.0	Mobilization / Demobilization	$ 50,000
4.0	Dike 1C	$ 516,000
5.0	Dike 2A	$ 302,300
6.0	Dike 2B - Sheet Pile Wall Option	$ 3,256,000
	South Dike	
10.0	Dike 5	$ 1,219,400
10.1	Dike 5 - Riprap	$ 243,600
11.0	Dike 6A	$ 1,810,700
12.0	Dike 6B	$ 602,800
12.1	Dike 6B - Riprap	$ 391,500
13.0	Dike 7A	$ 348,300
	Recommended Option - North & South Dike Total Cost	$ 8,740,600
	Rerouting of existing Utilities (5% of construct ion cost)	$ 657,400
	PTH 110 Dike Cost Estimate (Provided by MIT)	$ 2,500,000
	North side LDS	$ 2,970,100
	Lift station and forcemain to abandon the existing	$ 4,167,300
	15% Engineering Fees	$ 2,480,300
	15% Contingencies	$ 2,480,300
	TOTAL	**$ 24,000,000**

9.6.4 Conceptual Design of a 1:300 Year Flood Protection Scheme

9.6.4.1 General

As discussed in Section 9.6.2, the 1,076 m^3/s (38,000 cfs) flood protection level was originally believed to equal approximately a flood event with a 1:300 year return period. However, based upon an updated hydrologic analyses conducted by MIT that now incorporates the larger floods of 2011 and 2014, the 1:300-year flood is equivalent to a flow of 1,345 m^3/s (47,500 cfs) on the Assiniboine River at Brandon.

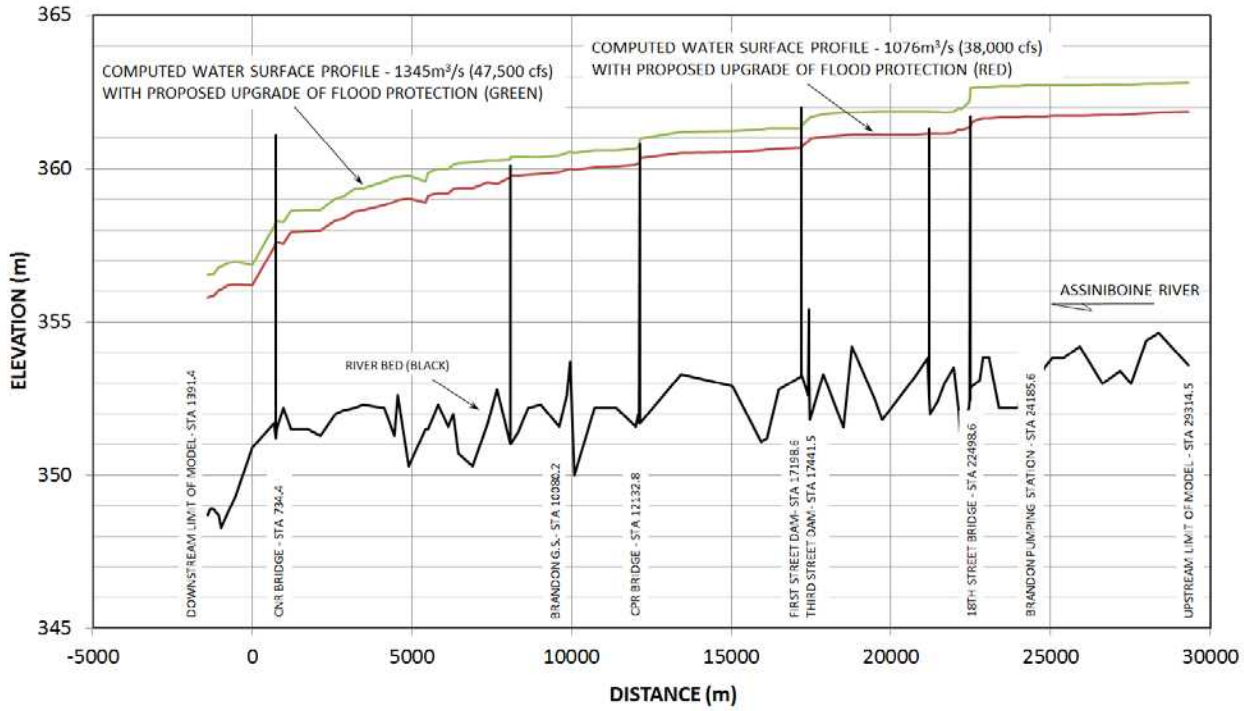
The requirements, as well as the incremental costs at a conceptual level, have been defined to increase the level of protection to the current 1:300 year return period event. The 1,345 m^3/s

(47,500 cfs) flood protection scheme and incremental costs were based on the recommended dike arrangement and concept proposed by AECOM (2013) for the 1,076 m^3/s (38,000 cfs) flood protection as presented in their report. Dike height requirements were based on the hydraulic analysis described below.

9.6.4.2 Hydraulic Considerations

The hydraulic requirements were defined with the HEC-RAS Model that was discussed in Section 8.3.3. Dike level requirements for increased flood protection were defined for a discharge of 1,345 m^3/s (47,500 cfs) and compared to the water surface profiles for the 1,076 m^3/s (38,000 cfs) event. The results are shown on Figure 9.82. Based on the results of the hydraulic analysis, it was determined that the 1,345 m^3/s (47,500 cfs) profile was on average 0.8 m (2.6 ft) higher than the 1,076 m^3/s (38,000 cfs) profile.

Figure 9.82
Assiniboine River Water Surface Profile for 1:300 Year Flood Event



9.6.4.3 Dike Requirements

Dike requirements for the 1,345 m^3/s (47,500 cfs) flood protection are based on "extending" the AECOM 1,076 m^3/s (38,000 cfs) flood protection scheme. Since a continuous profile of the dikes was not available with the AECOM report (2013), it was assumed that, for the 1,345 m^3/s (47,500 cfs) flood protection, the entire dike infrastructure would need to be raised by 0.8 m (2.6 ft). As discussed above, this is equivalent to the average increase in the water surface profile between the two events.

Incremental dike volumes for the 1,345 m^3/s (48,500 cfs) flood are based on the difference in area between the 1,076 m^3/s (38,000 cfs) level of protection and the 1,345 m^3/s (47,500 cfs) level of protection. The layout of the dikes shown on Figure 9.80 was used to determine the total length of dikes that required upgrades. Based on typical cross sections and data available in the AECOM report (2013), it was estimated that the average height of dikes for the 1,076 m^3/s (38,000 cfs) flood protection ranged between 2.1 m (6.9 ft) and 4.8 m (15.7 ft) depending on location and assumed 5:1 side slopes at all locations. The 5:1 slopes are also consistent with AECOM's design criteria discussed in Section 9.6.3.7. As per AECOM's analysis, a 15% allowance was considered for the compaction of the dikes. The 5% allowance for unsuitable material within the existing dikes was not re-incorporated into the quantities as this was already assumed to be part of the original 1,076 m^3/s (38,000 cfs) dike design.

For Dike 2B, the length of dike that required a sheet pile wall was based on the 0.8 m (2.6 ft) height increase discussed in Section 9.6.4.2 and the 400 m (1,312 ft) length identified by AECOM (2013).

9.6.4.4 Incremental Cost for 1:300-Year Protection

Incremental costs are based on the flood protection requirements described above. A unit price of \$20/m^3 was applied to the dike volume quantities to estimate the cost for supply, placement and compaction of clay borrow, as outlined in the Economic Guidelines (Section 7.0) for this study and was consistent with the base costs developed by AECOM (2013) for the 1,076 m^3/s (38,000 cfs) flood protection scheme. A unit rate of \$1,200/m^2 was used for the sheet pile wall to be consistent with AECOM's (2013) cost estimate.

Incidental costs such as clearing and grubbing, supply and placement of riprap, installation of silt fence, and ditching, to name a few, were incorporated into the estimate by applying a 30% increase to the cost for supply, placement and compaction of clay borrow as outlined previously. This factor was based on a review of AECOM's detailed cost estimate.

To remain consistent with AECOM's (2013) cost estimate, a 5% allowance of the total north and south dike costs was added to the estimate for rerouting of the existing utilities. An allowance was not included for the PTH 110 dikes, as no information was available on MIT's 1,076 m^3/s (38,000 cfs) flood protection at this location. It was also assumed that no additional costs would be incurred at the 1,345 m^3/s (47,500 cfs) flood protection for the north side land drainage system and for the lift station and forcemain to abandon the existing siphon.

As per the Economic Guidelines (Section 7.0) for this study, a 20% allowance of the total direct cost was added to account for indirect costs such as engineering, approvals, administration and owner's costs. A contingency allowance of 30% of the direct and indirect cost was added for uncertainty. The 20% and 30% allowances are slightly larger than AECOM's (2013) estimate for the 1,076 m^3/s (38,000 cfs) flood protection scheme; however, considering the uncertainties and limited data available to compute the incremental dike volume, the larger allowance values were deemed appropriate for this conceptual level estimate. The incremental cost to increase the flood protection level from the proposed 1,076 m^3/s (38,000 cfs) flood of record to the 1:300 year return period is estimated to be approximately $11 million. The detailed estimate is provided in Table 9.14.

Table 9.14
Summary of Estimated Incremental Costs for
1:300-Year Flood Protection in Brandon

Proposed Work	Units	Quantity	Unit Price	Estimated Incremental Cost
Dike 1C	m^3	16,400	$20 / m^3	$ 328,000
Dike 2A	m^3	33,700	$20 / m^3	$ 674,000
Dike 2B – Without Retaining	m^3	29,100	$20 / m^3	$ 582,000
Dike 2B – Sheet Pile Wall	m^2	320	$1,200 / m^2	$ 384,000
Dike 5	m^3	60,400	$20 / m^3	$ 1,208,000
Dike 6A	m^3	19,400	$20 / m^3	$ 388,000
Dike 6B	m^3	69,000	$20 / m^3	$ 1,380,000
Dike 7A	m^3	14,700	$20 / m^3	$ 294,000
			Incidentals Costs	$ 1,456,000
			Sub Total of Incremental Cost - North & South Dike	**$ 6,694,000**
			Rerouting of existing Utilities (5% of construction cost)	$ 335,000
			20% Indirect Costs	$ 1,406,000
			30% Contingencies	$ 2,530,000
			Total Incremental Cost	**$ 11,000,000**

The total incremental cost of $11 million for 1:300-year flood protection assumes that the work can be added to the $24 million project for the proposed 1,076 m^3/s (38,000 cfs) flood protection scheme described in Section 9.6.3 and that construction would be completed at the same time, minimizing the need for the Contractor to remobilize. However, construction on various components of the proposed 1,076 m^3/s (38,000 cfs) flood protection scheme is currently underway, and construction of the 18th Street dike has already been completed. The cost to increase the flood protection to a 1:300 year level independently of the ongoing project and after construction has been completed would be higher since several components of construction, such as mobilization, stripping of existing top soil and revegetation would have to be re-done.

9.6.5 Conclusions

1. The 1,076 m^3/s (38,000 cfs) flood protection upgrades to the City of Brandon outlined in the AECOM report (2013) provide a 1:140 year level of protection, which is equivalent to the highest flood on record (2011).

2. The flood event in the City of Brandon, which has a return period of 1:300 years, has a flow of 1,345 m^3/s (47,500 cfs), based on frequency curves that now include both the 2011 and 2014 floods.

3. At this conceptual level of design, it is practical to base the 1:300-year flood protection scheme of 1,345 m^3/s (47,500 cfs) on "extending" the 1,076 m^3/s (38,000 cfs) flood protection scheme defined by AECOM.

4. Based on the HEC-RAS Model, the 1:300 year flood level of 1,345 m^3/s (47,500 cfs) is approximately 0.8 m (2.6 ft) higher than the flood of record of 1,076 m^3/s (38,000 cfs).

5. Incremental costs, including contingencies, to increase the level of protection from the proposed 1,076 m^3/s (38,000 cfs) scheme to the 1:300 year flood level of 1,345 m^3/s (47,500 cfs) are estimated to be $11 million.

6. The cost to increase flood protection to a 1:300 year level independently of the ongoing 1,076 m^3/s (38,000 cfs) protection scheme and after construction has been completed would be higher since several components of construction would have to be re-done.

DRAWINGS

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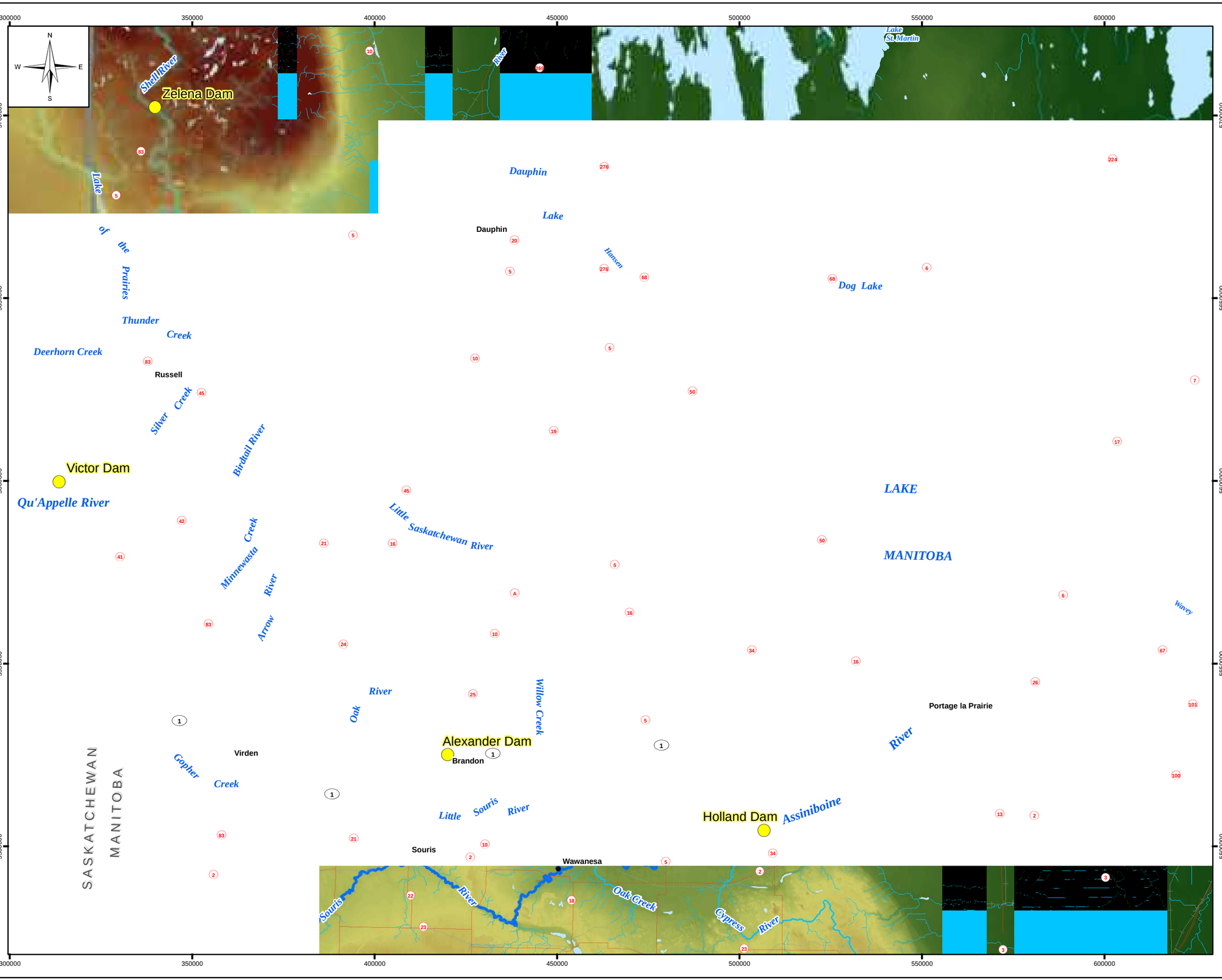


LEGEND:

- Large Storage Reservoir
- Rail Line
- Road
- Major River
- Other Streams
- Waterbody
- Provincial Forest
- Province or State

10 0 10 20 30 40
Kilometers

SCALE: 1:1,000,000METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
0	16/01/04	ISSUED WITH FINAL REPORT	PAL

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

MMM INFRASTRUCTURE

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD

CONCEPTUAL LOCATION OF LARGE DAMS ON ASSINIBOINE RIVER

JANUARY 2016	DWG 9.1	REV: 0

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\Rev0\Final Report\12-0300-011_D09.1_Rev0.mxd
11"x17" PLOT SCALE 1:1

10.0 LOWER ASSINIBOINE RIVER

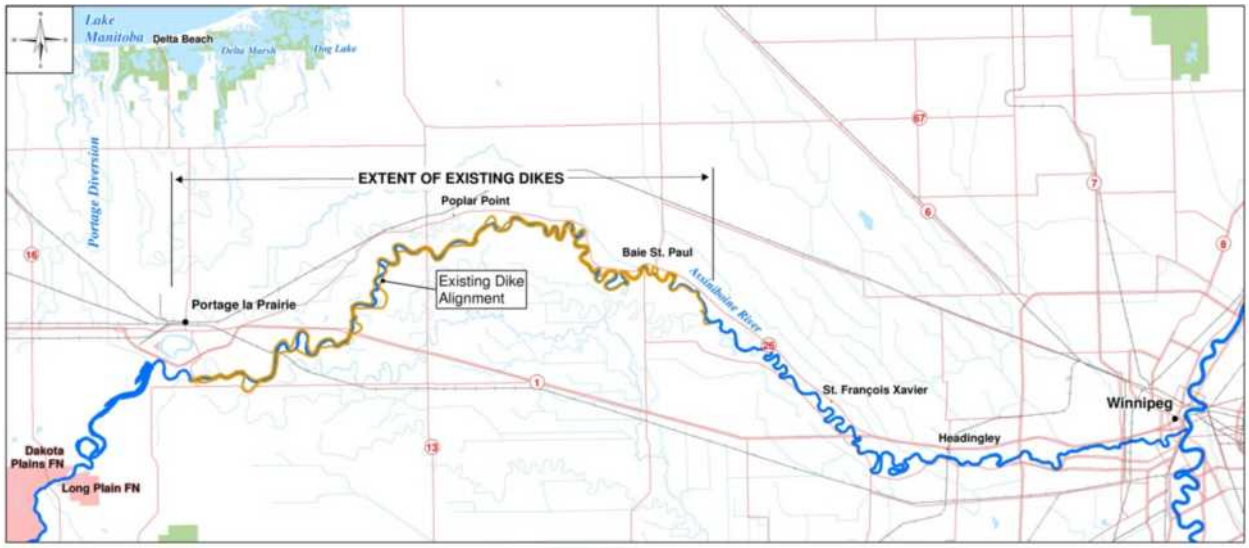
10.1 CONCEPTUAL DESIGN OF IMPROVEMENT SCHEMES FOR THE LOWER ASSINIBOINE RIVER

10.1.1 Introduction / Background

Upgrades to the Lower Assiniboine River capacity from Portage la Prairie to the City of Winnipeg limits in Headingley were considered for the Assiniboine River & Lake Manitoba Basins Flood Mitigation Study. As discussed in Section 4.2, the Lower Assiniboine River was identified as the most vulnerable area within the Assiniboine River & Lake Manitoba Basins, as it had infrastructure that would be vulnerable at flood magnitudes that were much less than both of the largest floods on record, as well as the 1:200 year event.

Along the Lower Assiniboine River are the existing provincial dikes, which consist of a 71.3± km (44.3 mi) long north dike and a 77.3± km (48.0 mi) long south dike that extend from Portage la Prairie to approximately Baie St. Paul. The extent of the existing diking system is shown on Figure 10.1. Currently, there is no continuous diking system downstream from there to Headingley.

Figure 10.1
Plan of Lower Assiniboine River Dikes



The existing dikes were tested to a peak capacity of 637 m³/s (22,500 cfs) for a period of two days in 1976, at a lower flow of about 510 m³/s (18,000 cfs) for two months in 2011, and again at approximately the same flow for approximately two weeks in July 2014. Although the dikes held in 2011, significant repairs and ongoing inspection and maintenance were required, with temporary top up, to safely pass the flow during the flood period (KGS Group, 2013a). The dikes held again in 2014 but also required significant emergency work. KGS Group had concluded in other studies that the 1976 flood peak was of such short duration that they dikes probably were frozen and remained hard frozen through that flood peak. In both 2011 and 2014 the dikes thawed out and steady state seepage conditions were established, probably exposing geotechnical weaknesses in the dikes that would have not been apparent after the much shorter 1976 flood event. The dikes are now under consideration for additional permanent repairs and upgrading (i.e., raising, side slope flattening, seepage control, erosion protection) to safely pass a recommended design flow, the magnitude of which is to be determined as part of the current flood mitigation study.

Minimal damages occurred downstream of the diking system during the 2011 and 2014 floods, although temporary flood mitigation measures such as sandbag dikes were implemented to protect vulnerable properties from flooding. For this study, two options were considered for permanent protection: the first was to extend the existing dikes all the way to the City of Winnipeg limits in Headingley based on an alignment that followed the river, and the second consisted of a combination of neighborhood or ring diking, individual flood proofing and the purchase of vulnerable properties.

This report summarizes the conceptual assessment of improvement schemes for upgrading the Lower Assiniboine dikes and for the reach of river downstream of the existing dikes, with an objective to increase the capacity of the Assiniboine River from Portage la Prairie to the City of Winnipeg limits in Headingley. This report also describes the results of the hydraulic and geotechnical analysis used to estimate the capacity and probabilities of failure of the main areas of the Lower Assiniboine River dikes. The assumptions made in the assessment and cost estimates for the repairs and upgrading of the diking system for a range of flow conditions have also been documented.

10.1.2 Design Water Surface Profiles

The hydraulic analyses of the Lower Assiniboine River were based on two separate HEC-RAS models representing two portions of the Assiniboine River as discussed in Section 8.3.2 of the report:

1. The "upstream" model which extends from Portage la Prairie to approximately St. Francois Xavier.
2. The "downstream" model, which extends approximately from Baie St. Paul to Headingley.

These two models formed the basis for the design upgrades to the Lower Assiniboine River.

For this analysis, flows that exceeded the existing dike height were considered. The geometry in both models was, therefore, modified to simulate the effects of raised dikes by inserting "ineffective areas" in the model, as required, to eliminate the conveyance outside of the dikes. Since most cross sections only extended from the North dike to the South dike, conveyance outside of the dikes was, in general, already excluded; and, therefore, modifications to the geometry were only required at a few locations. The final water surface profiles for various flow conditions along the river are shown in Figure 10.2. The figure also shows the North and South Dike height based on the 2011 LiDAR Survey provided by MIT. Downstream of where the dikes end near Baie St. Paul, the highest point on the north and south banks of the river was taken from the model cross sections and is shown on Figure 10.2.

It should be noted that a small portion of the "upstream" and "downstream" models overlap between Baie St. Paul and St. Francois Xavier. Therefore, the end of the existing provincial dikes was selected as the transition point between the two models for the analysis.

Since the existing North and South Dike alignments do not follow the exact same alignment as the river, a relationship was developed between the river stationing and the North and South Dike stationing to calculate the water levels along the dikes. This was particularly important along meandering sections of the river where the observed gradient along the dikes appeared shorter or longer than that experienced along the centre of the river. The water surface profiles

projected along the North Dike stationing and South Dike stationing for various flow conditions are provided on Figures 10.3 and 10.4.

Figure 10.2
Assiniboine River Water Surface Profiles from Portage la Prairie to Headingley

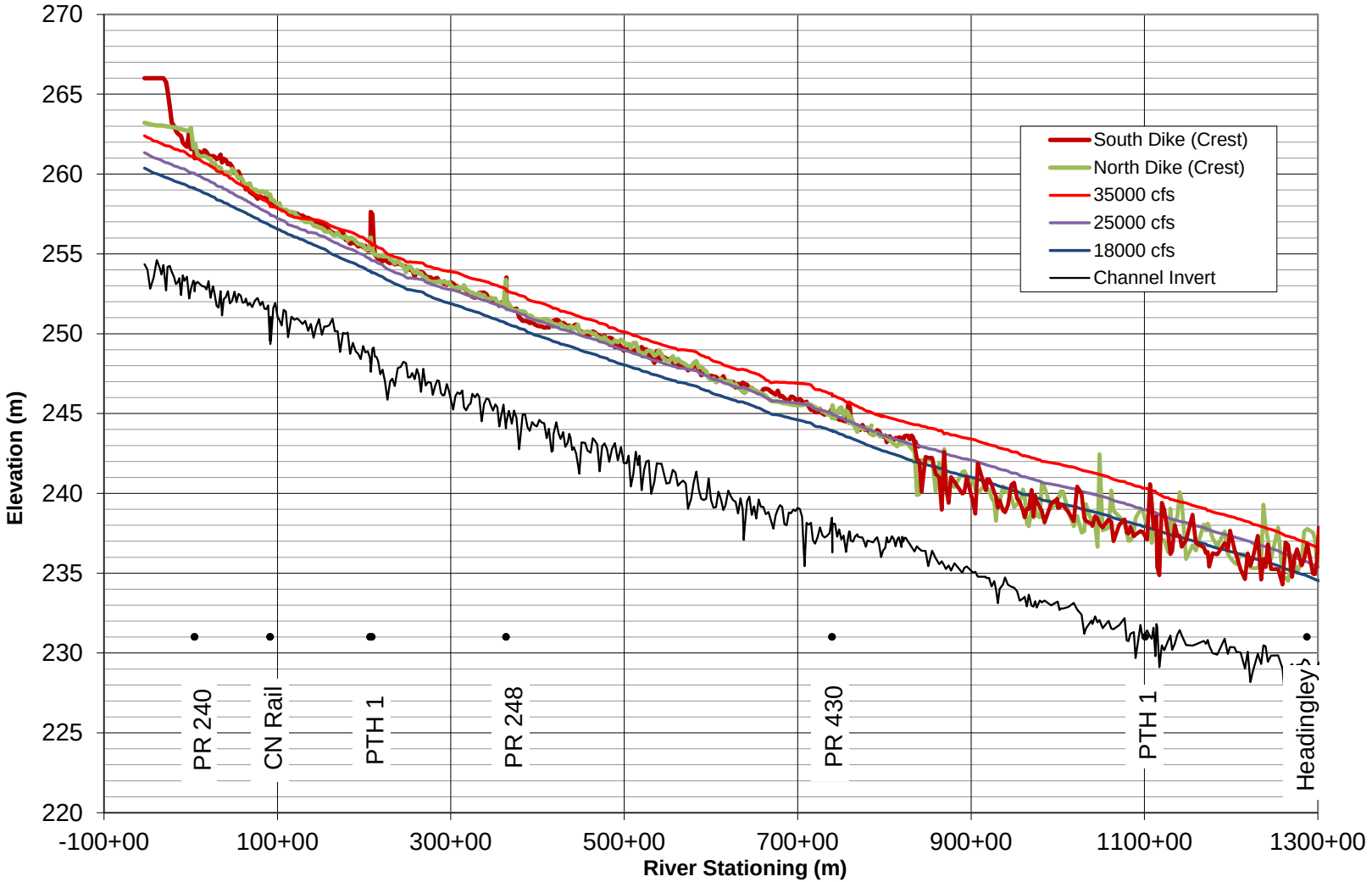


Figure 10.3
Assiniboine River Water Surface Profiles Projected along North Dike Stationing

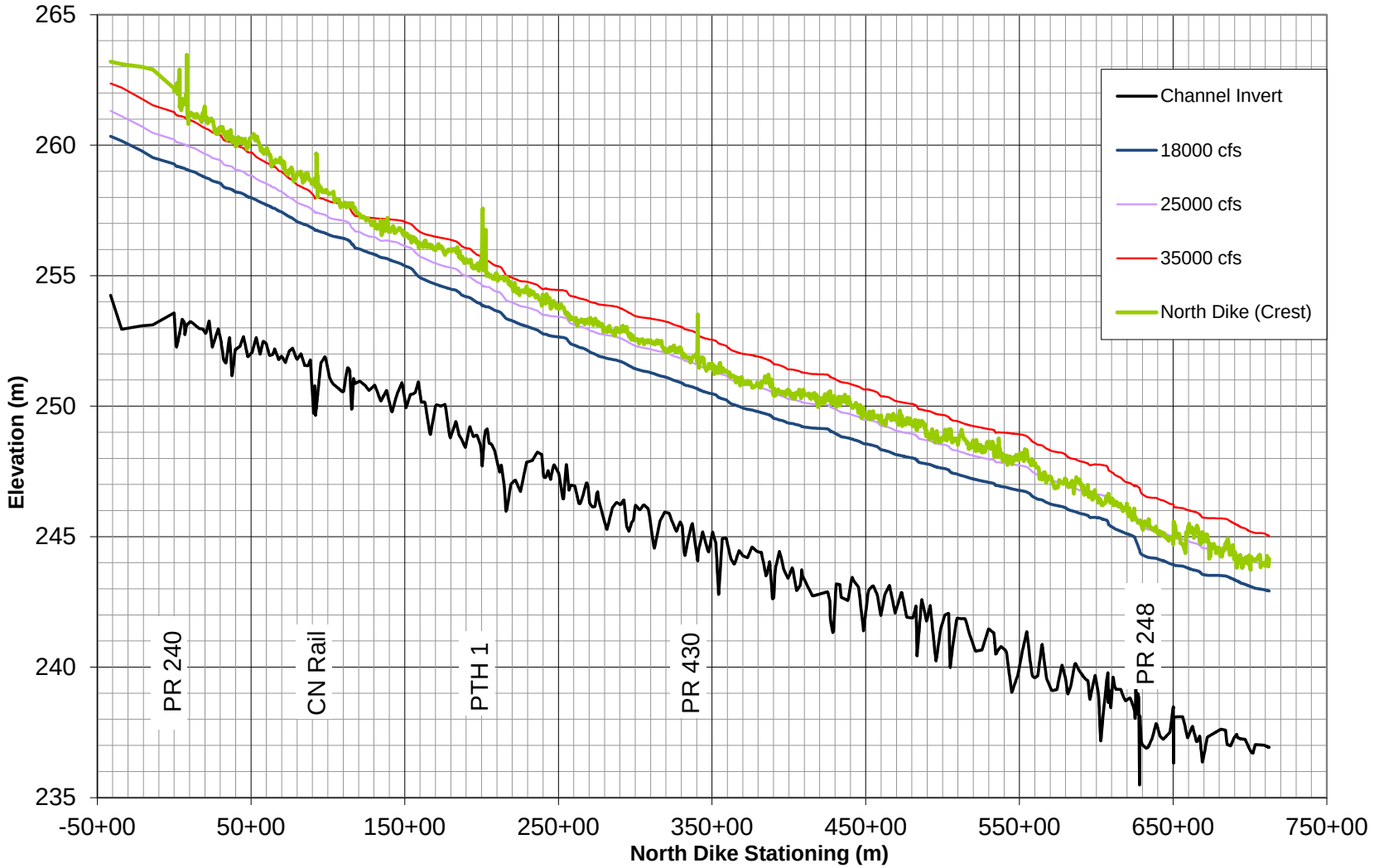
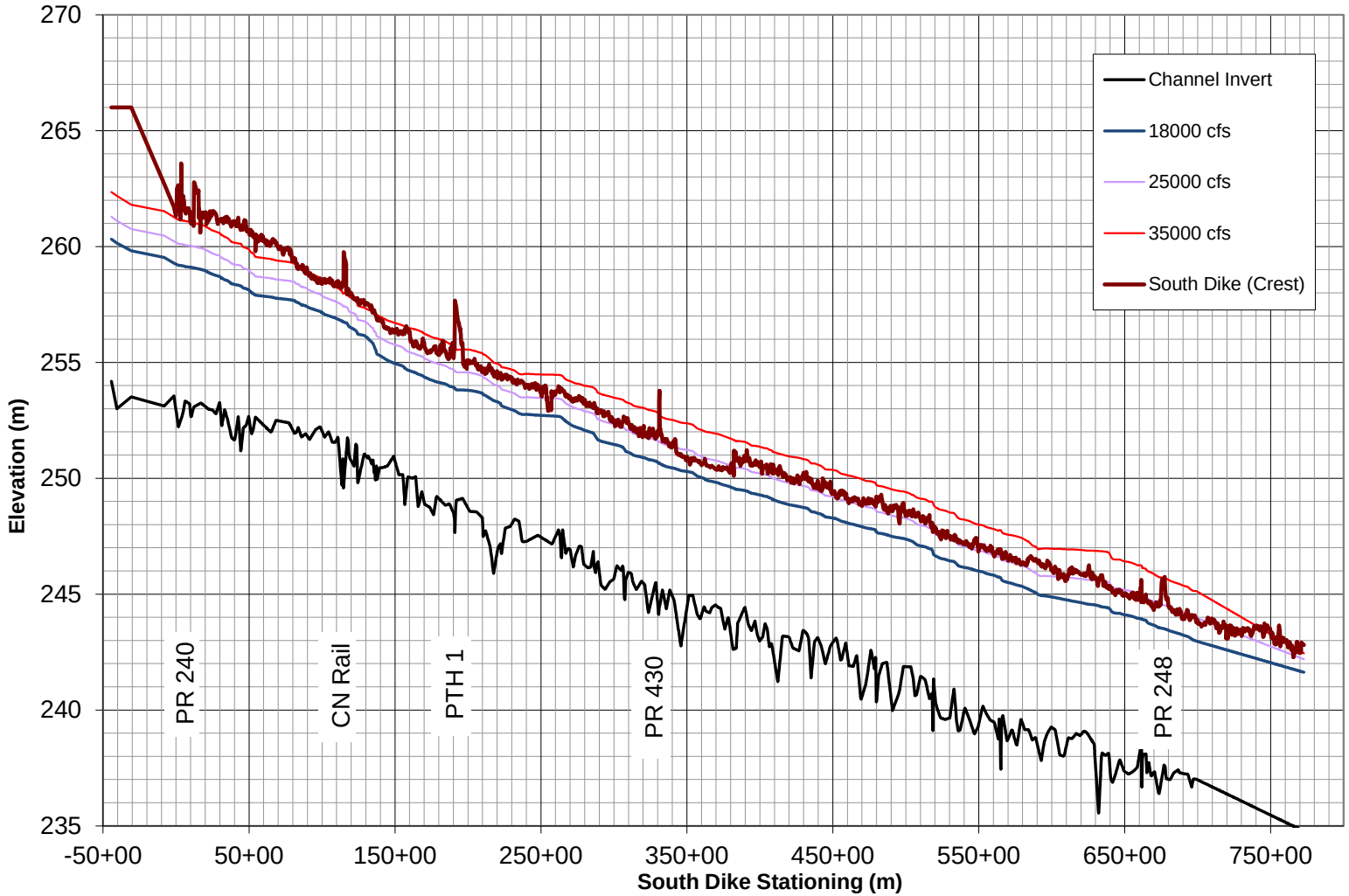


Figure 10.4
Assiniboine River Water Surface Profiles Projected along South Dike Stationing



10.1.3 Reaches of the Diking System

10.1.3.1 2011 KGS Group Flood Report

Based on the Geotechnical Emergency Engineering Services, Assiniboine River Dikes – 2011 Flood Report (KGS Group, 2013a), the existing diking system, which extend from Portage la Prairie to approximately Baie St. Paul, was prioritized into three main reaches for repairs and upgrading. This prioritization was based on their performance during the 2011 Flood event and is summarized in Table 10.1 from upstream to downstream.

Table 10.1
Reaches of the Existing Lower Assiniboine River Diking System

Priority Rating	North Dike	South Dike
3	Southport Bridge to Km 27	Southport Bridge to Km 22
1	Km 27 to Km 43 and Willy Fothe's Farm Site (N1a)	Km 22 to Km 46 and Super Sandbag Site (S1a)
2	Km 43 to Km 71	Km 46 to Km 77

As shown on the table, the middle reach had the highest priority rating of Priority 1. The downstream dike reach was rated as Priority 2 and the upstream dike reach was rated as Priority 3. Those priority reaches are identified on Drawing 10.1. Detailed maps of the existing dikes taken from a 2014 freeboard analysis report by KGS Group (2014a) are also provided in Appendix G. These maps show the alignment and stationing system of the existing dikes.

During the 2011 Flood event, 32 critical sites were identified. These are shown on Drawing 10.1. Most of these sites were located in the middle reach with a Priority 1 rating for repairs and upgrading.

10.1.3.2 Flood Mitigation Study Dike Sections

For the purpose of the current Flood Mitigation Study, the three priority reaches of the existing 145+ km diking system were further divided into ten separate sections. These distinctions were

made primarily from a risk perspective based on the post-2011 Flood conditions of the dikes with respect to various flow conditions in the Assiniboine River.

In addition, two other sections were included in the study for the river reach that extended from the end of the existing diking system near Baie St. Paul to the City of Winnipeg limits in Headingley. At the present time, a continuous diking system parallel to the river does not currently exist along this reach. The twelve sections are listed in Table 10.2 and are also shown on Drawing 10.1.

Table 10.2
Sections of the Lower Assiniboine River Diking System

Dike	Dike Section	Station Location	Referenced Stationing System	Remarks
North Dike	N1a	Km 27 to Km 43	North Dike Alignment	Priority 1 for repairs and upgrading, including the highest risk of failure sites
	N1b	Km 49.5 to Km 51		
	N2a	Km 43 to Km 49.5		Priority 2 for repairs and upgrading
	N2b	Km 51 to Km 71.3		
	N3	Km -4.7 to Km 27		Priority 3 for repairs and upgrading
South Dike	S1a	Km 22 to Km 46	South Dike Alignment	Priority 1 for repairs and upgrading, including the highest risk of failure sites
	S1b	Km 47.3 to Km 48.6		
	S2a	Km 46 to Km 47.3		Priority 2 for repairs and upgrading
	S2b	Km 48.6 to Km 77.3		
	S3	Km -4.4 to Km 22		Priority 3 for repairs and upgrading
No Diking System	N4	Km 71.3 to Km 130	Assiniboine River	
	S4	Km 77.3 to Km 130		

Note: Priority Locations based on the "Geotechnical Emergency Engineering Services, Assiniboine River Dikes – 2011 Flood Report" (KGS Group, 2013a).

The ten existing dike sections identified in Table 10.2 and shown on Drawing 10.1 (N1a to N3 and S1a to S3) were reviewed and analysed to estimate the probability of failure for a range of Assiniboine River flows from 510 m³/s to 991 m³/s (18,000 cfs to 35,000 cfs). In addition, based on the preliminary remedial measures suggested in the Geotechnical Emergency Engineering Services, Assiniboine River Dikes – 2011 Flood Report (KGS Group, 2013a), as well as

additional measures discussed with MIT and summarized in Section 10.1.7 of this report, the estimated cost of repairs and upgrading for the ten dike sections were estimated. The cost to construct new dikes was also estimated where none currently exist along the two other lower sections. These analyses and cost estimates are described in the sections that follow.

10.1.4 Current Condition of the Existing Diking System

The data from the September 2011 LiDAR survey of the existing Assiniboine River Dikes was used to assess the current basic geometry conditions of the diking system. This included the height and the side slopes (both the wet and dry side slopes) of the diking system. Due to the length of the diking system, these geometric features were analysed and "lumped" into broad categories to simplify the assessment. The results of the assessment were used to:

- Estimate the probability of failure of each dike section at various design flow conditions.
- Determine the construction quantities for dike upgrading and remediation, and their cost estimates according to the preliminary remedial design recommendations in the 2011 Flood Report (KGS Group, 2013a).

10.1.4.1 Dike Height

The cumulative length and percentage of each dike section that falls into one of four height categories is listed in Table 10.3. When calculating quantities for the cost estimate, the initial height of the dikes was assumed to be either all 2 m in height (based on the extent of diking that fell into the 0 m to 2 m category) or all 3 m in height (based on the extent that fell into the 2 to 3 m and up categories) relative to the average prairie elevation on the dry side of the dike.

Table 10.3
Height of Existing Dike Sections as of September 2011

Dike	Dike Section	Quantifier	Height Category (m)			
			0 - 2	2 - 3	3 - 4	4 – 6
North	Section N1a	Length (m):	7,200	8,400	400	-
	Km 27 to Km 43	% of Section:	45%	52%	3%	-
	Section N1b	Length (m):	900	600	-	-
	Km 49.5 to Km 51	% of Section:	60%	40%	-	-
	Section N2a	Length (m):	1,400	4,300	800	-
	Km 43 to Km 49.5	% of Section:	22%	66%	12%	-
	Section N2b	Length (m):	13,450	5,800	1,000	-
	Km 51 to Km 71.3	% of Section:	66%	29%	5%	-
	Section N3	Length (m):	27,700	4,000	-	-
	Km -4.7 to Km 27	% of Section:	87%	13%	-	-
South	Section S1a	Length (m):	6,500	15,700	1,800	-
	Km 22 to Km 46	% of Section:	27%	65%	8%	-
	Section S1b	Length (m):	300	1,000	-	-
	Km 47.3 to Km 48.6	% of Section:	23%	77%	-	-
	Section S2a	Length (m):	900	400	-	-
	Km 46 to Km 47.3	% of Section:	69%	31%	-	-
	Section S2b	Length (m):	23,480	5,200	-	-
	Km 48.6 to Km 77.3	% of Section:	82%	18%	-	-
	Section S3	Length (m):	20,800	4,600	600	400
	Km -4.4 to Km 22	% of Section:	79%	17%	2%	2%

10.1.4.2 Dike Side Slopes

The cumulative length and percentage of dike sections falling into one of four side slope categories is listed in Table 10.4 for both the dry and wet side slopes. When calculating quantities for the cost estimate, the dikes in each section were assumed to have constant side slopes equal to the weighted average side slopes listed in Table 10.4.

Table 10.4
Side Slopes of Existing Dike Sections as of September 2011

Dike	Dike Section	Slope	Quantifier	Slope Category (H:1V)				Average Side Slope*
				<1	1 - 2	2 - 3	> 3	
North	Section N1a Km 27 to Km 43	Dry	Length (m):	-	500	7,000	8,500	2.74H:1V
			% of Section:	-	3%	44%	53%	
		Wet	Length (m):	-	7,750	7,750	500	2.01H:1V
			% of Section:	-	48%	48%	4%	
	Section N1b	Dry	Length (m):	-	-	1,500	-	2.5H:1V
			% of Section:	-	-	100%	-	
	Km 49.5 to Km 51	Wet	Length (m):	-	750	750	-	2H:1V
			% of Section:	-	50%	50%	-	
	Section N2a Km 43 to Km 49.5	Dry	Length (m):	-	250	5,650	600	2.51H:1V
			% of Section:	-	4%	87%	9%	
		Wet	Length (m):	-	5,150	1,250	100	1.72H:1V
			% of Section:	-	79%	19%	2%	
	Section N2b Km 51 to Km 71.3	Dry	Length (m):	-	3,000	13,000	4,250	2.46H:1V
			% of Section:	-	15%	64%	21%	
		Wet	Length (m):	-	6,000	11,500	2,750	2.3H:1V
			% of Section:	-	30%	57%	13%	
	Section N3 Km -4.7 to Km 27	Dry	Length (m):	-	300	16,150	15,250	2.73H:1V
			% of Section:	-	1%	51%	48%	
		Wet	Length (m):	-	1,550	13,650	16,500	2.71H:1V
			% of Section:	-	5%	43%	52%	
South	Section S1a Km 22 to Km 46	Dry	Length (m):	500	2,350	10,750	10,400	2.62H:1V
			% of Section:	2%	10%	45%	43%	
		Wet	Length (m):	-	15,250	8,250	500	1.87H:1V
			% of Section:	-	64%	34%	2%	

Dike	Dike Section	Slope	Quantifier	Slope Category (H:1V)				Average Side Slope*
				<1	1 - 2	2 - 3	> 3	
	Section S1b	Dry	Length (m):	-	500	800	-	2.12H:1V
			% of Section:	-	38%	62%	-	
	Km 47.3 to Km 48.6	Wet	Length (m):	50	500	750	-	2.06H:1V
			% of Section:	4%	38%	58%	-	
	Section S2a	Dry	Length (m):	-	-	550	750	2.79H:1V
			% of Section:	-	-	42%	58%	
	Km 46 to Km 47.3	Wet	Length (m):	-	500	800	-	2.12H:1V
			% of Section:	-	38%	62%	-	
	Section S2b	Dry	Length (m):	-	550	13,068	15,000	2.74H:1V
			% of Section:	-	2%	46%	52%	
	Km 48.6 to Km 77.3	Wet	Length (m):	150	3,200	15,200	10,068	2.55H:1V
			% of Section:	1%	11%	53%	35%	
	Section S3	Dry	Length (m):	-	700	13,350	12,350	2.73H:1V
			% of Section:	-	3%	50%	47%	
	Km -4.4 to Km 22	Wet	Length (m):	-	1,850	16,000	8,550	2.59H:1V
			% of Section:	-	7%	61%	32%	

* Calculated as weighted average of 1H:1V, 1.5H:1V, 2.5H:1V, 3H:1V based on % of each category in table.

10.1.4.3 Freeboard Deficiency of Existing Diking System

The Assiniboine River flow rates selected for the analysis included 510 m³/s, 566 m³/s, 708 m³/s, and 991 m³/s (18,000 cfs, 20,000 cfs, 25,000 cfs, and 35,000 cfs). Water surface profiles for these flows were based on the results of the pre-2011 Flood model discussed in Section 10.1.2. A freeboard of 0.6 m (2 ft) was added to each water surface profile to determine the minimum height requirement of the dikes. As discussed in Section 10.1.7.1, the freeboard of 0.6 m (2 ft) was a standard minimum applied everywhere to meet the design guidelines for the dikes agreed with MIT. In general, the existing diking system meets the minimum height

requirement for flows of 510 m³/s and 566 m³/s (18,000 cfs and 20,000 cfs). Although sufficient freeboard exists on the dikes, a number of dike sections (including the critical sites, Willy's Farm site, and the Super Sandbag site) could have stability and high seepage flow issues affecting the reliability of the dikes. As a result, there is still a possible risk of failure. This is discussed in more detail in Section 10.1.5 of this document.

When the discharge is greater than 566 m³/s (20,000 cfs), many areas have insufficient freeboard to meet the 0.6 m (2 ft) freeboard requirement. The cumulative length and percentage of the existing dike sections falling into one of the four freeboard deficiency categories is shown in Tables 10.5, 10.6, and 10.7 for flows of 566 m³/s, 710 m³/s, and 991 m³/s (20,000 cfs, 25,000 cfs, and 35,000 cfs), respectively. Additional information on dike freeboard can also be obtained from the drawings included in Appendix G. The drawings show the freeboard condition of the existing dikes for the 1976 flood event (flow of approximately 637 m^3/s or 22,500 cfs) and the 2011 flood event (flow of approximately 510 m^3/s or 18,000 cfs).

When calculating quantities to address deficient dyke sections, the proportions in Tables 10.5, 10.6, and 10.7 were used to determine the final height of the dikes. The four freeboard deficiency categories were 0.25 m, 0.75 m, 1.25 m, and 1.75 m (0.82 ft, 2.41 ft, 4.10 ft, and 5.74 ft). The final dike height dictated the appropriate side slopes used to determine the extra footprint area and dike fill required for the cost estimate.

Table 10.5
Freeboard Deficiency for Existing Dike Sections at 20,000 cfs Design Discharge

Dike	Dike Section	Quantifier	Freeboard Deficiency Category (m)				Cum.
			0 - 0.5	0.5 - 1	1 - 1.5	1.5 - 2	
North	Section N1a	Length (m):	-	-	-	-	**-**
	Km 27 to Km 43	% of Section:	-	-	-	-	**-**
	Section N1b	Length (m):	-	-	-	-	**-**
	Km 49.5 to Km 51	% of Section:	-	-	-	-	**-**
	Section N2a	Length (m):	-	-	-	-	**-**
	Km 43 to Km 49.5	% of Section:	-	-	-	-	**-**
	Section N2b	Length (m):	1,395	-	-	-	**1,395**
	Km 51 to Km 71.3	% of Section:	7%	-	-	-	**7%**
	Section N3	Length (m):	-	-	-	-	**-**
	Km -4.7 to Km 27	% of Section:	-	-	-	-	**-**
South	Section S1a	Length (m):	1,717	-	-	-	**1,717**
	Km 22 to Km 46	% of Section:	7%	-	-	-	**7%**
	Section S1b	Length (m):	-	-	-	-	**-**
	Km 47.3 to Km 48.6	% of Section:	-	-	-	-	**-**
	Section S2a	Length (m):	-	-	-	-	**0**
	Km 46 to Km 47.3	% of Section:	-	-	-	-	**0%**
	Section S2b	Length (m):	159	-	-	-	**159**
	Km 48.6 to Km 77.3	% of Section:	1%	-	-	-	**1%**
	Section S3	Length (m):	-	-	-	-	**-**
	Km -4.4 to Km 22	% of Section:	-	-	-	-	**-**

Table 10.6
Freeboard Deficiency for Existing Dike Sections at 25,000 cfs Design Discharge

Dike	Dike Section	Quantifier	Freeboard Deficiency Category (m)				Cum. Deficient
			0 - 0.5	0.5 - 1	1 - 1.5	1.5 - 2	
North	Section N1a	Length (m):	14,742	-	-	-	**14,742**
	Km 27 to Km 43	% of Section:	92%	-	-	-	**92%**
	Section N1b	Length (m):	1,391	-	-	-	**1,391**
	Km 49.5 to Km 51	% of Section:	93%	-	-	-	**93%**
	Section N2a	Length (m):	5,528	-	-	-	**5,528**
	Km 43 to Km 49.5	% of Section:	85%	-	-	-	**85%**
	Section N2b	Length (m):	14,209	3,882	-	-	**18,091**
	Km 51 to Km 71.3	% of Section:	71%	19%	-	-	**90%**
	Section N3	Length (m):	1,534	-	-	-	**1,534**
	Km -4.7 to Km 27	% of Section:	5%	-	-	-	**5%**
South	Section S1a	Length (m):	13,475	4,266	-	-	**17,741**
	Km 22 to Km 46	% of Section:	56%	18%	-	-	**74%**
	Section S1b	Length (m):	1,028	-	-	-	**1,028**
	Km 47.3 to Km 48.6	% of Section:	76%	-	-	-	**76%**
	Section S2a	Length (m):	1,259	-	-	-	**1,259**
	Km 46 to Km 47.3	% of Section:	100%	-	-	-	**100%**
	Section S2b	Length (m):	24,462	2,520	-	-	**26,982**
	Km 48.6 to Km 77.3	% of Section:	85%	9%	-	-	**94%**
	Section S3	Length (m):	50	-	-	-	**50**
	Km -4.4 to Km 22	% of Section:	0%	-	-	-	**0%**

Table 10.7
Freeboard Deficiency for Existing Dike Sections
at 35,000 cfs Design Discharge

Dike	Dike Section	Quantifier	Freeboard Deficiency Category (m)				Cum. Deficient
			0 - 0.5	0.5 - 1	1 - 1.5	1.5 - 2	
North	Section N1a	Length (m):	-	75	15,960	-	**16,035**
	Km 27 to Km 43	% of Section:	-	0%	100%	-	**100%**
	Section N1b	Length (m):	-	-	1,500	-	**1,500**
	Km 49.5 to Km 51	% of Section:	-	-	100%	-	**100%**
	Section N2a	Length (m):	-	-	6,467	-	**6,467**
	Km 43 to Km 49.5	% of Section:	-	-	100%	-	**100%**
	Section N2b	Length (m):	-	1,667	11,762	6,836	**20,265**
	Km 51 to Km 71.3	% of Section:	-	8%	58%	34%	**100%**
	Section N3	Length (m):	8,318	9,674	1,599	-	**19,591**
	Km -4.7 to Km 27	% of Section:	26%	31%	5%	-	**62%**
South	Section S1a	Length (m):	3	4,699	14,292	4,943	**23,937**
	Km 22 to Km 46	% of Section:	0%	20%	59%	21%	**100%**
	Section S1b	Length (m):	-	-	-	1,344	**1,344**
	Km 47.3 to Km 48.6	% of Section:	-	-	-	100%	**100%**
	Section S2a	Length (m):	-	-	514	745	**1,259**
	Km 46 to Km 47.3	% of Section:	-	-	41%	59%	**100%**
	Section S2b	Length (m):	239	1,336	17,038	9,611	**28,224**
	Km 48.6 to Km 77.3	% of Section:	1%	5%	60%	34%	**100%**
	Section S3	Length (m):	9,064	5,179	-	-	**14,243**
	Km -4.4 to Km 22	% of Section:	34%	20%	-	-	**54%**

10.1.5 Assessment of Probability of Failure vs. Design Discharge

Based on the freeboard deficiency discussed in Section 10.1.4.3, experience from the 2011 Flood and engineering judgement, Table 10.8 summarizes the estimated probability of failure for each dike section for the different design flows under consideration. Results from this assessment were incorporated into the damage model of the Lower Assiniboine River, as documented in Section 8.6.1.4.

Table 10.8
Probability of Failure of Assiniboine River Dike Sections at Various Flows

Dike	Dike Section	Probability of Failure			
		18,000 cfs	20,000 cfs	25,000 cfs	35,000 cfs
North	Section N1a Km 27 to Km 43	20%	60%	100%	100%
	Section N1b Km 49.5 to Km 51	80%	100%	100%	100%
	Section N2a Km 43 to Km 49.5	Very Low or 0%	Very Low or 0%	80%	100%
	Section N2b Km 51 to Km 71.3	Very Low or 0%	40%	100%	100%
	Section N3 Km -4.7 to Km 27	Very Low or 0%	Very Low or 0%	20%	100%
South	Section S1a Km 22 to Km 46	20%	60%	100%	100%
	Section S1b Km 47.3 to Km 48.6	80%	100%	100%	100%
	Section S2a Km 46 to Km 47.3	Very Low or 0%	Very Low or 0%	80%	100%
	Section S2b Km 48.6 to Km 77.3	Very Low or 0%	40%	100%	100%
	Section S3 Km -4.4 to Km 22	Very Low or 0%	Very Low or 0%	20%	100%

Though the dikes can hydraulically accommodate a flow of 510 m³/s (18,000 cfs) without being overtopped, the existing diking system is not fully reliable geotechnically due to the risk of a dike breach due to piping, dike slope failure, etc. Generally, it is assumed there is a very low or "0% probability" of failure for the existing diking system for flows up to 510 m³/s (18,000 cfs) with the exception of the previously noted critical sites along the river where significant dike rehabilitation is required (Sections N1b and S1B). During the 2011 Flood, dike sections N1a and S1a experienced downstream toe piping and slope instabilities at sustained flows of 510 m³/s (18,000 cfs), which necessitated regular dike inspections and emergency repairs. Similarly, dike

Sections N1b and S1b had secondary dikes constructed during the 2011 Flood, which have since been removed. The successful performance of Sections N1b and S1b at 510 m³/s (18,000 cfs), without the secondary flood control measures, cannot be guaranteed. The current reliable flow capacity of the dikes has, therefore, been estimated at 425 m^3/s (15,000 cfs).

As the design flow increases, the probability of failure increases for all the dike sections. Due to the risk of overtopping, the probability of failure increases to almost 100% for the dike sections downstream of Km 27 (North Dike) and Km 22 (South Dike) at a design flow of 708 m³/s (25,000 cfs), and reaches 100% for the entire existing diking system at a flow of 991 m³/s (35,000 cfs).

The probability of failure for each flow condition depends on the final height of the dike. A probability of failure of 100% generally corresponds to a breach of the dike due to overtopping because the dike is too low. However, as experienced in 2011 and discussed above, the existing dikes are not reliable in some areas despite having adequate freeboard. In those areas, breaching of the dikes could occur due to piping and/or dike slope failures. For this reason, upgrades to the existing dikes are required to pass flows of 510 m³/s (18,000 cfs) reliably.

10.1.6 Current Condition Downstream of the Existing Dikes

For the river reach that extends from the end of the existing diking system to the City of Winnipeg limits in Headingley (N4 and S4 on Drawing 10.1), the results of the hydraulic analyses (2015 calibration) showed that the water level exceeds the existing river bank at flows of 510 m³/s (18,000 cfs). Although a limited amount of damages occurred along this reach during the floods of 2011 and 2014, temporary flood mitigation was required to protect vulnerable properties against flows at the time. In 2014, temporary measures were deployed at 108 properties located in the Rural Municipalities of St. Francois Xavier and Cartier, to protect against projected water levels one foot above the 2011 peak of 510 m³/s (18,000 cfs). This was equivalent to an approximately 566 m³/s (20,000 cfs) water surface profile on the Assiniboine River.

On this basis, extending the existing diking system all the way to the City of Winnipeg limits in Headingley by constructing continuous dikes on either side of the river was considered for the

current study, and is further discussed in Section 10.1.7 of this report. The height of the dikes required along this reach (N4 and S4) was determined for the design flows of 510 m³/s, 566 m³/s, 708 m³/s, and 991 m³/s (18,000 cfs, 20,000 cfs, 25,000 cfs, and 35,000 cfs), based on the original "downstream" model of the Assiniboine River discussed in Section 8.3.2. Updating the height of the dikes based on the 2015 calibration of the model was not necessary. This is discussed in Section 10.1.7.1.

A freeboard of 0.6 m (2 ft) was added to each water surface profile to determine the minimum height requirement of the dikes, assuming that the highest point on the north and south bank of the river would form the base elevation for the dikes. Since the highest points on the banks were generally located in close proximity to the river, it was judged reasonable to assume at this conceptual phase that there would be sufficient room to construct the dikes between homes or other infrastructure and the river. Nonetheless, the exact location of homes and other infrastructure along this reach of river would have to be considered in more detail at the next stage of study should this option be selected to proceed.

The height of dikes required along this reach was divided into five categories. The lengths of the proposed dikes falling into each height category, and the percentage of these lengths compared to the total reach length, are summarized in Table 10.9.

Table 10.9
Height of Proposed Dikes Extended to Headingley

Dike	Design Discharge (cfs)	Quantifier	Height Category (m)					Cum. Length
			0 - 1	1 - 2	2 - 3	3 - 4	4 - 5	
North (N4)	18,000	Length (m):	18,779	8,109	-	-	-	**26,888**
		% of Section:	41%	17%	-	-	-	**58%**
	20,000	Length (m):	17,258	13,109	1,758	-	-	**32,125**
		% of Section:	38%	28%	4%	-	-	**70%**
	25,000	Length (m):	14,064	18,613	7,042	1,127	-	**40,846**
		% of Section:	31%	40%	15%	2%	-	**88%**
	35,000	Length (m):	1,289	12,439	17,380	9,127	1,399	**41,633**
		% of Section:	3%	26%	38%	20%	3%	**90%**

Dike	Design Discharge (cfs)	Quantifier	Height Category (m)					Cum. Length
			0 - 1	1 - 2	2 - 3	3 - 4	4 - 5	
South (S4)	18,000	Length (m):	20,753	8,537	1,074	-	-	**30,364**
		% of Section:	51%	21%	3%	-	-	**75%**
	20,000	Length (m):	17,762	13,708	1,719	489	-	**33,677**
		% of Section:	44%	34%	4%	1%	-	**83%**
	25,000	Length (m):	5,954	20,634	8,656	1,074	-	**36,318**
		% of Section:	15%	51%	21%	3%	-	**90%**
	35,000	Length (m):	3,148	5,314	19,924	10,418	1,074	**39,878**
		% of Section:	8%	13%	49%	26%	3%	**99%**

The analysis of the dikes for Sections N4 and S4 was based on the river stationing along this reach. However, it was determined that the total lengths of the dikes would likely be shorter in some areas due to the alignment cutting through meander sections of the river. Based on a review of possible dike alignments along this reach, the lengths provided on Table 10.9 were reduced by 10% when compared to a dike alignment that would be parallel to the river. The reduced lengths were then used for the cost estimates for Sections N4 and S4

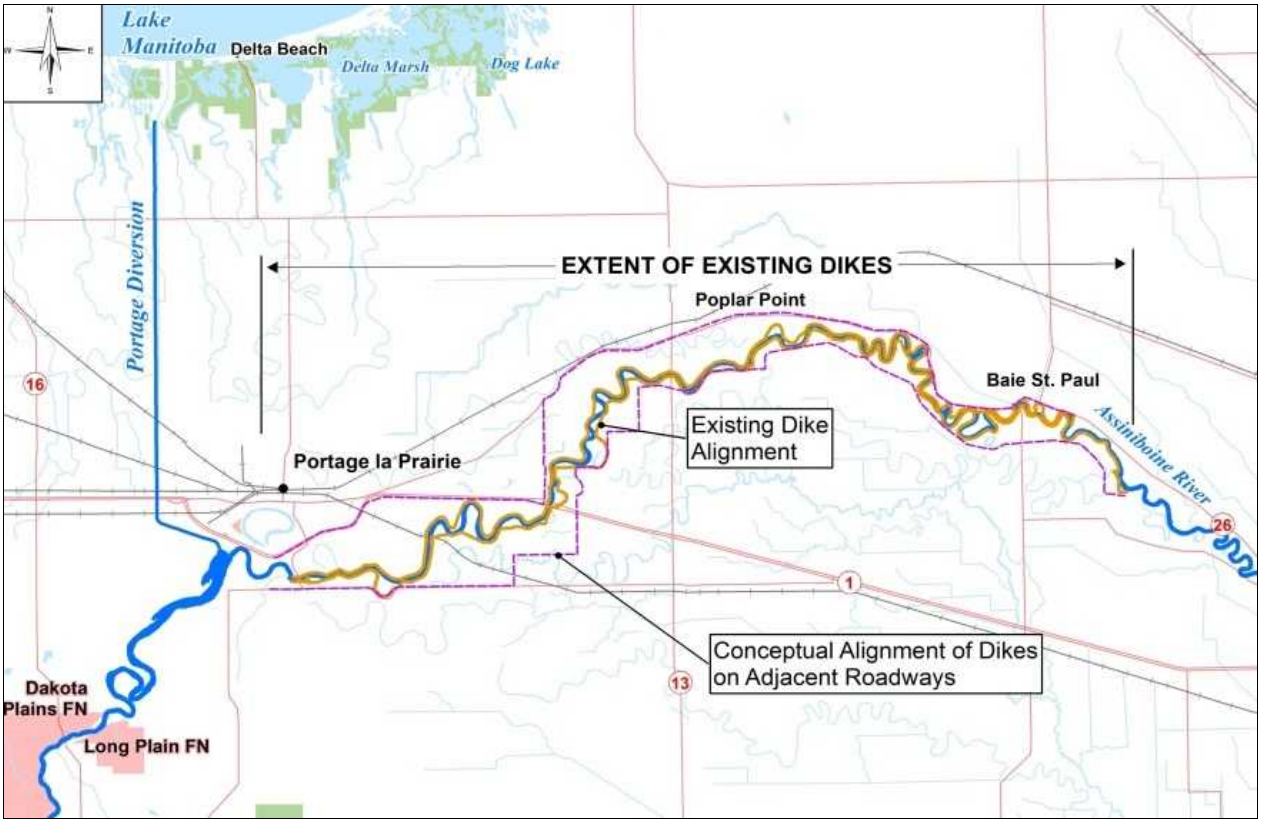
10.1.7 Conceptual Design Alternatives and Cost Estimates

Conceptual designs and cost estimates were prepared for three improvement schemes for the existing Provincial diking system, and two improvement schemes were considered downstream of the existing dikes. The improvement schemes between Portage la Prairie and Baie St. Paul consisted of:

Option 1a – Upgrading the existing dikes in their current location.
Option 1b – A major setback of the dikes to adjacent roadways.
Option 1c – A combination of existing dike upgrades and major setbacks to adjacent roadways.

Figure 10.5 shows the existing dike alignment and the conceptual alignment of dikes setback to adjacent roadways. As discussed in Sections 10.1.3.1 and 10.1.4.3, detailed maps of the existing diking system are provided in Appendix G.

Figure 10.5
Alignment of Assiniboine River Dikes



The improvement schemes between Baie St. Paul and the City of Winnipeg limits in Headingley consisted of:

Option 2a – Extend the existing dikes along the river to the City of Winnipeg limits in Headingley.
Option 2b – Combination of individual flood proofing, neighbourhood diking and the purchase of vulnerable properties severely affected by flooding.

Options 1a, 1b, and 1c, are independent of options 2a and 2b since they are improvement schemes for two separate reaches of the Lower Assiniboine River. Therefore any combination of improvements for the existing dikes (options 1a, 1b, and 1c) could proceed with any combination of improvements for the reach downstream of Baie St. Paul (options 2a and 2b), including the option of improving one reach but not the other.

Depending on the improvement scheme, the following items were included in the cost estimate:

- All aspects of dike construction or dike upgrading including ROW acquisition.
- Floodplain land acquisition and individual ring dikes.
- Control structure with pumping requirements.
- Bridge replacements.
- Neighbourhood diking and individual flood proofing.
- Purchase of vulnerable properties.

Although there are utilities present in the vicinity of the proposed dike schemes, such as along PR 430 and PR 248, reconstruction or relocation costs of utilities are anticipated to be relatively low and, therefore, considered as part of the contingency cost allowance. Nonetheless, utility services and connections would need to be identified during the next phase of study to locate any dike sections that may need design consideration for utilities.

The details and results of the analysis, including assumptions, quantities and costs for each of the items listed above are summarized individually in the sections that follow. The costs of each individual item are then combined at the end and are summarized in Section 10.1.7.7.

10.1.7.1 Dike Construction

It was assumed that the construction of the dikes would meet the following design guidelines, as outlined in the KGS report on the 2011 Flood (KGS Group, 2013a):

- Minimum long-term stability Factor of Safety of 1.5.
- A standard minimum 0.6 m (2 ft) freeboard above design water level applied everywhere.
- Riprap erosion protection at critical areas.
- Acquisition of dike right-of-way to ensure maintenance access.
- Minimum 6 m (19.7 ft) top width for construction equipment.
- Gravel topping on the dikes for all season maintenance and emergency response access.
- All season road access at minimum 3 km (1.9 mi) spacing.

To meet these guidelines, eight quantity items, common to options 1a, 1b, 1c and 2a were used to develop the cost estimate for each upgrade alternative. These items included:

1. ***Land Acquisition*** – The land acquisition area was based on a dike footprint large enough to pass each design flow plus an additional 30 m (98.4 ft) right-of-way provision for continuous toe drainage and ditching. Parts of the Option 1b alignment consisted of upgrades to existing roads to serve as a new dike. No additional provision for internal drainage was assumed for sections of Option 1b that consisted of a road upgrade. Option 2a requires entirely new dikes to the City of Winnipeg so an additional 25 m

(82 ft) right-of-way provision was included for the dike footprint such that the final dike alignment and its footprint can be offset at least 25 m (82 ft) from the edge of the river. The assumed purchase price of land acquisition was $15,000 per hectare. Crown Land and Property Agency (CLPA) costs and other fees associated with land acquisition were assumed to be included in the allowance for indirect costs discussed in Section 10.1.7.7.

In some areas, there are areas of the existing diking system that are already owned by the Province, as shown on the detailed maps provided in Appendix G. To remain conservative in the cost estimates and to account for the potential need to purchase isolated land parcels or bisected parcels rather than just "strips", areas already owned by the Province were not subtracted from the totals.

It should be noted that the cost to purchase all of the property between the existing dikes and a major setback upgrade alternative (Option 1b or parts of Option 1c) was calculated separately in addition to this land acquisition estimate as discussed in Section 10.1.7.2.

2. ***Clearing and Grubbing*** – The clearing and grubbing area was considered for a dike footprint large enough to pass each design flow plus half the additional 30 m (98.4 ft) right-of-way provision for internal drainage. The assumed clearing and grubbing unit cost was $16,500 per hectare.

3. ***Supply and Placement of Dike Fill*** – The calculated volume was separated into two quantities: re-shaping in-situ fill at $5 per cubic metre and importing new fill at $20 per cubic metre. It should be noted that the unit rate for re-shaping in-situ fill could be slightly higher. MIT construction staff suggested a unit rate of about $8 per cubic metre. This higher unit rate was not incorporated into the cost estimates at this conceptual phase of design. However, potential opportunities of optimization and the potential variability of the $5 per cubic metre unit rate should be considered at the next phase of study. Re-shaping in-situ fill was unique to Option 1a. The existing dikes were raised under winter emergency conditions in 2011 and buttressed further during the flood. As such, there would be unsuitable material and stumps, etc., within the dike prism. The construction approach would be to tear down the dikes, dispose of the unsuitable material and rebuild the dikes in an engineered fashion to current standards. This would maximize reuse of competent material and ensure the dike prism would meet current design guidelines.

   The new dike will be much wider than the old dikes spanning over the 2011 borrow areas. The volume calculations described below, therefore, include the necessary volume to "fill in" the 2011 borrow.

   The volume for reshaping and suitably compacting the existing fill was estimated from the available LiDAR data. An average existing side slope was assumed for the entirety of each section of the existing dikes together with a 4 m (13.1 ft) average crest width. The existing height of the dikes was simplified to be either 2 m or 3 m (6.6 ft or 9.8 ft) based on the respective proportions of the dikes (by length) that fell into the 0 m to 2 m category and the 2 m (6.6 ft) and above category, as previously summarized in Section 10.1.4.1 (Table 10.3). The overall volume included a +25% correction factor to account for compaction and handling losses and rejection of unsuitable fill.

Importing new dike fill for Option 1a was calculated by first determining the overall height deficiencies by length of the existing dikes for each design flow, as previously summarized in Section 10.1.4.3 (Table 10.5 to 10.7). These proportions were used to create design dike geometries with new heights based on the existing 2 m or 3 m (6.6 ft or 9.8 ft) high dikes plus the required average freeboard top-up, a crest width of 6 m (19.7 ft), and design slopes of 4H:1V. The overall volume included a +25% correction factor to account for compaction and handling losses. As discussed in Section 8.3.2, updated water surface profiles became available with the post-2011 Flood model. Therefore, a correction factor was applied to the initially assumed fill quantities to account for the change in water level. This correction factor was calculated as the ratio of the overall deficient length of dike for a particular section considering the new water surface profile to the overall deficient length of dike for a particular section considering the old water surface profile. For example, for the 20,000 cfs flow condition, 5.7% more of the north dike from Portage la Prairie to the Baie St. Paul Bridge was freeboard deficient when considering the new water surface profile. The additional fill quantity for this section was, therefore, increased by 1.057.

Imported dike fill for Option 1b was calculated by average end areas at intervals of 50 m±. An existing road profile was generated along the most likely road alignments for the setback dikes. As shown on Figure 10.5, the setback north dike alignment consists of the TransCanada Highway from Portage la Prairie to its crossing at the Assiniboine River, Rural Road 28W between the TransCanada Highway and PTH 26, and PTH 26 from Rural Road 28W to the Baie St. Paul Bridge. The setback south dike alignment consists of PR 331 and miscellaneous rural roads west of the TransCanada Highway and River Road and miscellaneous rural roads east of the TransCanada Highway. New dikes offset from the main roadways were assumed for paved road sections of the setback alignment. For gravel topped road sections, it was assumed that the roads would be raised to the design discharge water surface profile level with 0.6 m (2 ft) freeboard and assigned the same existing road width with 6H:1V side slopes. At this conceptual level of analysis, the additional cost of raising the approach of intersecting roadways or access roads was not included in the estimate but could be required if this option was deemed feasible. Water surface profiles for Option 1b were solely based on the post-2011 Flood model and, therefore, applying an additional correction factor to the dike fill quantities for this option was not necessary.

Importing new dike fill for Option 2a was calculated following a similar methodology as Option 1a, based on the proposed dike heights summarized in Section 10.1.6 (Table 10.9). It should be noted that the analysis was completed using a water surface profile based on the original pre-2011 “downstream” model of the Assiniboine River. As discussed in Section 8.3.2, the model was re-calibrated in the winter of 2015, based on new data that was made available from the 2011 and 2014 floods, which showed that the original model was underestimating the water levels by about 0.3 m to 0.5 m (1.0 ft to 1.6 ft) in some locations. This suggested that the calculated volumes for Option 2a were underestimated. However, since the final cost estimate for Option 2a (based on the older model) was well above the cost of Option 2b, which was the alternate improvement scheme along this reach of the river (the total costs are summarized in Section 10.1.7.7), it was determined that updating the quantities for 2a was not necessary, as it would not change the conclusion of the study that Option 2b was preferred over Option 2a based on cost.

4. ***Supply and Placement of Granular Toe Drain Material*** – A provision for a granular toe drain was included in the design dike geometry everywhere along the dikes to control seepage and piping potential. Costing included drainage swales to convey the water to the local drainage system such as at the closest mile road ditch or provincial drain. A separate allowance was included for drainage structures and pumping requirements where required and is discussed in Section 10.1.7.3.

   The quantity for granular toe drain material was calculated assuming a triangular cross-sectional area extending from the toe of the dike with a height equal to one-third of the required dike height and a base width equal to the required dike height. An additional 2 m wide by 2 m deep (6.6 ft wide by 6.6 ft deep) toe drain granular quantity was assumed to remediate critical sites identified during the 2011 Flood. No reductions in the clay dike fill quantities were assumed for the toe drain quantity. Sections of Option 1b that consisted of a road upgrade were not assumed to require a granular toe drain due to the shallower side slope of 6H:1V. The assumed cost was $60 per cubic metre of granular toe drain.

5. ***Supply and Placement of Granular Road Topping*** – Granular road topping was included to ensure construction and monitoring equipment could access all areas of the upgraded dikes in an emergency situation. The minimum road topping thickness was assumed to be 0.3 m (1 ft) on new dikes and 0.6 m (2 ft) on upgraded roads for the full width of the upgraded dike or road section. No reductions in the clay dike fill quantities were assumed for the granular road topping. The assumed cost was $45 per cubic metre of gravel topping.

6. ***Revegetation*** – The revegetation area was based on a dike footprint large enough to pass each design flow plus half the additional 30 m (98.4 ft) right-of-way provision for internal drainage. The assumed revegetation cost was $5,500 per hectare for a standard cover crop. An allowance for a reforestation program along the Assiniboine River was not included at this conceptual level of analysis but could be considered at the next stage of design.

7. ***Supply and Placement of Riprap*** – Riprap was included to ensure a 50-year design life for dike upgrades along the original dike alignment. The riprap quantity was based on the total length of dike located within 50 m (164 ft) of an outside bend of the river that did not currently have erosion protection. Riprap was not required above the edge of the normal river channel because the upgraded dike itself would be revegetated. The assumed riprap cost was $90 per cubic metre.

8. ***Access Road Construction*** – A provision for access roads was required for dikes upgraded along the original dike alignment (Option 1a and parts of 1c). These roads would be required to fully access all areas of the dikes with heavy construction equipment in an emergency situation. The cumulative length of access road was calculated as the distance between existing gravel-topped roads and the crest of the dike at minimum 3 km (1.9 mi) intervals along the dikes. Option 1b was assumed not to require additional access roads, as the alignment itself was either a roadway or immediately adjacent to a major roadway. The assumed access road construction cost was $750,000 per kilometre and included the cost of land acquisition for the road.

Summarized below are the estimated quantities and cost of dike construction for Options 1a, 1b, 1c and 2a.

Option 1a – Upgrade Existing Dikes

Table 10.10 summarizes the estimated quantities and costs to upgrade the existing dikes from Portage la Prairie to approximately Baie St. Paul in their current position. The current cost estimate includes erosion protection and seepage control everywhere along the dikes including Willy's Farm and the Super Sandbag site.

At this conceptual stage, no special provisions were made to optimize the dike alignment. Should Option 1a be selected to proceed, opportunities and locations of potential minor setbacks for the dikes should be identified during the next stage of study. In general, all inside bends are candidates for potential dike setbacks. Virtually all are highly susceptible to seepage, being founded on alluvial deposits. Typically there is no development within the inside bends because they have been particularly susceptible historically to flooding. Therefore, a setback at the inside bends does not represent a significant change to land use. However, it does significantly reduce risks, Willy's Farm and the Super Sandbag site being classic examples in that regard. On the other hand, the study has assumed that, on outside bends, sufficient erosion protection would be constructed to secure the dikes.

Table 10.10
Estimated Cost of Dike Construction – Option 1a

Design Flow			18,000 cfs		20,000 cfs		25,000 cfs		35,000 cfs	
Work Item	**Units**	**Unit Price**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**
Land Acquisition	ha	$15,000	304	$4.6	500	$7.5	836	$12.5	991	$14.9
Clearing and Grubbing	ha	$16,500	220	$3.6	325	$5.4	596	$9.8	756	$12.5
Reshaped Clay Fill	m^3	$5	1,454,125	$7.3	2,810,125	$14.1	4,646,125	$23.2	4,646,125	$23.2
Imported Clay Fill	m^3	$20	572,125	$11.4	1,133,250	$22.7	3,230,875	$64.6	9,705,875	$194.1
Granular Toe Drain Rock	m^3	$60	115,500	$6.9	162,800	$9.8	272,800	$16.4	338,500	$20.3
Gravel Topping	m^3	$45	267,600	$12.0	267,600	$12.0	267,600	$12.0	267,600	$12.0
Revegetation	ha	$5,500	220	$1.2	325	$1.8	596	$3.3	756	$4.2
Riprap	m^3	$90	200,000	$18.0	200,000	$18.0	200,000	$18.0	200,000	$18.0
Access Roads	km	$750,000	32.05	$24.0	32.05	$24.0	32.05	$24.0	32.05	$24.0
Subtotal				**$89.1**		**$115.3**		**$183.8**		**$323.2**

Option 1b – Major Setback of Dikes to Adjacent Roadways

Table 10.11 summarizes the estimated quantities and costs to build new dikes setback to adjacent roadways, as was shown on Figure 10.5.

This upgrade alternative results in many properties and structures now being located on the flooded side of the new dike alignment. Therefore, it was assumed that additional land acquisition and the construction of ring dikes would also be necessary. This is further addressed in Section 10.1.7.2.

The main benefit of this alternative was having the dikes set back from the river meander pattern, which provides greater security during flood events; and the major roadways would allow access to all areas of the new dike alignment. The drawbacks of this alternative were the high land purchase and ring diking costs discussed in Section 10.1.7.2. Dike construction costs were also higher than Option 1a mainly because the Assiniboine River channel is perched, requiring greater quantities of imported clay fill the further away in location from the river to construct the setback dikes than to upgrade the existing dikes.

Table 10.11
Estimated Cost of Dike Construction – Option 1b

Design Flow			18,000 cfs		20,000 cfs		25,000 cfs		35,000 cfs	
Work Item	**Units**	**Unit Price**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**
Land Acquisition	ha	$15,000	335.3	$5.0	379.0	$5.7	466.8	$7.0	649.1	$9.7
Clearing and Grubbing	ha	$16,500	281.8	$4.7	325.0	$5.4	406.0	$6.7	576.8	$9.5
Reshaped Clay Fill	m^3	$5	0	$0.0	0	$0.0	0	$0.0	0	$0.0
Imported Clay Fill	m^3	$20	5,421,000	$108.4	6,619,500	$132.4	9,728,400	$194.6	17,452,800	$349.1
Granular Toe Drain Rock	m^3	$60	73,300	$4.4	93,100	$5.6	145,100	$8.7	289,500	$17.4
Gravel Topping	m^3	$45	270,000	$12.2	290,500	$13.1	304,700	$13.7	322,600	$14.5
Revegetation	ha	$5,500	281.8	$1.6	325.0	$1.8	406.0	$2.2	576.8	$3.2
Riprap	m^3	$90	44,000	$4.0	9,200	$0.8	9,200	$0.8	9,200	$0.8
Access Roads	km	$750,000	0	$0.0	0	$0.0	0	$0.0	0	$0.0
Subtotal				**$140.2**		**$164.7**		**$233.8**		**$404.2**

Option 1c – Combination of Existing Dike Upgrades and Setbacks

Table 10.12 summarizes the quantities and costs to upgrade the existing dikes (Option 1a) from Portage la Prairie to the TransCanada Crossing and to build new dikes along roadways adjacent to the existing dikes (Option 1b) between the TransCanada Crossing and approximately Baie St. Paul. The TransCanada crossing was determined to be the most suitable transition point, as it provided an opportunity to reduce the high land acquisition costs of the numerous properties nearer to Portage la Prairie, while reducing risk by relocating the Priority 1 dikes, which are east of the TransCanada Crossing.

The benefit of this alternative was that it maintained the added security from relocating the dikes away from critical areas, while not carrying the extra cost of land purchase and ring dikes in areas where it may not have been warranted. The disadvantage of this alternative was that, overall, it remained slightly more costly than upgrading all of the existing dikes in their current location.

Table 10.12
Estimated Cost of Dike Construction – Option 1c

Design Flow			18,000 cfs		20,000 cfs		25,000 cfs		35,000 cfs	
Work Item	**Units**	**Unit Price**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**
Land Acquisition	ha	$15,000	233.4	$3.5	267.4	$4.0	561.9	$8.4	734.4	$11.0
Clearing and Grubbing	ha	$16,500	216.3	$3.6	252.5	$4.2	427.5	$7.1	572.8	$9.5
Reshaped Clay Fill	m^3	$5	0	$0.0	0	$0.0	1,572,375	$7.9	1,572,375	$7.9
Imported Clay Fill	m^3	$20	4,065,500	$81.3	5,014,250	$100.3	7,778,500	$155.6	14,757,625	$295.2
Granular Toe Drain Rock	m^3	$60	48,400	$2.9	62,400	$3.7	147,200	$8.8	242,400	$14.5
Gravel Topping	m^3	$45	318,100	$14.3	333,300	$15.0	339,400	$15.3	344,000	$15.5
Revegetation	ha	$5,500	216.3	$1.2	253.0	$1.4	427.5	$2.4	572.8	$3.2
Riprap	m^3	$90	48,500	$4.4	48,500	$4.4	48,500	$4.4	48,500	$4.4
Access Roads	km	$750,000	12.6	$9.5	12.6	$9.5	12.6	$9.5	12.6	$9.5
Subtotal				**$120.6**		**$142.4**		**$219.2**		**$370.5**

Option 2a – Extend the Dikes from Approximately Baie St. Paul to the City Of Winnipeg Limits in Headingley

Table 10.13 summarizes the estimated quantities and costs to extend the existing dikes from approximately Baie St. Paul to the City of Winnipeg limits in Headingley. The methods of estimating the dike quantities and costs were as discussed previously in Section 10.1.7.1. Details on dike heights and lengths were summarized in Table 10.9. As discussed in Section 10.1.6, the dike alignment would follow a path generally parallel to the river, with the exception of a few locations where the alignment would likely cut across river meanders.

The benefit of this alternative was that the dikes would provide flood protection to all stakeholders along the river, including agricultural producers; and local access would be maintained at times of flood. The disadvantages of this alternative were the large cost of extending the dike system to Headingley, as well as the added requirement for pumping, as discussed in Section 10.1.7.2.

Table 10.13
Estimated Cost of Dike Construction – Option 2a

Design Flow			18,000 cfs		20,000 cfs		25,000 cfs		35,000 cfs	
Work Item	**Units**	**Unit Price**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**
Land Acquisition	ha	$15,000	410.00	$6.2	473.00	$7.1	580.00	$8.7	683.00	$10.3
Clearing and Grubbing	ha	$16,500	240.00	$4.0	282.00	$4.7	355.00	$5.9	441.00	$7.3
Reshaped Clay Fill	m^3	$5	0	$0.0	0	$0.0	0	$0.0	0	$0.0
Imported Clay Fill	m^3	$20	623,375	$12.5	961,500	$19.2	1,880,250	$37.6	4,191,250	$83.8
Granular Toe Drain Rock	m^3	$60	10,200	$0.6	17,900	$1.1	40,400	$2.4	103,600	$6.2
Gravel Topping	m^3	$45	174,000	$7.8	174,000	$7.8	174,000	$7.8	174,000	$7.8
Revegetation	ha	$5,500	240.00	$1.3	282	$1.6	355.00	$2.0	441.00	$2.4
Riprap	m^3	$90	147,500	$13.3	147,500	$13.3	147,500	$13.3	147,500	$13.3
Access Roads	km	$750,000	14.39	$10.8	14.39	$10.8	14.39	$10.8	14.39	$10.8
Subtotal				**$56.4**		**$65.5**		**$88.4**		**$141.9**

10.1.7.2 Floodplain Land Acquisition and Individual Ring Dikes

For Options 1b and 1c, it was assumed that all of the property located between the existing dikes and the new dikes setback to adjacent roadways would have to be purchased, as it would be prone to flooding. As discussed in Section 10.1.7.1, for Option 2a, it was assumed that the dike could be constructed between existing structures and the River.

For Options 1b and 1c, it was then assumed that all existing buildings and structures (homes, farmyards, businesses) in this area would be protected with ring dikes. It could be that property owners resist the ring dike option and that buyouts or relocations would be required, but that option was not considered at this level of study. The assumed purchase price of land acquisition was $15,000 per hectare while the cost for ring dikes was assumed to be $1 million per lineal kilometre of dike. Properties protected by ring dikes would not have to be purchased (i.e., the purchase of residential homes, farmyards, businesses would not be required). The costs were assumed to be independent of the Assiniboine River capacity. Crown Land and Property Agency (CLPA) costs and other fees associated with land acquisition were assumed to be included in the allowance for indirect costs discussed in Section 10.1.7.7. Table 10.14 summarizes the estimated additional quantities and costs for floodplain land acquisition and individual ring dikes associated with options 1b and 1c. These costs are in addition to those identified in Tables 10.11 and 10.12.

Table 10.14
Estimated Cost of Floodplain Land Acquisition and Individual Ring Dikes

Work Item	Option 1b		Option 1c	
	Quantity	Cost (Million)	Quantity	Cost (Million)
Individual Ring Dikes	125 km	$125.0	60 km	$60.0
Floodplain Land Acquisition	11,160 ha	$167.4	6,230 ha	$93.40
Estimated Total Cost		$292.4		$153.4

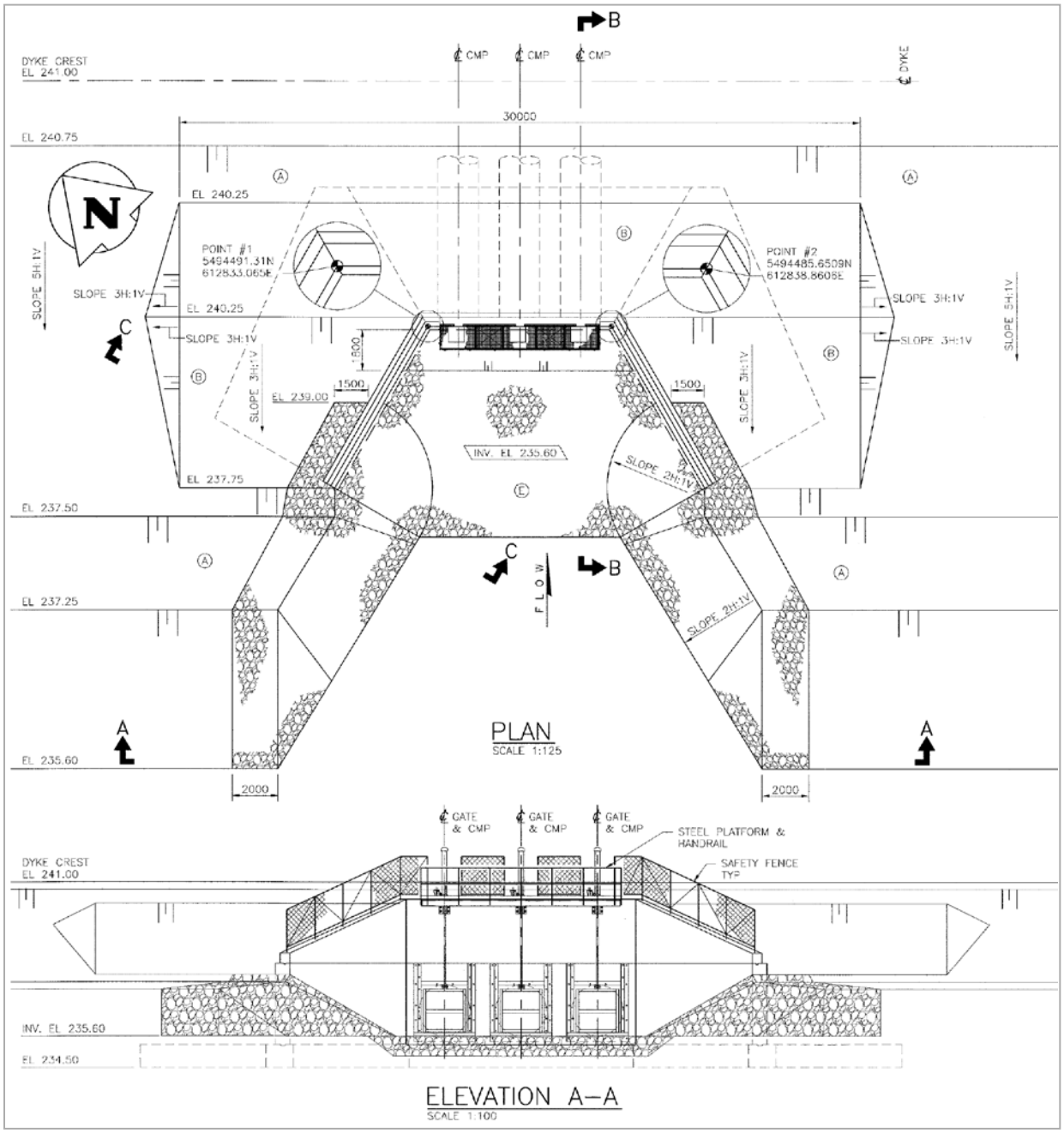
10.1.7.3 Drainage Structures and Pumping Requirements

Multiple streams and provincial drains discharge into the Assiniboine River between Portage la Prairie and Headingley, particularly downstream of Baie St. Paul. During flood conditions, high

water levels from the Assiniboine River back up into the streams causing flooding of adjacent land. Currently, most of these streams can accommodate the backwater from the Assiniboine River, without causing significant damage. However, the proposed dike improvement schemes and corresponding higher water levels within the dikes will result in higher flood levels in the tributaries that drain into the Assiniboine River, and in significantly larger back flooded areas outside of the dikes.

The risk of flooding in the streams and drains due to backwater from the Assiniboine River was deemed unacceptable for the improvement schemes for dike upgrades (Options 1a, 1b, 1c and 2a). To mitigate this risk, gated drainage structures were incorporated into the conceptual design. It is anticipated that the gated drainage structures would be similar to the drainage structures that were constructed along the West Dike of the Red River Floodway. A drawing of a typical structure, a Bolen Drainage Structure on the West Dike is provided on Figure 10.6 and is considered representative of the type of structure that would be required on the Lower Assiniboine River.

Figure 10.6
Plan of Bolen Drainage Structure on the West Dike of the Red River Floodway



This type of drainage structure was designed as a reinforced concrete cantilever gravity structure consisting of a footing, a headwall and two sloping wing walls that allow for drainage of runoff through the West Dyke into the downstream portion of the Bolen Drain. Flow is conveyed with three 1,800 mm (70.9 in) diameter culverts and controlled by three cast iron sluice gates,

which can be operated either in a fully open or fully closed position. Control of the sluice gates is provided near the top of the structure.

On the Assiniboine River, temporary pumping would also be necessary to accommodate runoff during periods when the gates of the drainage structure were closed. Where designated provincial drains or other larger streams enter the Assiniboine River, it was assumed that permanent pump stations would be installed. The proposed concept would utilize pumping stations to pump the stream or drain flows into the Assiniboine River when the river level is high. A gate chamber with conduits installed through the dikes would allow the stream to discharge by gravity when the river level is low. The pump station would include a gate chamber to prevent backflow through the conduits when the river level exceeded the level in the tributaries.

Based on a cursory review of the existing drainage network, twelve common discharge points (see Drawing 10.1) were identified along the Lower Assiniboine River where permanent pumping stations would be constructed. The following four sites are located along the existing Assiniboine River dike alignment and would be considered for Options 1a, 1b and 1c:

1. Prestash Drain.
2. Long Lake Drain.
3. Marquette Road Drain.
4. Mill Creek.

The remaining eight sites are located downstream where a continuous diking system currently does not exist and would be considered for Option 2a only:

1. Barickman Coulee.
2. Hammercrest Creek.
3. Houston Road Drain.
4. No. 2 Drain.
5. Elkins Drain.
6. First Drain.
7. Second Drain.

8. Third Drain.

The design discharge capacity for pumping was based on the Province of Manitoba Regional Flood Formula using a 1:5 year design event. Hydrographs for various sizes of drainage areas were developed for the design event based on recorded data along the adjacent Grassmere Drain watershed. Past experience has shown that spring runoff governs over summer rainfall events in this area and that generally, approximately half of the runoff has already occurred before Assiniboine River levels reach flood stage at the drain outlets. It was, therefore, assumed that pumping could occur over a period of three weeks, and that only half of the total hydrograph volume would require pumping. The head requirement was based on the average height of dikes required for the improvement scheme plus an assumed 2 m (6.6 ft) depth for the drain or stream invert.

The estimated cost of pumping was based on a minimum fixed cost of $1,200,000, plus a unit rate of $120,000 per cubic metre per metre of head, which was determined based on a review of the actual construction costs of recent pumping stations constructed in Winnipeg and other locations in Manitoba. The estimate assumes that a 3-phase power supply is available in close proximity to the pumping sites, however, this would have to be verified at the next stage of study should the pumping concept at any of the 12 sites be selected. The estimated cost would be more if a reliable 3-phase power supply was not available in the area. Alternatively, a stand-alone temporary or permanent diesel-driven power station could be considered at the pumping sites, but the merits of this option would have to be based on both cost and performance. A summary of the design capacity and cost for pumping is shown in Table 10.15.

Table 10.15
Estimated Cost of Flood Pump Stations

Improvement Scheme	Name of Stream	Design Discharge (m^3/s)	18,000 cfs	20,000 cfs	25,000 cfs	35,000 cfs
			Cost (Million)	Cost (Million)	Cost (Million)	Cost (Million)
Option 1a, 1b, & 1c	Prestash Drain	1	1.5	1.6	1.7	1.8
	Long Lake Drain	9	3.9	4.4	5.5	6.6
	Marquette Road Drain	1	1.5	1.6	1.7	1.8
	Mill Creek	2	1.8	1.9	2.2	2.4
Sub Total			8.7	9.5	11.0	12.6
Option 2a	Barickman Coulee	1	1.5	1.6	1.7	1.8
	Hammercrest Creek	1	1.5	1.6	1.7	1.8
	Houston Road Drain	2	1.8	1.9	2.2	2.4
	No. 2 Drain	2	1.8	1.9	2.2	2.4
	Elkins Drain	2	1.8	1.9	2.2	2.4
	First Drain	2	1.8	1.9	2.2	2.4
	second Drain	2	1.8	1.9	2.2	2.4
	Third Drain	2	1.8	1.9	2.2	2.4
Sub Total			13.8	14.6	16.3	18.0

10.1.7.4 Bridge Replacements

The need for bridge replacements was considered for Options 1a, 1b, 1c and 2a. For this study, it was assumed that bridges would require replacement once the design water level submerged the bridge deck by one-third of its depth. For example, the bottom and top elevations of the PR 248 Bridge deck were estimated to be El 244.0 m and El 245.05 m (El 800.5 ft and El 804 ft), respectively, which corresponded to a bridge deck depth of 1.05 m (3.4 ft). It was assumed that this bridge would require replacement once the design water level reached El 244.35 m (El 801.7 ft) at the bridge, which is 0.35 m (one-third of the depth) above the assumed bottom elevation of 244.0 (El 800.5 ft). Bridge deck elevations were taken from either the data within the HEC-RAS model or were estimated based on surveyed water levels and photos taken during the 2011 and 2014 floods. Future studies would be required to confirm the

actual bridge deck elevations with surveys and/or design drawings and to confirm that a submergence of one-third of the bridge deck is acceptable.

The cost of bridge replacements was calculated based on a unit rate of $5,200/m^2 ($55,972/ft^2) of deck area plus $3 million per structure to account for approaches and traffic control. The deck area was estimated based on the existing footprint of the bridge.

Table 10.16 summarizes the water level at which the bridges need to be replaced along the Lower Assiniboine River based on the criteria noted above. As shown on the table, only two bridges would require replacement at discharges of less than 991 m³/s (35,000 cfs). The cost estimate to replace the two bridges is shown on the table.

Table 10.16
Summary of Assumed Bridge Levels and Estimated Replacement Cost

Bridge Location	Assumed Deck Elevation		Replacement Level (1/3 of Bridge Deck)	Discharge for Replacement	Replacement Cost (Million)
	High	Low			
	m	m	m	cfs	
PR 240	262.84	260.63	261.37	> 35,000	n/a
CN Rail	260.07	258.75	259.19	> 35,000	n/a
PTH 1 E	257.58	255.86	256.43	> 35,000	n/a
PTH 1 W	257.04	255.26	255.85	> 35,000	n/a
PR 430	253.97	251.88	252.58	34,000	$10.6
PR 248	245.05	244.00	244.35	21,000	$10.7
PTH 1 St FX E	243.70	241.98	242.55	> 35,000	n/a
PTH 1 St FX W	243.70	241.98	242.55	> 35,000	n/a

The Baie St. Paul Bridge (PR 248) is the first bridge that would require replacement. Figure 10.7 is a photo taken during the flood of 2014 showing the water level in the Assiniboine River just below the bottom of the PR 248 Bridge deck.

Figure 10.7
Baie St. Paul Bridge (PR 248) during the Flood of 2014

It is recognized that the bridges needing replacement are already part way through their existing life span. This has been considered in the economic analysis of the alternatives.

10.1.7.5 Combination of Neighbourhood Diking, Individual Flood Proofing, and Purchase of Vulnerable Properties

Option 2b is a scheme, which considers a combination of flood protection measures downstream of the current diking system to Headingley. In this area, a combination of flood protection measures were considered due to the variability in ground elevation, density and the location of vulnerable properties. The Option 2b measures considered included:

- Neighbourhood Diking.
- Individual Flood Proofing.
- Purchase of Vulnerable Properties.

Neighbourhood diking consists of constructing a ring diking system that protects numerous households and other vulnerable structures or a neighbourhood. Examples of neighbourhood dikes are those along Kingston Crescent in Winnipeg, or the dikes in the towns of Melita and

Souris on the Souris River. Along the lower Assiniboine River, Hutterite colonies or neighbourhoods similar to Lido Plage would be example candidates for neighbourhood diking.

Individual flood proofing can be achieved by raising the property foundation or constructing ring dikes similar to what has been done in the Red River Valley. For this study, it was assumed that the construction of ring dikes would be representative of the overall cost of the different individual flood proofing alternatives. The selection of a ring dike or a raised foundation could be optimized at the next stage of study.

In some locations, neither permanent neighbourhood diking or individual flood proofing are feasible solutions due to the very close proximity of the structures to the Assiniboine River and the high depth of flooding. In those locations, a purchase of the property was considered.

Contrary to the continuous dike system proposed for Option 2a, Option 2b would not mitigate potential impacts to agricultural land in the area. However agricultural damages were considered in the economic analyses of the study. This has been discussed in more detail in the economic guidelines developed for this project.

Vulnerable structures located along the Assiniboine River downstream of the provincial dikes were identified using air photos. The ground elevation at each structure was determined based on a combination of LiDAR data and survey data from previous studies (KGS Group, 2013a). The depth of flooding at each structure for flows of 510 m^3/s, 654.1 m^3/s and 792.9 m^3/s (18,000 cfs, 23,100 cfs and 28,000 cfs) was determined based on the water surface profiles discussed in Section 10.1.2. Flooded area maps of the Lower Assiniboine River between Baie St. Paul and Headingley for these three flows have been prepared and are attached in Appendix J.

A conceptual design for individual ring dikes and neighbourhood dikes was then prepared for vulnerable structures in the area, excluding barns, granaries and shops. These types of structures were assumed to be in close proximity to other residential and commercial buildings and, therefore, would already be protected by dikes surrounding those properties. Based on a review of potential dike alignments in the community of St. Francois Xavier and the

neighbourhoods of Golden Oak Cove and Lido Plage, it was estimated that the average dike length per structure was 200 m (656.2 ft).

A 0.6 m (2 ft) freeboard was incorporated into the design of the dikes. A 4 m (13.1 ft) top width and 4:1 side slopes were assumed for the dike geometry. A 2 m^2 allowance per lineal metre of dike for a cut-off trench was incorporated into the analysis, as well as an allowance for the purchase of right-of-way and revegetation. Based on the location and proximity of the structures to the river, it was assumed that 25% of the structures would require riprap protection on half the dike length. Dikes located in close proximity to an outside bend of the river would be areas most likely to require riprap protection.

The purchase of vulnerable properties was assumed necessary when the height of the dike exceeded 2.1 m (6.9 ft) (1.5 m water level plus 0.6 m freeboard (4.9 ft water level plus 2 ft freeboard)). This was considered the highest feasible elevation to which dikes would be constructed at this conceptual level of analysis; and those properties were typically located in areas where the dikes would have been in very close proximity to the River, rendering the construction of these dikes even more difficult. At a more detailed stage of design, a definitive set of criteria would be defined in collaboration with the Province and the affected communities, to determine when a property should be purchased. It is anticipated that a program, similar to the Red River Economic Anomalies Program (Canada-Manitoba, 2000) would be implemented on the Lower Assiniboine River. The Red River program, implemented after the flood of 1997, provided a buyout option to landowners who had one of the following two anomalies:

1. **Physical anomalies** – defined as those properties for which flood proofing was not feasible due to physical limitations of the property, including riverbank instability, restricted lot size or existing building design which prevented raising or moving the building.
2. **Economic Anomalies** – defined as those properties for which the cost to flood proof a home exceeded the value of the building.

The same unit rates of costs used for Options 1a, 1b, 1c and 2a were applied to the Option 2b scheme. The price for the purchase of vulnerable properties was assumed to be carried at fair market value. The average purchase price per structure was assumed to be $370,000, based on a review of 2012 property assessment reports in the area from the Government of Manitoba - Property Assessment website and escalated to 2014$. Crown Land and Property Agency

(CLPA) costs and other fees associated with this option were assumed to be included in the allowance for indirect costs discussed in Section 10.1.7.7.

Table 10.17 summarizes the estimated cost and quantities of Option 2b.

Table 10.17
Combination of Individual Flood Proofing, Neighbourhood Diking and the Purchase of Vulnerable Properties

Design Flow			**18,000 cfs**		**23,100 cfs**		**28,000 cfs**	
Work Item	**Units**	**Unit Price**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**
Estimated Number of Properties Affected			110		270		400	
Land Acquisition	ha	$15,000	32	$0.5	90	$1.3	116	$1.7
Dike Volume	m^3	$20	244,600	$4.9	0	$16.4	1,210,400	$24.2
Cut-off Volume	m^3	$10	40,800	$0.4	99,600	$1.0	118,000	$1.2
Revegetation	ha	$5,500	24	$0.1	71	$0.4	95	$0.5
Riprap	m^3	$90	15,300	$1.4	37,400	$3.4	44,300	$4.0
Purchase of Vulnerable Property	Property	$370,000	7	$2.6	22	$8.1	97	$35.9
Subtotal				**$9.9**		**$30.6**		**$67.5**

10.1.7.6 Seepage Collection Systems

Seepage under and through all dikes posed a major flooding and instability threat to the diking system in 2011. The costs summarized in Table 10.17 for the Combination of Individual Flood Proofing, Neighbourhood Diking and the Purchase of Vulnerable Properties does not include an allowance to address seepage issues. Therefore, to mitigate against seepage that could occur through the foundation below the dikes of the individual flood proofing and neighbourhood diking schemes (Option 2b), a conceptual design was prepared for seepage collection systems.

It was assumed that seepage would be controlled with perforated pipes installed within interceptor drains approximately 1 m wide and 2 m deep (3.2 ft wide and 6.6 ft deep) and backfilled with pervious free draining sand. Similar to the conceptual dike design, it was assumed that the seepage collection systems would cover a 200 m (656.2 ft) length per property. It was also assumed that two pumping wells would be constructed per property. Table 10.18 summarizes the estimated cost for seepage collection systems.

Table 10.18
Estimated Cost for Seepage Collection Systems

Design Flow			18,000 cfs		23,100 cfs		28,000 cfs	
Work Item	**Units**	**Unit Price**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**	**Quantity**	**Cost (Million)**
Excavation Volume	m^3	$10	81,600	$0.8	199,200	$2.0	236,000	$2.4
Fill volume with pipes	m^3	$50	40,800	$2.0	99,600	$5.0	118,000	$5.9
Pumping Wells	pump	$5,000	204	$1.0	498	$2.5	590	$3.0
Subtotal				**$3.8**		**$9.5**		**$11.3**

Other methods of addressing seepage issues, such as incorporating deeper cut-offs under the dikes, should be investigated at the next level of design to determine the most feasible and cost effective solution on a site by site basis. The cut-off concept consists of excavating a trench below the dikes and backfilling with an impervious material (clay) to stop seepage through the foundation below the dikes. The required depth of excavation will vary depending on site conditions, but would have to be deep enough to cut-off the existing sand layer that is present along the Assiniboine River in some areas. The deeper the sand layer, the more costly the option would become.

It should be noted that for the current conceptual phase of analysis it was assumed that every property protected by a neighbourhood dike or with individual flood proofing required seepage control. If there are properties that require less seepage control, the costs could be less.

10.1.7.7 Total Cost Summary

The capital costs for the five upgrade schemes investigated for the Lower Assiniboine River are summarized in Tables 10.19 to 10.23. The total costs combine the estimated costs of the individual items discussed in Sections 10.1.7.1 to 10.1.7.6. Included in the estimate, where applicable, is an allowance for environmental mitigation costs, such as the costs for easements, compensation, and mitigation of environmental impacts. For Option 1a, since the concept consists of upgrading dikes that already exist, the environmental mitigation costs were assumed to be negligible and, therefore, part of the contingency. For Option 1b, the environmental mitigation costs were assumed to be moderate, as the dikes would be new and require the purchase of right-of-way. Also, there are the potential impacts of flooding an area of land that is currently protected by the existing dike alignment, but which would be outside of the proposed setback dike alignment. A cost allowance equivalent to 10% of the total direct cost was added to the estimate of this option. For Option 1c, the environmental mitigation costs were assumed to be minor, as the concept is a combination of upgrading existing dikes and the setback of dikes. A cost allowance equivalent to 5% of the total direct cost was added to the estimate of this option.

For Option 2a, the environmental mitigation costs to extend the dikes to Headingley were assumed to be minor as the dikes would be new and require the purchase of right-of-way. A cost allowance equivalent to 5% of the total direct cost was, therefore, added to the estimate of this option. However, for Option 2b, the environmental mitigation costs were assumed to be negligible and part of the contingency, as there would be no changes to the flooded area; and the flood mitigation works would be local in extent.

As per the economic guidelines set out for the study, an allowance of 20% of the total direct cost was also added to account for indirect costs such as engineering, administration, approvals and owner's costs; and a contingency allowance of 30% of the direct and indirect cost was added for uncertainty. Future operation and maintenance costs for the various upgrade schemes, as well

as administration of the acquired land in the flood plain, were considered in the economic analysis of the options discussed in Section 22.0 of the report, and were estimated to be 1% of the direct construction costs (1% annually for the life of the project, calculated as present value).

Table 10.19
Summary of Costs for Option 1a – Upgrade Existing Dikes (Millions)

Design Flow (cfs)	Dike Construction	Floodplain Land Acquisition and Individual Ring Dikes	Bridge Replacements	Drainage Structures and Pump Stations	Sub Total	Indirect Costs (20%)	Contingency (30%)	Total
18,000	$89	n/a	n/a	$8.7	$97.8	$19.6	$35.2	$152.6
20,000	$115	n/a	n/a	$9.5	$124.7	$24.9	$44.9	$194.6
25,000	$184	n/a	$10.7	$11.0	$205.5	$41.1	$74.0	$320.5
35,000	$323	n/a	$21.3	$12.6	$357.1	$71.4	$128.6	$557.1

Table 10.20
Summary of Costs for Option 1b – Major Setback of Dikes (Millions)

Design Flow (cfs)	Dike Construction	Floodplain Land Acquisition and Individual Ring Dikes	Bridge Replacements	Drainage Structures and Pump Station	Sub Total	Indirect Costs (20%)	Enviro. Mitigation Costs (10%)	Contingency (30%)	Total
18,000	$140	$292.4	n/a	$8.7	$441.2	$88.2	$44.1	$172.1	$745.7
20,000	$165	$292.4	n/a	$9.5	$466.6	$93.3	$46.7	$182.0	$788.6
25,000	$234	$292.4	$10.7	$11.0	$547.8	$109.6	$54.8	$213.7	$925.9
35,000	$404	$292.4	$213	$12.6	$730.4	$146.1	$73.0	$284.9	$1234.4

Table 10.21
Summary of Costs for Option 1c – Combination of Existing Dikes and Setback (Millions)

Design Flow (cfs)	Dike Construction	Floodplain Land Acquisition and Individual Ring Dikes	Bridge Replacements	Drainage Structures and Pump Station	Sub Total	Indirect Costs (20%)	Enviro. Mitigation Costs (5%)	Contingency (30%)	Total
18,000	$121	$153.4	n/a	$8.7	$282.7	$56.5	$14.1	$106.0	$459.4
20,000	$142	$153.4	n/a	$9.5	$305.3	$61.1	$15.3	$114.5	$496.1
25,000	$219	$153.4	$10.7	$11.0	$394.3	$78.9	$19.7	$147.8	$640.7
35,000	$371	$153.4	$213	$12.6	$557.7	$111.5	$27.9	$209.1	$906.3

Table 10.22
Summary of Costs for Option 2a – Extend the Dikes to Headingley (Millions)

Design Flow (cfs)	Dike Construction	Floodplain Land Acquisition and Individual Ring Dikes	Bridge Replacements	Drainage Structures and Pump Stations	Sub Total	Indirect Costs (20%)	Enviro. Mitigation Costs (5%)	Contingency (30%)	Total
18,000	$56	n/a	n/a	$13.8	$70.2	$14.0	$3.5	$26.3	$114.1
20,000	$66	n/a	n/a	$14.6	$80.1	$16.0	$4.0	$30.1	$130.2
25,000	$88	n/a	n/a	$16.3	$104.8	$21.0	$5.2	$39.3	$170.2
35,000	$142	n/a	n/a	$18.0	$159.9	$32.0	$8.0	$60.0	$259.8

Table 10.23
Summary of Costs for Option 2b – Combination of Individual Flood Proofing, Neighbourhood Diking and Purchase of Vulnerable Properties (Millions)

Design Flow (cfs)	Individual Flood Proofing, Neighbourhood Diking and Purchase of Vulnerable Properties	Seepage Collection Systems	Sub Total	Indirect Costs (20%)	Contingency (30%)	Total
18,000	$9.9	$3.8	$13.7	$2.7	$4.9	$21.4
23,100	$30.6	$9.5	$40.1	$8.0	$14.4	$62.6
28,000	$67.5	$11.2	$78.8	$15.8	$28.4	$122.9

A summary of costs for all options is presented on Table 10.24. For Options 1a, 1b, 1c and 2a, the totals at flows of 654 m³/s and 793 m³/s (23,100 cfs and 28,000 cfs) were interpolated from the results at flows of 566 m³/s, 708 m³/s and 991 m³/s (20,000 cfs, 25,000 cfs and 35,000 cfs) that were presented in Tables 10.19 to 10.22. Since the analyses for Option 2b were conducted later in the study, the cost estimates presented in Table 10.23 had already been developed for flows of 654 m³/s and 793 m³/s (23,100 cfs and 28,000 cfs) and, therefore, did not have to be interpolated.

Table 10.24
Summary of Costs for All Options (Millions)

Design Flow (cfs)	Portage la Prairie to Baie St. Paul			Baie St. Paul to Headingley	
	Option 1a	Option 1b	Option 1c	Option 2a	Option 2b
	Upgrade Existing Dikes in Current Location	Major Setback of Dikes to Adjacent Roadways	Combination of Existing Dike Upgrades and Setback Dikes to Adjacent Roadways	Extend the Existing Dikes to Headingley	Combination of Individual Flood Proofing, Neighbourhood Diking and Purchase of Vulnerable Properties
18,000	$152.6	$745.7	$459.4	$114.1	$21.4
23,100	$272.7	$873.7	$585.7	$155.0	$62.6
28,000	$391.5	$1018.4	$720.4	$197.1	$122.9

10.1.8 Conclusions

Based on the estimated costs of the options, the most feasible option to increase the capacity of the Assiniboine River between Portage la Prairie to approximately Baie St. Paul is to upgrade the existing dikes in their current location (Option 1a). Some local improvements to the dike alignment would likely be incorporated at the detailed design stage to address riverbank instabilities and improve access should this option proceed to the next level of design.

From Baie St. Paul to the City of Winnipeg limits in Headingley, the most feasible option is a Combination of Individual Flood Proofing, Neighbourhood Diking and Purchase of Vulnerable Properties (Option 2b). At this level of study, it was assumed that seepage collection systems would be incorporated with the option.

10.2 EMERGENCY DIVERSION AT HOOP AND HOLLER BEND

10.2.1 Introduction

A wide variety of flood protection works is being considered in the study of the Assiniboine River & Lake Manitoba Basins (AR&LMB). One of many emergency actions that were deployed in the flood of 2011 was the breaching of the dike along the south bank of the Assiniboine River at the Hoop and Holler Bend. The plan in 2011 was to release excess flood waters into the La Salle River via the Elm River to relieve the growing concern for failures along the dikes on the Lower Assiniboine River. A limited amount of flow was successfully released in 2011. However, experience showed that the scheme could be improved with construction of a control structure and possibly a dedicated release channel. These would be used under well-defined conditions acceptable to the stakeholders involved.

The experience in 2011 and the concept of emergency releases at the Hoop and Holler Bend were reviewed in the “Manitoba 2011 Flood Review Task Force Report” (MFRTF, 2013). Recommendation #86 in that report stated:

> *“As one component of a properly engineered dike system, determine the capacity of a permanent controlled wasteway (either north or south of the Assiniboine River), and construct such a structure to pass Assiniboine River flows in excess*

of the combined capacity of the Portage Diversion and Assiniboine River channel and dikes. The facility should be sized with due consideration of the potential flood risks/benefits and capacity of the landscape to accommodate the design flow through the breach".

This recommendation has been considered for the current study. The information described herein summarizes the key aspects of the study findings, including assumptions, analyses of operation and approximate construction costs.

The findings to date are that the scheme could be viable to a limited extent without major impacts to downstream interests along the La Salle River. However, that would ideally involve construction of a stoplog-controlled structure at the Assiniboine River in conjunction with a separate excavated channel (or "wasteway" as suggested by the 2011 Flood Review Task Force) (MFRTF, 2013) that would convey flow effectively away from the Assiniboine River. It was not clear whether the concept recommended by the Flood Review Task Force would include a "wasteway" that avoided flooding of the landscape entirely. If such flooding were to be avoided, excess flows from the Assiniboine River up to possibly 110 m^3/s (3,900 cfs) could be conveyed during emergency conditions. The cost of that scheme would be high, and must be weighed against other options being considered in this study.

Larger amounts of diverted flow (i.e., greater than approximately 110 m^3/s or 3,900 cfs) could only be allowed if considerable upgrades were made to protect the communities of Starbuck, Sanford, possibly La Salle, and other critical locations along the Elm Creek Drain and La Salle River. The cost of that undertaking would clearly be prohibitive and has not been assessed in detail.

10.2.2 Preliminary Concepts

10.2.2.1 Key Factors and Design Considerations

The Lower Assiniboine River from Portage la Prairie to Headingley is vulnerable to excessive river flows. Considerable damage to existing infrastructure and agricultural interests along the river could be caused if the dikes were to be overtopped or fail during an event such as in 2011. In the 2011 Flood, an emergency action plan was carried out that involved breaching of the south embankment of the Assiniboine River at the Hoop and Holler Bend. Some limited outflows

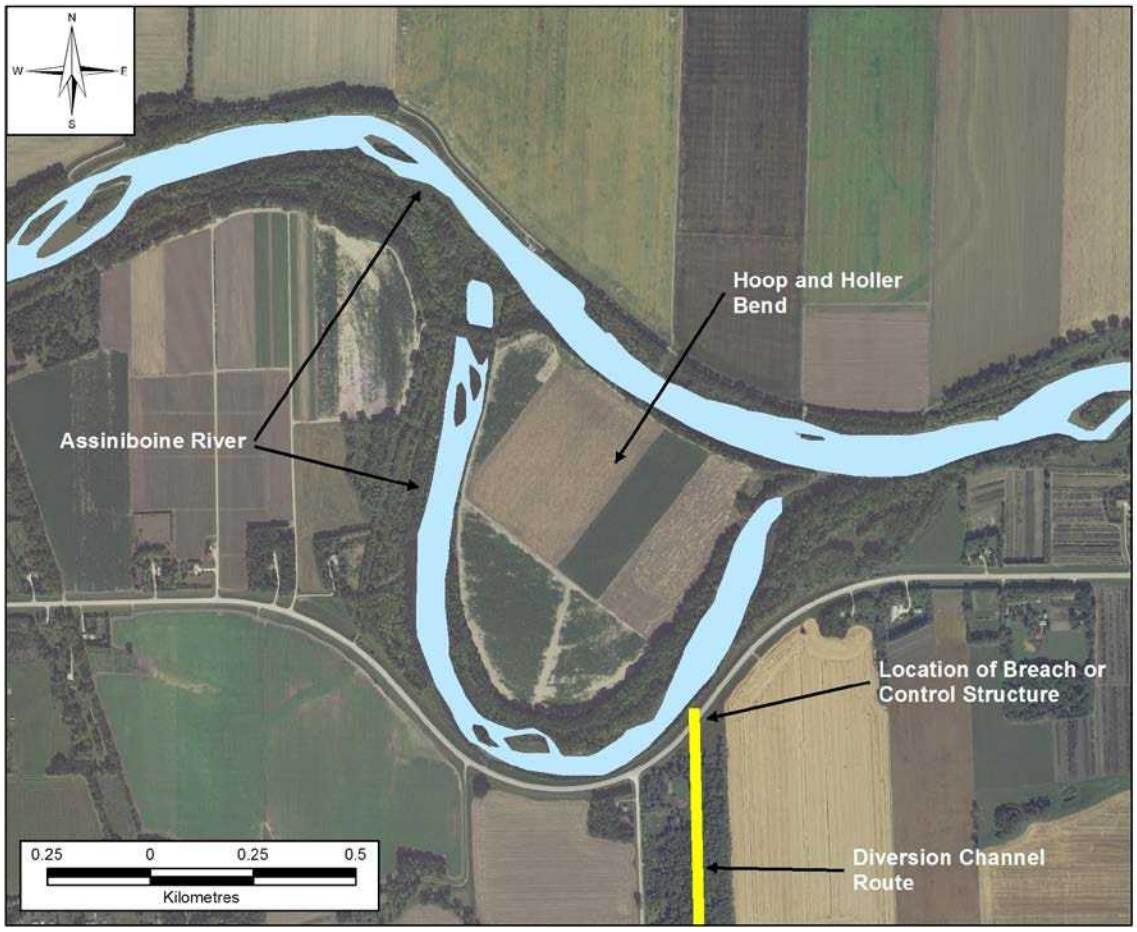
through the breach were achieved and continued over a period of several days. Some damages were incurred downstream. However, the release was estimated to be only about 20 m^3/s (700 cfs). It has been speculated that as much as 40 m^3/s (1,400 cfs) could have been released, if a longer, more extensive breach of the river bank had been carried out.

The main reasons for this limited release in 2011 were that the breach itself was limited in size (less than 40 m (131.2 ft) wide), and the natural ground elevations immediately downstream of the breach were relatively high compared to the river flood stage. Although acceptable for the circumstances in 2011, it became evident that the breach excavation at the Hoop and Holler Bend would have to have been considerably larger to be effective on a significant scale.

A review of the events in 2011 and consideration of possible diversion schemes has led to a number of observations and conclusions that have shaped the selection of options:

- Release of flow from the Assiniboine River should be planned to occur at, or within about 1,000 m (3,281 ft) of, the Hoop and Holler Bend so that the released flow does not enter the upper reaches of the La Salle River that is close by. Figure 10.8 shows an aerial image of the area. The upper reach of the La Salle River meanders northeastward parallel to the Assiniboine River and has a very limited discharge capacity. Enlargement of that channel would be expensive and environmentally damaging. It would for example require two new major bridges at the Trans-Canada Highway, as well as enhancement of flood protection at the community of Elie. Diverted flows of only about 20 m^3/s (700 cfs) would be possible without incurring these large costs.

Figure 10.8
Location Map – Hoop and Holler Bend



- The topography of the riverbank in the area of the Hoop and Holler Bend and the ground to the south requires that a dedicated channel be excavated to induce the flow to move to the south into the upper watershed of the Elm Creek Drain.

- If flow in the Lower Assiniboine River downstream of the Hoop and Holler Bend were to be restricted to 510 m^3/s (18,000 cfs), the water level at the diversion point at Hoop and Holler Bend must be approximately El 257.4 m (El 844.5 ft). It is recognised that the current reliable flow capacity of the Lower Assiniboine River dikes, as discussed in Section 10.1.5, was estimated to be 425 m^3/s (15,000 cfs). However, at this flow, the water level at Hoop and Holler Bend would be lower. This would negatively impact the capacity of the diversion scheme and would, therefore, require significant increases in the excavation quantities of the diversion channel. For this reason, the 510 m^3/s (18,000 cfs) capacity has been adopted as the base case for the analysis of the potential diversion into the Elm Creek Drain / La Salle River system. Improvements to the existing Lower Assiniboine River diking system, such as those presented in Section 10.1, would be necessary to convey a flow of 510 m^3/s (18,000 cfs) reliably, without the implementation of emergency measures.

- Damages could be substantial if a dedicated pre-excavated diversion channel (or "wasteway" as suggested by the 2011 Flood Review Task Force (MFRTF, 2013)) that extends to the La Salle River near Starbuck were not provided. The limited release in 2011 did not have such a dedicated diversion channel, and as a result, triggered damage claims of approximately $6,000,000, for a relatively small, short-duration diversion of flow. Greater releases could result in substantially larger claims.

- Diversion of flow to the ***north*** of the Assiniboine River does not appear possible without a dedicated pre-excavated channel. For that route to be beneficial, the release point would have to be in an area close to the Hoop and Holler Bend (but on the opposite (north) side of the river). From there, a channel would be required to direct the water to a location well downstream of the vulnerable dikes along the Assiniboine River. This would appear to require a channel that would skirt communities such as Poplar Point and St. Francois Xavier, as well as the considerable number of private dwellings along the north bank of the Assiniboine River. The dedicated channel would have to be more than 65 km (40 mi) in length, along with bridges at critical road crossings, including at the TransCanada Highway. Given the obvious high costs of such a scheme, no further comparisons of this option on the north side of the river with a southern route were deemed necessary.

- The bank full discharge capacity of the La Salle River at Starbuck and Sanford is estimated to be approximately 110 m^3/s (3,900 cfs). At La Salle, the bank full capacity may be greater, depending on the coincident water levels in the Red River. Flows up to possibly 190 m^3/s (6,700 cfs) could be safely passed at La Salle under certain circumstances.

- Any diversion from the Assiniboine River to the La Salle River system should be planned so that the total of the released flow and the natural flow magnitude existing in the La Salle River at the time would not cause flood damages along the La Salle River (i.e., not exceed the bank full capacities quoted in the point above). The operation would clearly be vulnerable to the potential for sudden unforeseen precipitation to significantly augment the local inflows, and in combination with the diverted floodwaters, inadvertently cause damages to properties along the route.

- The simplest form of control structure at the Hoop and Holler area would be a rock fill overflow weir and chute leading to the release area or dedicated discharge channel. However, such an uncontrolled structure would not permit protection against the sharp increases in stage associated with temporary conditions during spring ice breakup. An uncontrolled overflow could release floodwaters at inopportune times and, therefore, may not be desirable, particularly if there is no dedicated channel downstream.

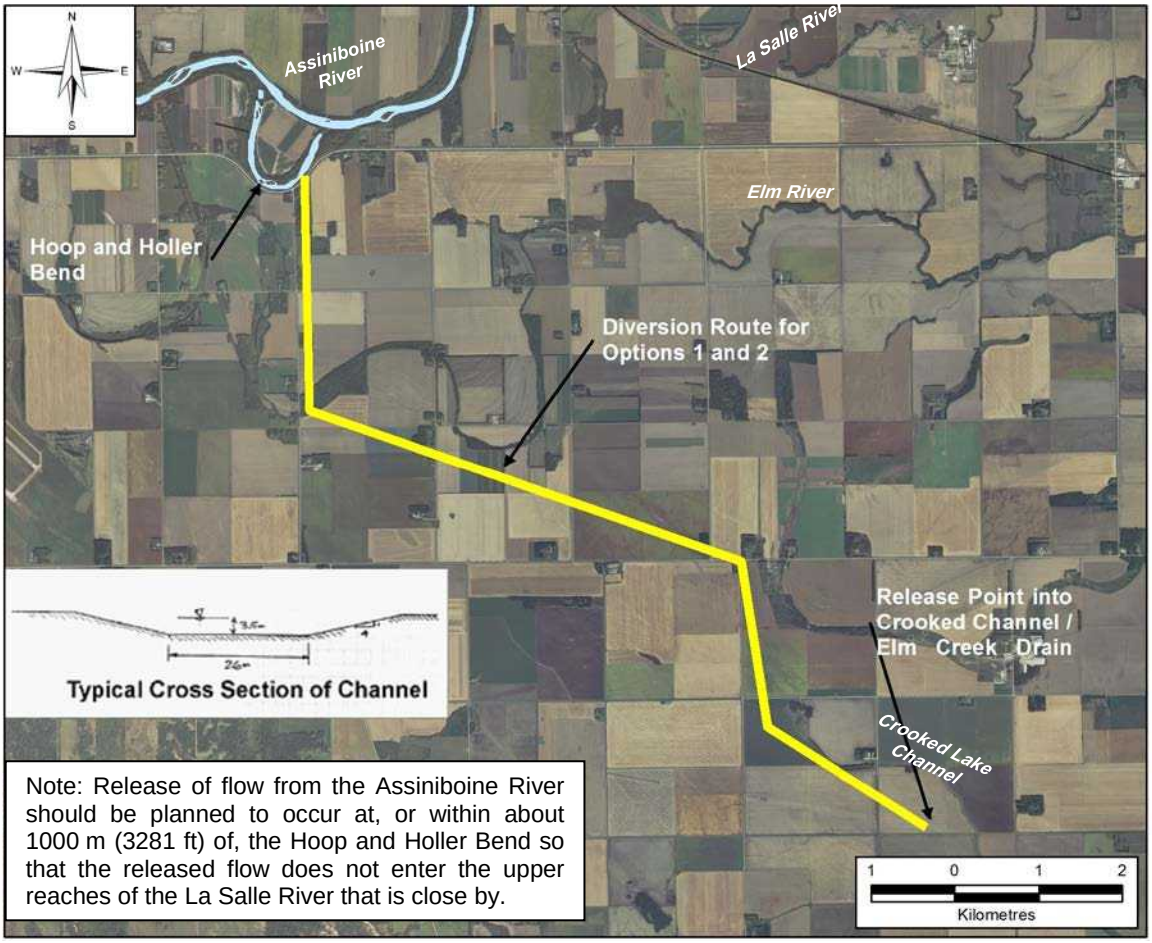
- A more reliable structure would be a reinforced concrete structure equipped with stoplogs that can prevent overflow if it is not needed. When the stoplogs are removed, the structure would permit safe diversion of the desired emergency discharge.

Based on the points discussed above, four options appear to be possible, assuming that the flow in the Lower Assiniboine River must be restricted to 510 m^3/s (18,000 cfs). This is a somewhat arbitrary limit chosen to be associated with a relatively low risk of failure of the dikes along the Assiniboine River. The options are as follows, and all would be initiated at the Hoop

and Holler Bend as shown in Figure 10.9. This is considered the optimum diversion point where the river bank is relatively low, and a channel would require the least excavation to permit flow to reach the LaSalle River:

- ***Option 1 –*** The 2011 Task Force recommended a control structure and a "wasteway" at the Hoop and Holler Bend of the Assiniboine River. However, since such a diversion would be expected to be used very infrequently, the first option has been considered to be a diversion channel (i.e., "wasteway") without a control structure. This would require emergency excavation of a breach in the south river bank of the Assiniboine River when required to divert excess floodwaters. Achieving a diversion of 110 m^3/s (3,900 cfs) would require that the breach be combined with a pre-excavated channel downstream of the breach in the river bank that would be about 4 m deep, 26 m wide (13.1 ft deep, 85.3 ft wide) and approximately 12 km (7.5 mi) in length. This would require a breach length of less than 50 m (164 ft), as was shown to be readily achievable in 2011. The breach and the channel downstream would assure that the desired discharge could be achieved through the relatively high ground that lies to the south of the Hoop and Holler Bend. It would release the floodwaters into the upper Elm Creek Drain, where it would be conveyed by that drain and the floodplain adjacent to it. The channel route would require five new bridges to avoid impact to the local road system. Road closures and realignments could also be considered at a more detailed stage of study; however, the savings from eliminating one or two bridges would be minor (about 2% of the total project cost) and, therefore, has not been considered at this conceptual phase of study. Considerable overbank flooding would occur along the Elm Creek Drain, possibly as much as a kilometre in width, up to the point where the flow entered the La Salle River. Damages in excess of those incurred in 2011 could be expected in the Elm Creek Drain area under the conditions of this option. A maximum of 110 m^3/s (3,900 cfs) is proposed for Option 1.
- ***Option 2 –*** The next level of option would be the installation of a control structure with stoplogs that could be removed in an emergency to allow the release of flood waters into the upper Elm Creek Drain in the La Salle River watershed. The structure would be located at the Hoop and Holler Bend, and would require a 12 km (7.5 mi) long channel, as described for Option 1, to assure that the desired discharge could be achieved through the relatively high ground that lies to the south of the Hoop and Holler Bend. A conceptual route of the channel is shown in Figure 10.9. The restriction for release of flood waters from the Assiniboine River appears to be the La Salle River discharge capacity at major downstream centres such as Starbuck and Sanford. A maximum discharge capacity of 110 m^3/s (3,900 cfs) is proposed for both Options 1 and 2.

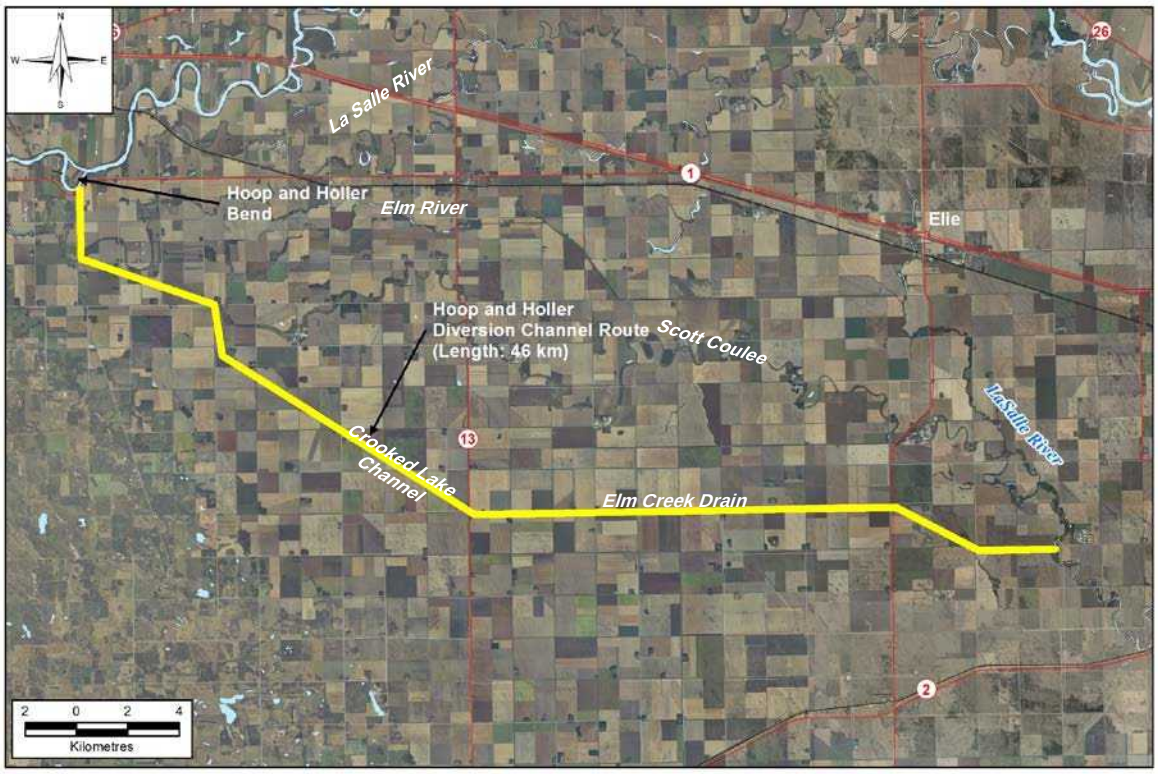
Figure 10.9
Conceptual Location of Diversion Channel for Options 1 and 2



- ***Option 3 –*** An alternative that may be more acceptable than Options 1 or 2 to local stakeholders might be a combination of the structure adopted in Option 2, with a dedicated discharge channel that could safely pass floodwaters some 46 km (28.6 mi) to the La Salle River without causing overbank flooding along the route. This would fulfil the concept that has been recommended by the 2011 Flood Review Task Force (MFRTF, 2013). Damage claims of the type and magnitude that occurred in 2011, and that would be incurred in varying degrees under Options 1 and 2, would be averted. The preferred route of this channel would originate from a structure location at the Hoop and Holler Bend, similar to Option 1 and 2, and then continue southeast along Crooked Channel and the Elm Creek Drain to meet the La Salle River approximately 7 km (4.4 mi) upstream of Starbuck. The channel would assure that the desired discharge could be achieved through the relatively high ground that lies to the south of the Hoop and Holler Bend. The conceptual location of the channel is shown in Figure 10.10. The channel route would require at least 12 new bridges to totally avoid impact to the local road system. Other routes, including diversion to the Morris River upstream of Brunkild were briefly reviewed, but rejected as being inferior to the one shown in Figure 10.10. All required substantially more excavation and longer diversion paths. It should also be noted that the existing drains along the route are very small relative to the desired

discharge capacity, and estimation of excavation quantities for the channel has ignored the conveyance provided by those small channels.

Figure 10.10
Conceptual Location of Diversion Channel for Option 3



- ***Option 4 –*** An extension of Option 3 could also be considered, whereby the diverted flow from the Assiniboine River could be increased above the limit of 110 m^3/s (3,900 cfs). However, it would have to be accompanied by the costs involved in a larger diversion structure, a larger excavation of the diversion channel, as well as enhancements of flood protection measures at critical downstream points such as Starbuck, Sanford, possibly La Salle, and at critical locations along the channel in between. This would be inordinately expensive and has not been planned to be studied in further detail.

Estimated costs of the options are summarized in Section 10.2.7.

10.2.2.2 Other Options and Considerations

Options 1 and 2 were developed to provide clear certainty of being able to pass the desired flow from the Assiniboine River with minimum effort at the time of a flood emergency. The schemes have included a 12 km (7.5 mi) long channel extending through the high ground adjacent to the

Assiniboine River to achieve this objective. However, it was recognized that it may be feasible to pass that flow with a shorter, wider, and possibly less expensive channel. Detailed analysis to prove this speculation would require a level of two-dimensional hydraulic analysis of overland flow over the complex topography of the area that was not appropriate for this screening level of study.

Furthermore, a lower target flow may be acceptable for the diversion from the Assiniboine River (i.e., less than the target of 110 m^3/s (3,900 cfs) used to define the options described above), which would correspondingly reduce the required size of the control structure and the extent of excavation for the channel. In light of these complications that would require extensive optimization to resolve, the notion of potentially less expensive alternatives to Options 1 and 2 is proposed to be addressed in future planning if the Province were to decide this had merit.

Similarly, other options might also be less expensive than the ones described herein; but those options would only be appropriate if the flow in the Lower Assiniboine River was allowed to be greater than 510 m^3/s (18,000 cfs). For example, investment in upgrading the dikes along the Lower Assiniboine River might allow as much as 710 m^3/s (25,000 cfs) to be passed downriver from Hoop and Holler safely. If that was the case, then the river water level at the point of diversion at Hoop and Holler could be higher than for the design case described in the previous sections. If the Hoop and Holler diversion were to be combined with the enhanced Assiniboine dikes, this higher water level at the point of diversion could permit a less costly control structure and channel, for the same target magnitude of diverted flow.

10.2.3 Conceptual Design Details of Control Structure and Related Works

The concept of the control structure for Options 2 and 3 was initially envisaged early in this study to be a simple vertical drop structure with stoplogs. It would have a floor slab at El 253 m (El 830 ft) and an overflow crest at El 255.8 m (El 839 ft). The crest would be about 1 m (3.3 ft) above the normal water levels in the Assiniboine River. The structure would have piers spaced at approximately 5 m (16 ft), and a bridge deck from which the stoplogs could be removed when required to release flow into the Elm Creek Drain / La Salle River watershed. The bridge would be used as a crossing for PTH 331. Sketches of the structure are shown in Figures 10.11 and 10.12 (plan and cross section of the structure). The dimensions shown have been based on

well-established design guidelines for vertical drop structures provided in the textbook "Hydraulic Structures", by C.D. Smith (1974). The layouts of structures and channels are considered conceptual in nature and would require structural and geotechnical analyses to prove that they are stable.

A similar arrangement of structures was adopted in the initial studies of a new control structure on Dauphin Lake (explained in more detail in Section 13.0). However, late in that portion of the studies, suggestions by MIT led to the discovery that an alternate arrangement of the control structure incorporating a horizontal sluice with intermediate piers and stoplogs could result in a modest reduction in overall cost of the combination of structure / channel. The structure would have to be wider (i.e., more discharge bays) than shown in Figure 10.11, but the extra cost incurred would be more than offset in the reduction in excavation for the shallower channel downstream. Cursory checks for the structure / channel at the Hoop and Holler Bend revealed the same conclusion. However, given the high cost of this scheme at Hoop and Holler in the overall plan of flood protection in the Assiniboine Valley, further refinement of the concept was not considered necessary at this stage of study. The layout shown in Figures 10.11 and 10.12 is considered adequate for planning purposes and for establishing approximate budgetary cost estimates. It should be emphasized that, if this scheme takes on a higher priority in future studies, the concept of a horizontal sluice is almost certain to be preferable over the vertical drop structure shown herein.

Figure 10.11
Cross Section of Conceptual Design of Control Structure

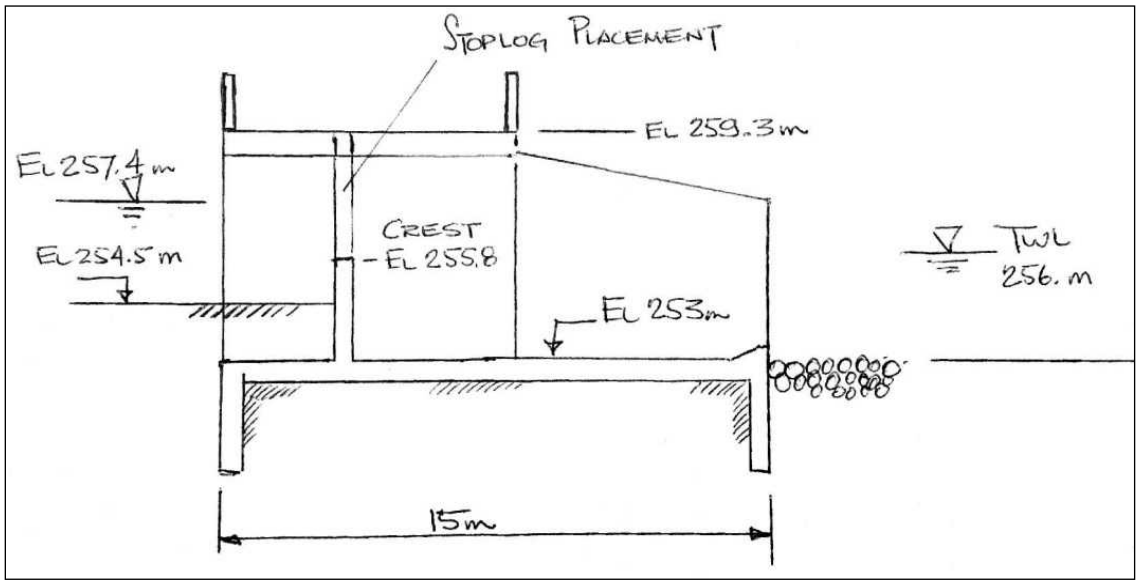
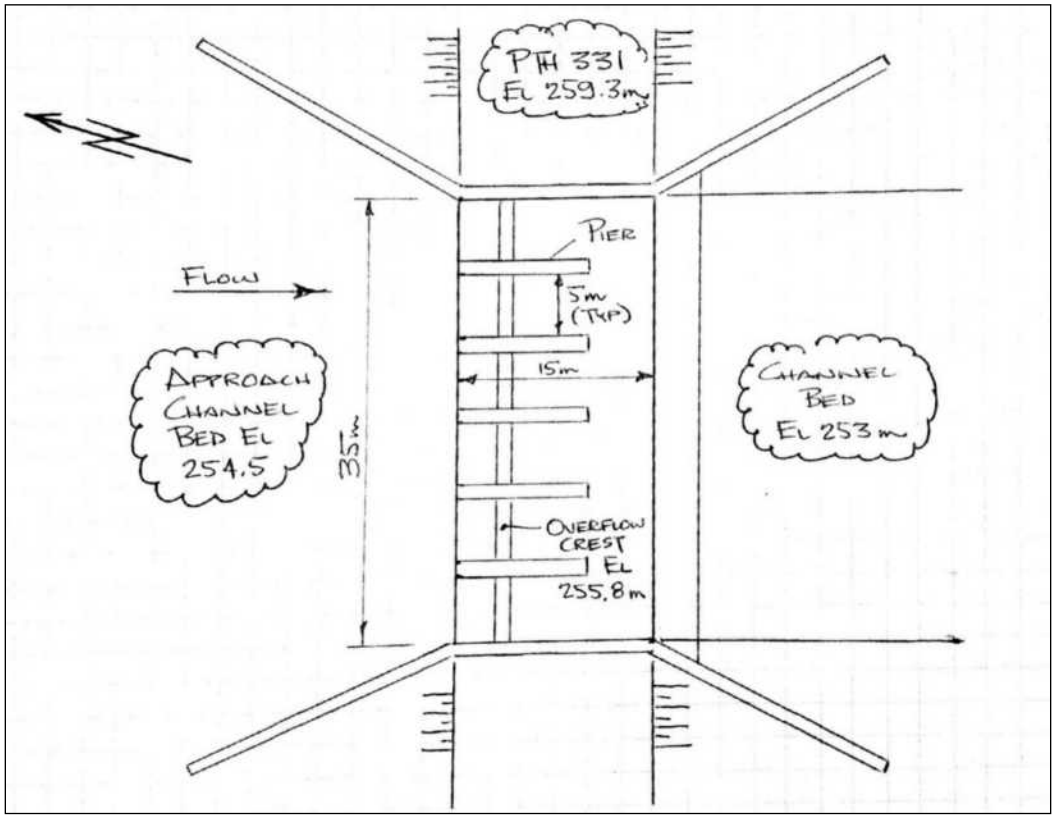


Figure 10.12
Plan View of Conceptual Design of Control Structure



10.2.4 Conceptual Design of Diversion Channel

Figure 10.9 shows the proposed route of the channel required for Options 1 and 2. Figure 10.13 shows the profile of the control structure and the diversion channel for those options. Since the concept is to release excess flood waters into the Elm Creek Drain and its surrounding floodplain, the channel ends before extending through to low ground that is many kilometres downstream. The downstream-most portion of the channel is envisaged to widen and gradually slope up to the surrounding ground level. The material excavated for the channel would be placed along the length of each side of the channel to contain the diverted flow as required. In the final planning for such a scheme, detailed design of the outlet arrangement would be required. However, at this stage of study the concept shown in Figure 10.13 is considered adequate.

A tentative route for a dedicated diversion channel for Option 3 has been selected as shown in Figure 10.10. Side slopes of 1V:4H as shown in Figure 10.9 have been selected at this conceptual stage of study. Flatter side slopes may possibly be shown eventually to be needed after testing of soil samples from the area and further geotechnical analyses of the slope stability. However, the cross section shown in Figure 10.9 is considered adequate for this level of study.

Figure 10.13
Profile of Conceptual Channel Design for Options 1 and 2

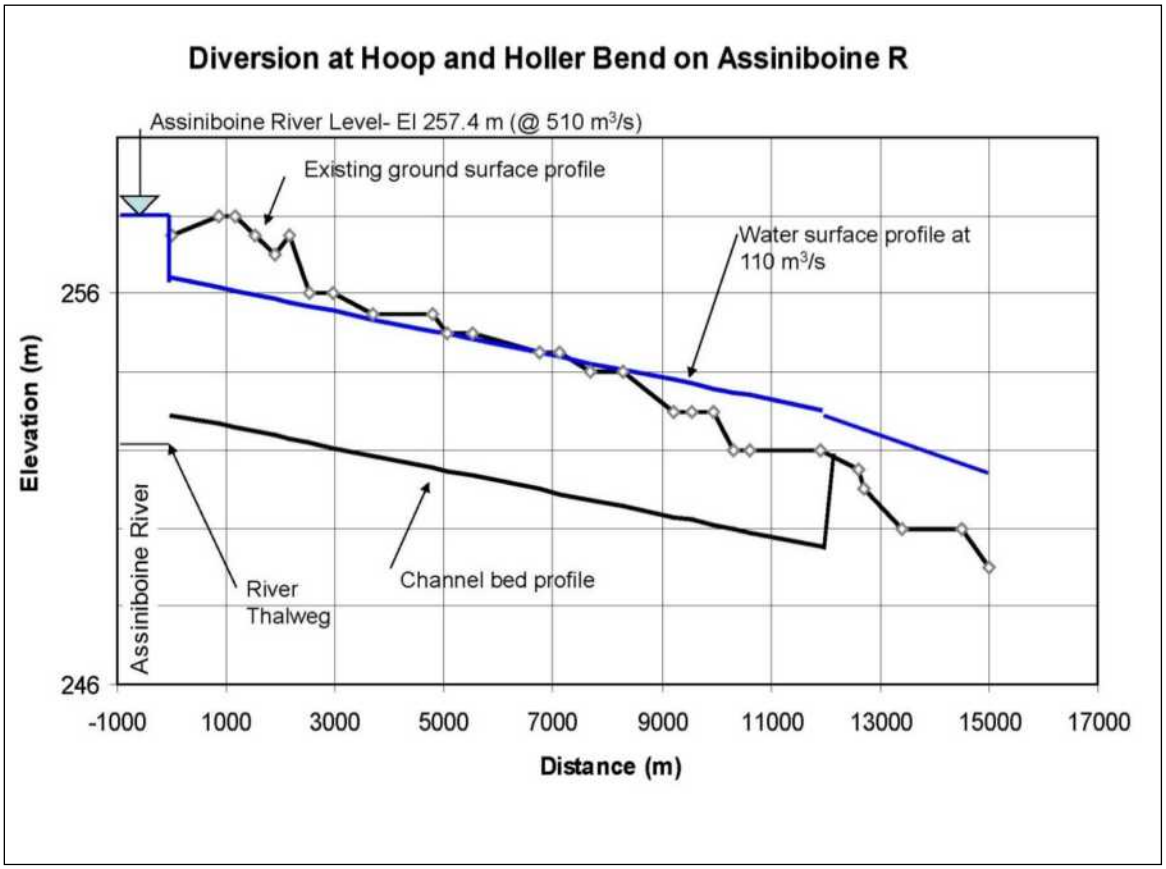
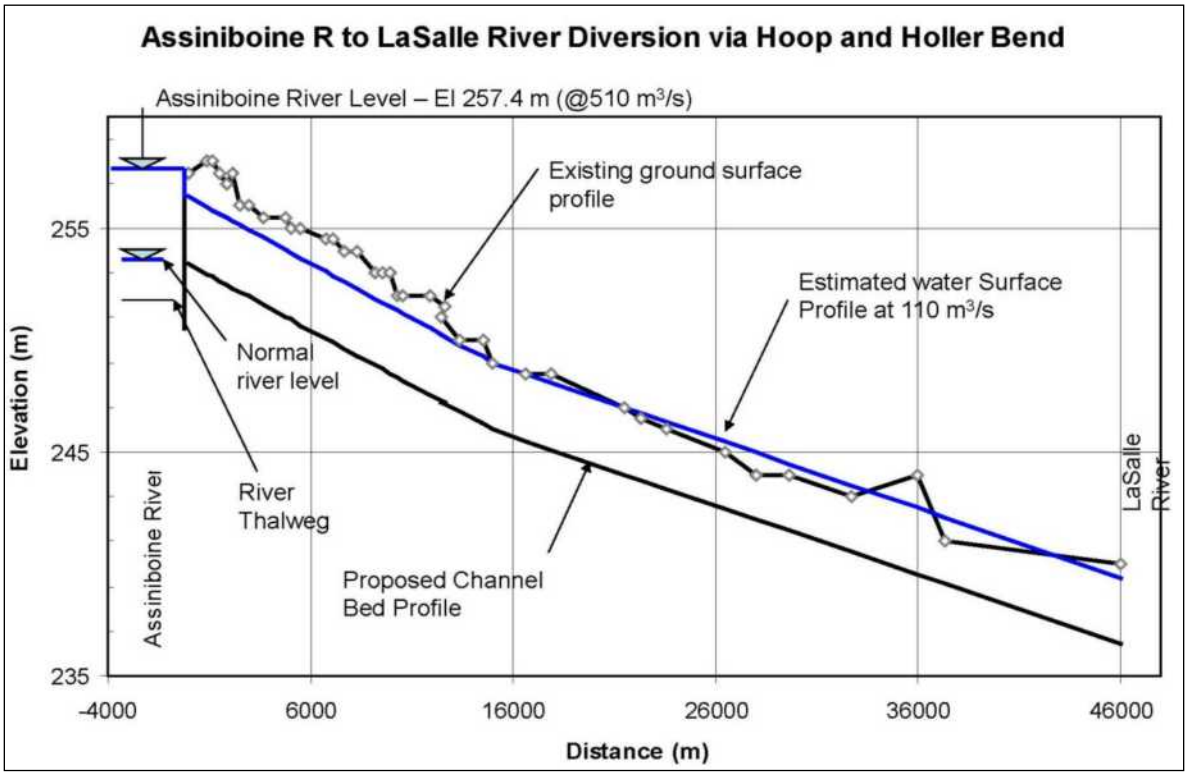


Figure 10.14 shows a similar profile for the longer channel associated with Option 3.

Figure 10.14
Profile of Conceptual Channel Design for Option 3



10.2.5 Hydrological Studies

The flood event of 2011 was extraordinary. Based on the statistics of the river flows from the historical record, this magnitude of flood has a very small chance each year of being exceeded.

Preliminary calculations in combination with the known limits of existing flood protection works were done to estimate the annual likelihood of requiring the activation of the emergency measures at the Hoop and Holler area. The maximum annual probability of needing to initiate the emergency measures would be based on the assumptions as follows:

- Diversion of flow into the Portage Diversion up to a maximum capacity of 709 m^3/s (25,000 cfs). It is recognized that additional flow could be passed through the diversion as it was in 2011, but only at an increasing risk of failure in that system.
- Maximum release into the Assiniboine River downstream of the Portage Diversion of 510 m^3/s (18,000 cfs). It is recognized that additional flow could be passed through this reach, but it would cause an increasing risk of failure along the diking system between Portage and Headingley. It is also recognised that the current reliable flow capacity of

the Lower Assiniboine River dikes, as discussed in Section 10.1.5, was estimated to be 425 m^3/s (15,000 cfs). Improvements to the existing Lower Assiniboine River diking system would be necessary to convey a flow of 510 m^3/s (18,000 cfs) reliably, without the implementation of emergency measures.

These calculations indicate that the 110 m^3/s (3,900 cfs) diversion at Hoop and Holler, combined with an increase in capacity to the Lower Assiniboine River diking system from 425 m^3/s (15,000 cfs) to 510 m^3/s (18,000 cfs), would boost the existing flood management capability on the Lower Assiniboine River from 1,133 m^3/s (40,000 cfs) to 1,329 m^3/s (46,900 cfs). Based on the historical statistics of river flows discussed in Section 5.0 of the report, this would reduce the frequency of having that capability exceeded from the existing once in 55 years, to once in 95 years with the Hoop and Holler Diversion.

If the Portage Diversion were pushed to the limits that proved successful in 2011, about 960 m^3/s (34,000 cfs), or if that channel were expanded to permit the safe release of this magnitude of flow to Lake Manitoba, combined with an increase in capacity to the Lower Assiniboine River dikes to 654 m^3/s (23,100 cfs), the capability for flood management would further increase to 1,617 m^3/s (57,100 cfs). The frequency of floods exceeding this limit would be 1:200 years on average.

Of course, if the limits at the Portage Diversion or of the Lower Assiniboine River dikes were further increased, or if other protection measures were to be developed and used, then the frequency of exceeding the capability of flood management would decrease further. The numbers expressed above are provided at this point only as a perspective on the alternatives and their performance.

10.2.6 Environmental & Social Considerations

The concept of diversion of excess floodwaters from the Assiniboine River south from the Hoop and Holler Bend has been adamantly resisted by the local residents. A Resolution of Council was prepared by the Rural Municipality of Cartier and submitted to MIT in June of 2013. It is clear that there would be substantial resistance to Options 1 or 2, and possibly even Options 3 and 4, as they would require acquisition of prime agricultural land. Considerable overbank flooding would also occur along the Elm Creek Drain downstream of the proposed diversion channel for Options 1 and 2, up to the point where the flow could enter the La Salle River. In

addition, there could be other environmental effects that would emerge from a thorough investigation of the options. Other impacts such as the transfer of water from the Assiniboine River Basin to the La Salle River Basin, as well as the potential impact of increased river flood flows on the La Salle River could require further consideration. On this basis, the environmental mitigation costs were assumed to be minor for Option 3 and moderate for Options 1 and 2. These costs were incorporated into the cost estimates summarized in Section 10.2.7. Since Option 4 would accommodate larger flows than Option 3, which could significantly impact the La Salle River, moderate environmental mitigation costs would be anticipated for this option.

10.2.7 Cost Estimation

Cost estimation was preliminary at this screening stage of analysis. Estimated capital costs and other pertinent information are summarized in Tables 10.25 to 10.27 for Options 1 to 3. Option 1 includes the capital cost to initiate the breach by excavation of an earth plug with excavation equipment. The cost is relatively small (likely less than $100,000). It has not been included with the other capital costs in Table 10.25 as it would only occur at some future juncture if diversion of floodwaters was required.

Option 3 represents the configuration of diversion and dedicated “wasteway” that was interpreted in this study as being recommended by the 2011 Flood Review Task Force.

Estimated costs for Option 4 have not been developed, as that scheme was not deemed to be practical.

Table 10.25
Summary of Design Parameters and Estimated Capital Costs for Option #1 – Emergency Breach of Assiniboine Dike When Needed

Item	Description	Cost
Pre-excavated channel from PTH 341 to the Elm Creek Drain	12-km length, volume of excavation – 5,420,000 m^3 @ \$10/m^3	\$54,200,000
Revegetation	42 ha @ \$5,500/ha	\$231,000
Five Municipal Bridges	400 m^2 per bridge @ \$2,500/m^2	\$5,000,000
Land Acquisition	300 mx12,000 m @ \$15,000/ha	\$5,400,000
Environmental Mitigation Costs	10% of Total Direct Costs	\$6,483,000
Indirect Costs (Owners Costs, Engineering, Administration and Approvals)	20% of Total Direct Costs	\$12,966,000
Contingency Allowance	30% of Total Direct and Indirect Costs	\$25,284,000
	Total Cost	**\$109,564,000**

Table 10.26
Summary of Design Parameters and Estimated Capital Costs for Option #2 – Controlled Release through a Control Structure with Short Dedicated Channel to Elm Creek Drain

Item	Description	Cost
Concrete Control Structure	Structure dimensions: 15 m x 37 m - 555 m^2 @\$10,000 per m^2	\$5,550,000
Pre-excavated channel from PTH 341 to the Elm Creek Drain	12-km length, volume of excavation – 5,420,000 m^3 @ \$10 per m^3	\$54,200,000
Revegetation	42 ha @ \$5,500/ha	\$231,000
Five Municipal Bridges	400 m^2 per bridge @ \$2,500/m^2	\$5,000,000
Land Acquisition	300 mx12,000 m @ \$15,000/ha	\$5,400,000
Environmental Mitigation Costs	10% of Total Direct Costs	\$7,038,000
Indirect Costs (Owners Costs, Engineering, Administration and Approvals)	20% of Total Direct Costs	\$14,076,000
Contingency Allowance	30% of Total Direct and Indirect Costs	\$27,449,000
	Total Cost	**\$118,944,000**

Table 10.27
Summary of Design Parameters and Estimated Capital Costs for Option #3 – Controlled Release through a Control Structure with a Dedicated Channel to the La Salle River

Item	Description	Cost
Concrete Control Structure	Structure dimensions: 15 m x 37 m - 555 m^2 @ $10,000 per m^2	$5,550,000
Excavation for Dedicated Channel	46 km long channel – 18,960,000 m^3	$189,600,000
Revegetation	161 ha @ $5,500/ha	$886,000
Bridges	12 bridges estimated to have deck surface areas of 400 m^2 each; unit price of $2,500 per m^2 for 10 local bridges, $7,000 per m^2 for two PTH bridges + $3M per bridge for approaches	$21,600,000
Land Acquisition	1,400 ha @ $15,000 per ha	$21,000,000
Environmental Mitigation Costs	5% of Total Direct Costs	$11,904,000
Indirect Costs (Owners Costs, Engineering, Administration and Approvals)	20% of Total Direct Costs	$47,617,000
Contingency Allowance	30% of Total Direct and Indirect Costs	$89,282,000
	Total Cost	**$386,890,000**

10.2.8 Conclusions

Any one of the options described above appears to be relatively expensive in comparison to the small relief of flood passage that would be provided in the Assiniboine River. Nevertheless, the results developed are considered representative of these concepts and have been used as a basis for comparisons with other flood mitigation measures along the Assiniboine River.

All options for the diversion at Hoop and Holler Bend would be vulnerable to unforeseen precipitation after the diversion commenced. There would be potential for the combination of diverted flow and local runoff from unpredictable rainstorms to cause damage.

Option 4 has been deemed to be impractical, given the far ranging impact on the communities downstream along the La Salle River, and has been dismissed from further consideration.

10.3 INDIVIDUAL AND COMMUNITY FLOOD PROTECTION

Without upgrades to the Lower Assiniboine River dikes, there could be widespread flooding along the Lower Assiniboine River area and overland flows could spill into the La Salle River system during a large flood. The potential extent of the flooded area and estimated damages due to a breach or overtopping of the Provincial Assiniboine River dikes was discussed in Section 8.6.1. An alternative solution for mitigating flood damages in the Lower Assiniboine River area consists of protecting the homes and communities individually (similar to what was done within the Red River Valley) rather than upgrading the Assiniboine River dikes. This option was evaluated for the study and is discussed within this section of the report.

10.3.1 Conceptual Design and Cost estimate

Individual protection could be achieved in the Lower Assiniboine River area by raising property foundations or constructing ring dikes. This would be similar to the individual flood proofing concept (Option 2b) discussed in Section 10.1.7.5 of the report for the reach that extends between Baie St. Paul and Headingley. However, instead of the scheme being confined to the structures located in close proximity to the river, structures located within the Lower Assiniboine River area and also within the La Salle River Basin would require individual protection.

A rigorous hydraulic analysis and modeling of the Lower Assiniboine River overflows would be necessary to define with precision all of the structures that would require protection under this scheme and to what elevation. Since such a detailed level of analysis was outside the scope of the current study, simplifying assumptions were used and the results of the overflow analyses conducted for the Lower Assiniboine River damage model discussed in Section 8.6.1 were adopted. The total number of structures requiring protection (excluding structures located within the communities listed below as well as the area that extends between approximately Baie St. Paul and Headingley) were estimated based on the assumptions described in Section 8.6.1.7 of the report and the estimated number of structures that incurred damages. As discussed in Section 8.6.1.7, for a flood resulting from a flow of 708 m^3/s (25,000 cfs) on the Assiniboine River, the total number of structures affected ranged between approximately 650 to 900 depending on the overflow scenario, and between approximately 1,050 and 1,280 for a flood resulting from a flow of 1,303 m^3/s (46,000 cfs).

In lieu of individual flood protection, structures located within communities would be protected with community ring dikes. It was assumed that the communities of Newton, Oakville, Elie and St. Eustache would require ring dikes to protect against floods that exceed the flow of 708 m^3/s (25,000 cfs) on the Assiniboine River. It was also assumed that the communities of Starbuck and Sandford would require lineal dikes along the La Salle River for flows of 708 m^3/s (25,000 cfs), and a full ring dike system at flows of 1,303 m^3/s (46,000 cfs). The community of La Salle was assumed to require a ring dike at flows of 1,303 m^3/s (46,000 cfs). The total community dike length for the flow of 708 m^3/s (25,000 cfs) was 37 km (23 mi) and was 75 km (46.6 mi) for a flow of 1,303 m^3/s (46,000 cfs).

A cost estimate was prepared for community dikes based on an assumed rate of $1 million per lineal kilometer for community dikes at a flow of 708 m^3/s (25,000 cfs) and $1.5 million per lineal kilometer at a flow 1,303 m^3/s (46,000 cfs). For individual flood protection, a cost estimate was prepared based on the same assumptions that were made for the individual flood proofing option (2b) between Baie St. Paul and Headingley discussed in Section 10.1.7.5 with the exception that the average dike length, which was assumed to be 70% longer based on a review of aerial imagery in the area, due to the lower number of properties that could share a dike.

The major environmental mitigation efforts expected for this scheme due to the potential negative impact caused by widespread flooding have been further discussed in Section 10.3.2. The cost for individual and community flood protection at the flow of 708 m^3/s (25,000 cfs), including a 15% allowance for major environmental mitigation efforts, as well as the standard project allowances for indirect costs (engineering, approvals, administration and owner's costs, etc.) and contingencies, was estimated to range between $240 million and $305 million. At the flow of 1,303 m^3/s (46,000 cfs), the cost was estimated to range between $540 million and $620 million.

The cost for this scheme was also calculated for a flow of 654 m^3/s (23,100 cfs) to compare with other flood mitigation options presented in this report. Assuming that the cost is $0 at a flow 425 m^3/s (15,000 cfs) (which is equivalent to the existing reliable capacity of the Assiniboine River dikes as discussed in Section 10.1.5), and that the cost can be interpolated between the flow of 425 m^3/s (15,000 cfs) and 708 m^3/s (25,000 cfs), the cost for individual and community

flood protection at the flow of 654 m³/s (23,100 cfs) was estimated to range between $190 million and $250 million. With the added cost of flood mitigation between Baie St. Paul and Winnipeg estimated at $63 million for Option 2b (discussed in Section 10.1), the total cost of the scheme ranged between approximately $255 million and $310 million. The total cost at the flow of 654 m³/s (23,100 cfs) is summarized in Table 10.28.

Table 10.28
Summary of Cost for Individual and Community Flood Protection along the Lower Assiniboine River for a flow of 23,100 cfs

Individual and Community Flood Protection	Combination of Individual Flood Proofing, Neighbourhood Diking and Purchase of Vulnerable Properties (between Baie St. Paul and Winnipeg)	Total
$190 to $250 million	$63 million	$255 to $310 million

10.3.2 Other Considerations

There are several potential negative impacts, which have rendered these options unfeasible. These are summarized below:

- Existing infrastructure such as provincial and municipal roads would not be protected with this option. Widespread flooding would result in a significant number of road closures including potentially the TransCanada Highway, PTH 13, PTH 26, and even PTH 2 and PTH 3 under severe flood conditions. Railway transportation would also be affected, with the most notable being the Canadian National Rivers Subdivision, which is CN's mainline connecting Winnipeg to the west. As discussed in Section 8.6.1, disruption to the transportation network could result in damages exceeding $100 million. The cost of repairs to roads, culverts and other infrastructure would also be significant.
- The road closures and limited access would trigger evacuations of several communities and most individuals within the flooded area.
- Agricultural land, which is currently protected by the existing diking system, would now be flooded. Without upgrades to the Portage Diversion or the existing diking system, flooding would occur on average once every 55 years and could occur more frequently if the existing diking system was not maintained.
- There has been significant opposition from local stakeholders to the scheme of diverting water from the Assiniboine River to the La Salle River at Hoop and Holler. The scheme for Individual and Community Flood Protection would likely be opposed by a greater number of stakeholders since it would impact a greater number of individuals in the area.

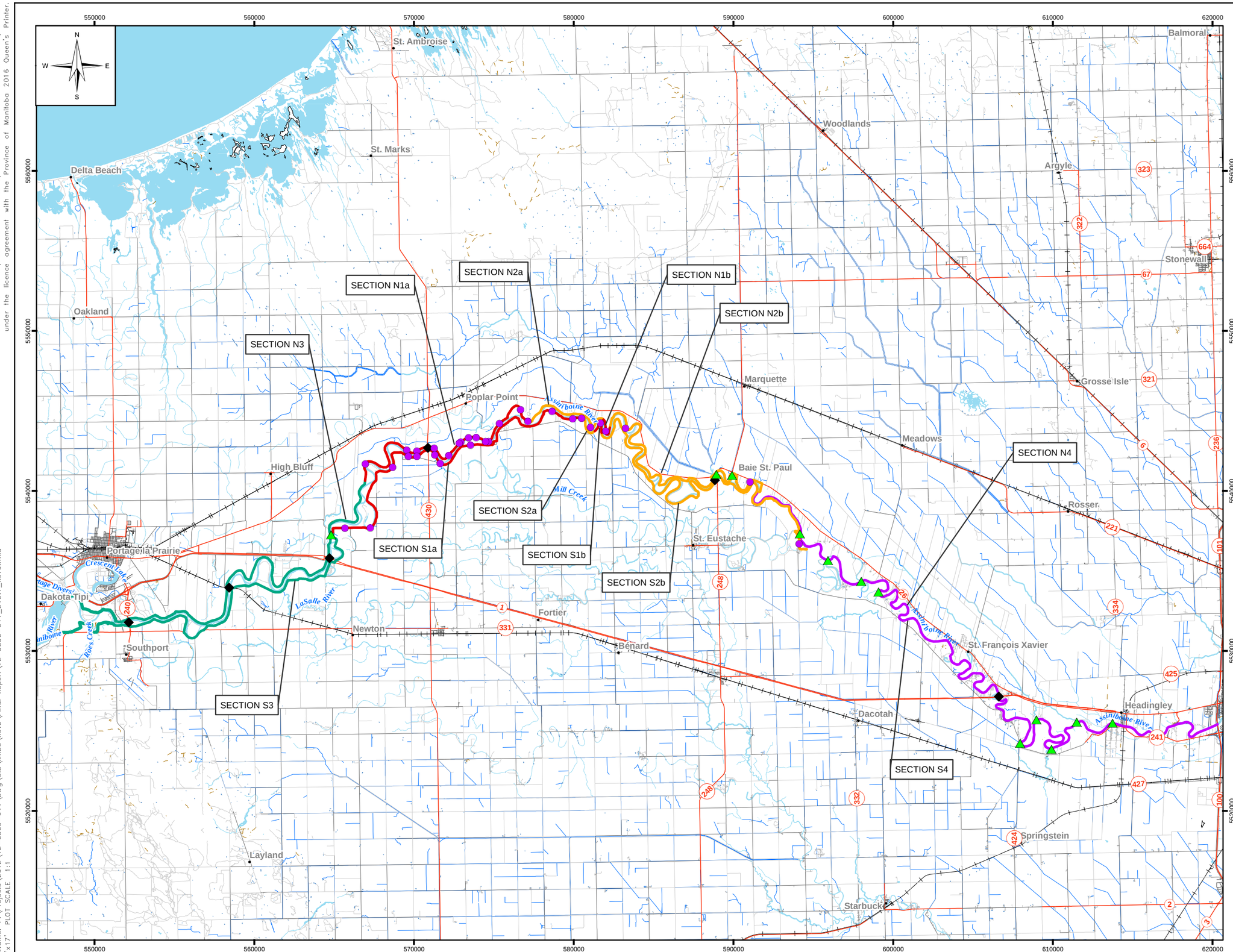
- Though several large floods have occurred on the Lower Assiniboine River in recent years, such as the floods of 2011 and 2014, the existing provincial Assiniboine Dikes as well as the Portage Diversion have significantly reduced the total area and the number of individuals impacted by those floods, as compared to what would have occurred without the infrastructure in place. The potential environmental and social impacts of allowing this area to flood again would be significant.

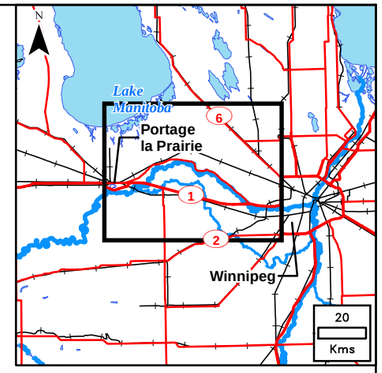
10.3.3 Conclusion

The total cost of the Individual and Community Flood Protection Scheme has been estimated to range between approximately $255 million and $310 million for a 654 m³/s (23,100 cfs) flow on the Lower Assiniboine River. Though this cost is comparable to other flood mitigation options discussed in this report, the Individual and Community Flood Protection Scheme was not identified as a feasible solution due to the potential significant negative impacts and added costs of damages resulting from the widespread flooding of the Lower Assiniboine River area and the La Salle River Basin.

DRAWINGS

Portions of data presented are owned by the Province of Manitoba and are produced under the licence agreement with the Province of Manitoba 2016 Queen's Printer.





LEGEND:

- Critical Dike Section (2011)
- Drainage Structures and Pumping
- Bridge Location
- Priority 1 Section of Existing Dikes
- Priority 2 Section of Existing Dikes
- Priority 3 Section of Existing Dikes
- Section Without Continuous Dikes
- Road Paved 2 or more lanes
- Paved Street or Road
- Gravel Road
- Unclassified Road
- Accessway or Backlane
- Trail
- Railway
- Dugout or Pond
- Lake
- Ditch
- River/Stream, Indefinite
- River/Stream

2 0 2 4 6 8 10
Kilometres
SCALE: 1:225,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
0	16/01/04	ISSUED WITH FINAL REPORT	PAL

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

DIKE SECTIONS OF THE LOWER ASSINIBOINE RIVER

JANUARY 2016 | DWG 10.1 | REV: 0

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\Rev0\Final Report\12-0300-011_D10.1_Rev0.mxd
11"x17" PLOT SCALE 1:1

11.0 PORTAGE DIVERSION

11.1 INTRODUCTION AND BACKGROUND

Construction of the Portage Diversion (PD) was completed in 1970 and is an important component of Manitoba's provincial flood control infrastructure. Its main purpose is to divert a portion of Assiniboine River flood flows at Portage la Prairie away from the City of Winnipeg and the Assiniboine River downstream of the PD (Lower Assiniboine River). Drawing 11.1 shows the location of the Portage Diversion and identifies the main components of its infrastructure.

The PD channel is approximately 29 km (18 mi) long and runs south to north from the Assiniboine River to Lake Manitoba. Flows entering the PD channel, as well as flows released downstream on the Assiniboine River, are regulated with a Diversion Control Structure located north of the PD reservoir on the Assiniboine River as well as with a River Control Structure located east of the PD reservoir. Other main components of the PD include dikes, two drop structures, an outlet structure to Lake Manitoba, and seven bridge crossings. The PD has a design capacity of 708 m^3/s (25,000 cfs).

The PD has been operated over 33 times in the 44 years since construction was completed in 1970. Most notably was the flood of 2011 on the Assiniboine River, when flows in the PD exceeded the design discharge capacity of 708 m^3/s (25,000 cfs) for a total of approximately 30 days. Furthermore, flows of approximately 963 m^3/s (34,000 cfs) were sustained for about one week in the PD; and the channel diverted more than three times the next highest recorded volume of water, which had previously occurred in 1976. The diversion of such high flows and volumes of water was necessary during the flood of 2011 to maintain peak flows on the Lower Assiniboine River at or below approximately 510 m^3/s (18,000 cfs). At the time, this flow was determined to be the maximum allowable capacity of the Lower Assiniboine River dikes downstream of Portage la Prairie.

More recently, similar conditions occurred again in the PD in 2014 due to a record rainfall event in the Assiniboine River Basin. The increase in discharge capacity of the PD during the 2011 and 2014 floods was achieved only with an exceptional effort to maintain the integrity of the water retaining and control structures and had a significant risk of failing. In response to the

flood, the Province of Manitoba has undertaken measures to repair and upgrade several components of the PD to improve the reliability of the PD channel up to a flow of 963 m^3/s (34,000 cfs). However, even with the upgrades, vulnerabilities and risk of failure exist on the PD when flows exceed its original design capacity.

Options to increase the discharge capacity of the PD were, therefore, considered for this study. The options were focused on design concepts that could be deployed without needing the exceptional efforts undertaken in 2011 and 2014, and without significant risk of failure. These are discussed within this section of the report.

In addition, a segment along the downstream end of the west embankment of the PD channel, called the "Failsafe", was originally designed to allow overtopping and spillage onto the adjacent land if diversion flows exceeded 425 m^3/s (15,000 cfs). Options to reduce or eliminate the damages due to overtopping of the Failsafe have been considered in the current study and are also discussed in this section of the report.

11.2 ELIMINATION OF FAILSAFE

11.2.1 Introduction

The original design of the PD channel included a segment along the downstream end of the west embankment of the channel that would allow overtopping and spillage into the adjacent land if diversion flows exceed 425 m^3/s (15,000 cfs). This zone was called the "Failsafe". The objective was to minimize the volume of material to be hauled to build the PD dikes. This would reduce construction costs. Hauling material during construction was required in the area located near and downstream of the Failsafe because the material adjacent to the PD was not suitable for the dikes. Since completion of the channel in 1969, the Failsafe has been overtopped on numerous occasions. Additionally, recent reports (Hatch, 2011b) suggest that the Failsafe could be overtopped when flows were as little as 340 m^3/s (12,000 cfs) due to a reduction in the hydraulic capacity of the channel compared to the dyke crest. The damages done by the overflow are considered unacceptable to local stakeholders. The location of the Failsafe is shown on Drawing 11.1.

Options to reduce or eliminate the damages due to overtopping of the Failsafe are being considered in the current study. Several options have been studied previously following the floods of 1974 and 1976. However, the results of the analyses are outdated and need to be revised to consider up to date costs and current hydrologic and hydraulic conditions. The following four options have been identified as potential mitigation options to reduce or eliminate the damages due the activation of the Failsafe:

1. **Raise the Portage Diversion Dikes** – Elimination of the Failsafe by raising the East and West dikes in the PD channel along the lower reach.
2. **Purchase of Vulnerable Property** – Buyout of property subject to flooding.
3. **Dike Agricultural Land** – Constructing dikes to protect agricultural land from flooding.
4. **Dike along Cram Creek** – Constructing dikes along Cram Creek to control PD overflows through the Failsafe.

Details on the four mitigation options, including cost estimates, are provided in the sections that follow.

11.2.2 Flooded Area Due to Overtopping of the Failsafe

The flooded area downstream of the Failsafe was determined for the following three historic floods on the PD:

1. 1974 – 510 m^3/s (18,000 cfs).
2. 1976 – 708 m^3/s (25,000 cfs).
3. 2011 – 983 m^3/s (34,700 cfs).

The flooded area for the 1974 and 1976 floods was based on data included in the June 1976 report titled, “Portage Diversion Proposed Modifications to Reach 1A” (Manitoba, 1976). The 2011 Flooded area was based on Radarsat Imagery collected on May 19, 2011 and attached in Appendix L. The flooded area was subdivided into the following three categories, based on landownership maps of the area:

1. **Private Land – Agricultural** – Private land used primarily for agricultural purposes.
2. **Private Land – Non-agricultural** – Private land that is not used for agricultural purposes. Mostly located within marsh area. Approximately 40% of this area is estimated to be covered by small lakes and streams.

3. **Crown Land** – Land owned by the crown. Includes a combination of agricultural and non-agricultural land, which may or may not be leased.

A summary of the flooded area vs discharge in the PD is provided on Table 11.1.

Table 11.1
Flooded Area due to Activation of the Failsafe

Portage Diversion Flows			Flooded Area (ha)			
Year	m^3/s	cfs	Private Land Agricultural	Private Land Non-agricultural	Crown Land	Total Land
1974	510	18,000	248	1620	320	2188
1976	708	25,000	555	1620	360	2535
2011	983	34,700	1170	1620	400	3190

Depending on the duration of a flood, the magnitude of flows through the Failsafe can increase with time due to erosion of the Failsafe section and deterioration of the Failsafe dike crest. However, a comparison of Radarsat Imagery collected on various days during and after the peak of the 2011 Flood did not show a significant change in flooded area downstream of the Failsafe. As well, 2014 MIT observed that the maximum flooded area was reached after several days. It was, therefore, assumed for this analysis that the duration of a larger flood is long enough that the flood duration would have negligible effects on the extent of the flooded area. Levels in the channel were higher in 2014 due to increased vegetation and implementation of a Carp exclusion structure.

The original design of the PD assumed that the Failsafe would be overtopped when flows exceeded 425 m^3/s (15,000 cfs). However, a February 2011 report by Hatch titled, “Portage Diversion Design Basis” (Hatch, 2011b) suggested that the Failsafe could be overtopped when flows were as little as 340 m^3/s (12,000 cfs). This appeared to be due to a reduction in the hydraulic capacity of the channel compared to the dyke crest, possibly caused by sediment accumulation in the lower PD channel. Furthermore, MIT has indicated that the presence of ice in the channel was also a possible cause for overtopping at flows below the original design capacity. This more frequent occurrence of activation of the Failsafe compared to the original design was considered in the assessment of the flood mitigation options (i.e., the Failsafe is currently being activated more frequently than the original design intent at less than 425 m^3/s

(15,000 cfs)). Additional information regarding the influence of ice jams in the Portage Diversion Reservoir and in the channel is documented in the Red River Floodway Operation Report (MIT, 2013). For example, the 2013 report documents the effects of ice jams in the reservoir and the resulting surges downstream of the River Control Structure.

Although there is normally ice in the channel, it depends on the circumstances of the break-up and nature of the inflow, whether or not it significantly affects the discharge capacity of the channel.

It should be noted that the Province of Manitoba has recently initiated maintenance activities in the channel to remove deposits of silt near the Failsafe area, which in turn would reduce the frequency at which the Failsafe is activated. The effects of the maintenance program to the mitigation options presented in this report have not been quantified since the maintenance program began after the analyses of the mitigation options were already completed. However, it is anticipated that its impact would not change the overall conclusions presented in this report.

11.2.3 Raising of the Portage Diversion Dikes

Reducing or eliminating the damages due to overtopping of the Failsafe can be achieved by raising the PD dikes in the lower reach to contain PD flows within the channel. Under current conditions, water spills out of the PD once the water level in the channel exceeds the top of the west embankment along the Failsafe area.

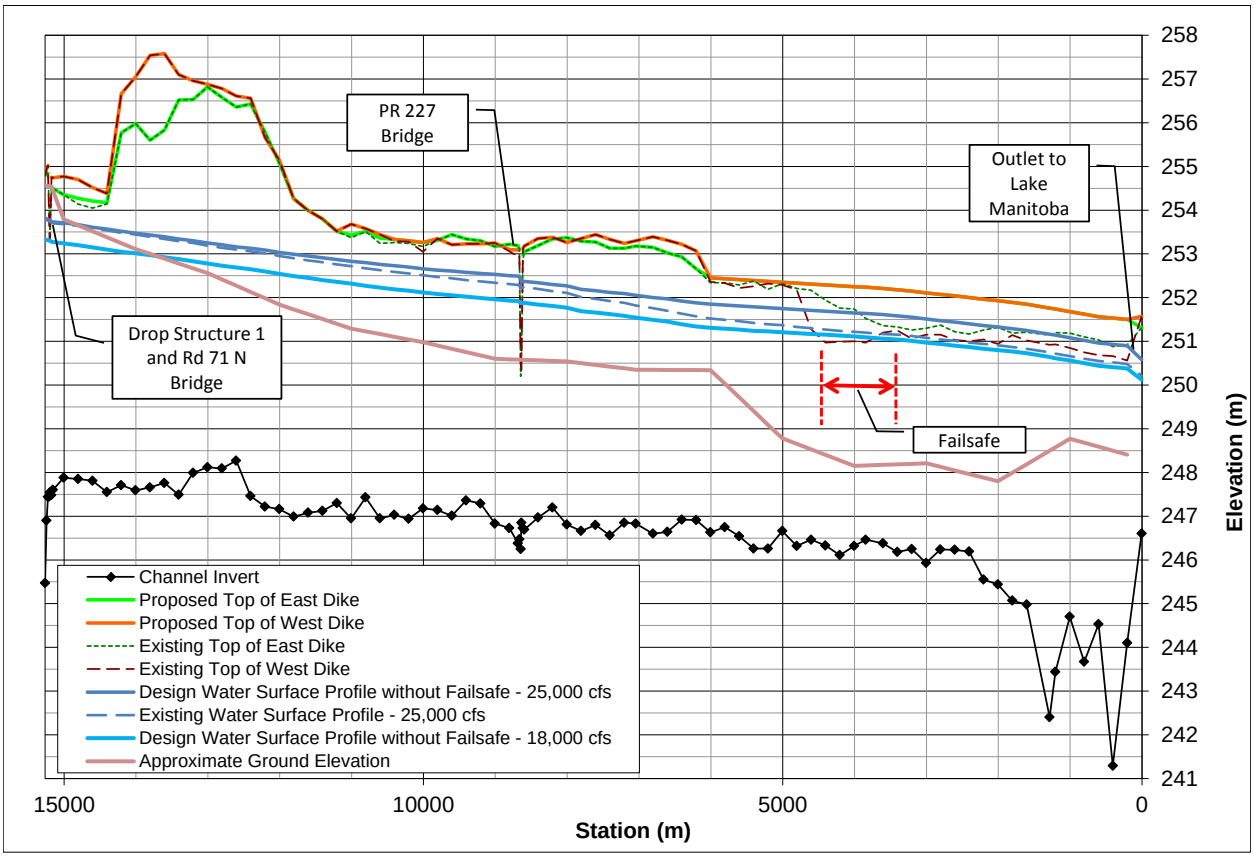
Two conceptual options to raise the PD dikes were investigated for this study:

1. Raising of the PD dikes to the maximum capacity of the channel of 708 m^3/s (25,000 cfs) to eliminate the Failsafe.
2. Raising of the PD dikes to 510 m^3/s (18,000 cfs) to reduce the frequency at which the Failsafe is overtopped.

The conceptual design to raise the PD dikes requires that the East and West dike be raised to contain the flows outlined above without overtopping of the Failsafe. The water surface profile in the channel, without the Failsafe, at the design flows plus 0.6 m (2 ft) of freeboard was set as the design elevation. The water surface profile for the proposed design condition was

determined with the HEC-RAS backwater model of the channel discussed in Section 8.3.4. The design water surface profile at 708 m^3/s (25,000 cfs) is provided on Figure 11.1. The proposed design elevation of the dikes compared to the existing configuration is also shown on the figure.

Figure 11.1
Portage Diversion Water Surface Profile With and Without Failsafe



Information on the existing top elevation of the dikes was taken from the HEC-RAS backwater model cross sections, which were surveyed prior to the 2011 Flood. It is recognised that the PD dikes were raised under emergency conditions at many locations during the 2011 and 2014 floods. However, since post flood profiles and cross sections were not available for the current study, the pre-2011 geometry formed the basis of the design. This has been considered the conservative approach for the purpose of cost estimating as it provides a conservatively high estimate of the total volume of fill required for construction.

The total volume of fill material required to raise the dikes was calculated by subtracting the volume of required dikes from the volume of the existing dikes along the channel. An average

top width of 5 m (16 ft) and side slopes of 6:1 were used to calculate volumes, based on a cursory review of the existing dike geometry in the area. It was assumed that the top width and side slopes of the existing dikes would also be adequate for the required dikes.

Land acquisition was not required for this alternative as sufficient right-of-way (ROW) already existed on either side of the channel to raise the dikes. Also, since the PD outlet structure was originally designed for 708 m^3/s (25,000 cfs), design modifications were not required at the PD outlet.

11.2.4 Purchase of Vulnerable Property

The purchase of the land inundated due to the activation of the Failsafe was considered as a means of reducing flood damages.

For this alternative, it was assumed that all of the private land affected during the 2011 Flood would be purchased. The 2011 flooded area was based on Radarsat Imagery collected on May 19, 2011 and attached in Appendix L. As previously indicated, a total of approximately 1,170 ha (2,891 ac) of private agricultural land and 1,620 ha (4,003 ac) of non-agricultural land were flooded in 2011. Since the flooded area is irregularly shaped and does not necessarily follow section or property lines, additional land would likely be purchased during the buyout process to minimize the division of land parcels or properties. It was estimated that an additional 25% of the private agricultural land area would have to be purchased as part of this alternative.

Land that would be purchased as part of this flood mitigation option could be leased back to local farmers or could be converted into a Wildlife Management Area. There would still be a maintenance cost for each Failsafe activation wherein debris would be strewn across the inundated farmlands. The end use of any land purchased would have to be determined by the various levels of government involved, with input from the various stakeholders. The definition of the end use of the purchased land was not part of the scope of this study. Since the Failsafe would continue to be activated during periods of high flows in the PD for this alternative, not all damages would be averted from the buyouts of the affected land. Certain types of damages such as repairs to the PD dikes at the Failsafe location and roadway repairs would continue to occur. Other environmental impacts, such as inundation of the adjacent wetland, erosion of the

Lake Manitoba shoreline and debris being strewn across the area would also continue to occur and would need to be evaluated.

11.2.5 Diking Agricultural Land

The concept of constructing dikes to protect agricultural land consists of controlling the spill out from the Failsafe by containing it within the marsh area. From a constructability point of view, this concept appears attractive, as the dikes could probably be constructed on dry land using adjacent material as borrow. This is compared to the mitigation alternatives of raising the PD dikes and diking Cram Creek where using adjacent material is likely not possible.

Since LiDAR or survey data was not available in the area where the dikes were proposed, the preliminary design of the dikes described in the 1976 report entitled, "*Portage Diversion Proposed Modifications to Reach 1A"* (Manitoba, 1976) formed the basis of this alternative. A schematic from the 1976 report showing the proposed dike alignment is provided in Appendix H.

Land purchases would be necessary for this option for the layout of the dikes, as well as for the properties not protected by the dikes. Most of this land is non-agricultural; however, based on the maps of the 1976 report (Manitoba, 1976), some agricultural land would also have to be purchased.

Similar to the option for purchasing the vulnerable properties, discussed in Section 11.2.4, diking the agricultural land would not stop the Failsafe from being activated during periods of high flows in the PD. Therefore, not all damages would be averted from diking the agricultural land. Certain types of damages such as repairs to the PD dikes at the Failsafe location and roadway repairs would continue to occur. Other environmental impacts, such as inundation of the adjacent wetland, erosion of the Lake Manitoba shoreline and debris being strewn across the area would also continue to occur and would need to be evaluated.

11.2.6 Diking Cram Creek

Cram Creek is a small watercourse on the west side of the PD that extends approximately 5 km (3 mi) from the Failsafe area to Lake Manitoba. Diking this creek was identified in the 1976

report (Manitoba, 1976) as a possible alternative to eliminate or reduce damages due to activation of the Failsafe. The location of Cram Creek is shown on Drawing 11.1.

Since LiDAR or survey data was not available in the area where the dikes are proposed, the preliminary design of the dikes described in the 1976 report (Manitoba, 1976) formed the basis of this alternative. Six different dike alternatives were identified in the report: two of them consisted of diking one side of the creek only, while the remaining four alternatives considered diking both sides of the creek. Since the exact dike alignments proposed for the different dike alternatives were not available with the report, it was assumed that the alignments followed a route parallel to the creek, from the southern edge of the Failsafe area to Lake Manitoba.

A non-erodible control structure was incorporated into the design to cap the outflow to 283 m³/s (10,000 cfs) when incoming flows into the PD are at a maximum capacity of 708 m^3/s (25,000 cfs). Although a similar non-erodible control structure concept has been tried in the past without success, the control structure concept proposed for the current study would be more robust and consist of an overflow weir protected by a cap of riprap, with a crest set to El 251.0 m (El 823.5 ft). This elevation was obtained from the survey data included in the backwater model of the PD discussed in Section 10.1.4.1 (pre-2014 calibration model Hatch, 2011). Since the average head above the weir was set to 0.24 m (0.8 ft) based on the existing water surface profile from the backwater model, the structure would have to be approximately 1.6 km (1 mi) wide to pass the desired flow of 283 m³/s (10,000 cfs). Assuming that riprap for the structure would be installed at a 0.6 m (2 ft) depth and extend a total of 25 m (82 ft) in length, the total volume of riprap required to construct the structure was estimated to be 25,000 m^3 (883,000 ft³).

For the alternatives where diking was proposed on one side only, it was assumed that land acquisition was required for the area not protected by the dikes. For the alternatives which had dikes on both sides of the creek, it was assumed that a 250 m (820 ft) wide area would have to be purchased; and that it would be sufficiently large to contain the maximum 283 m³/s (10,000 cfs) flow within the creek.

At this conceptual phase of study, only two of the six diking alternatives were carried forward for the cost analysis. These were:

1. The lowest cost alternative based on an alignment where diking was proposed on one side only.
2. The higher cost alternative based on an alignment where diking was proposed on both sides of the creek.

Several issues pertaining to constructability exist with this alternative. These are mainly due to the dikes being located within a marsh area, and include the potential requirements for dewatering, hauling of borrow material, as well as construction of a proper foundation for the dikes. These issues could significantly affect the level of effort and cost required to build the dikes. Additionally, having the dikes constructed and located within the marsh area could decrease the reliability of the scheme in the future. Additionally, Diking of Cram Creek would isolate about 30% of the west marsh area and may not be acceptable to Manitoba Conservation and Water Stewardship. These concerns would have to be considered carefully if these options were selected to proceed to a more detailed phase of study.

11.2.7 Cost Estimates

Cost estimation was preliminary at this screening stage of analysis. The estimated quantities and equivalent costs for the various alternatives have been summarized in Tables 11.2 to 11.7 and are based on the unit rates discussed in Section 7.8 of the report.

When calculating quantities for non-agricultural land, it was estimated that approximately 40% of the area was covered by small lakes and streams and, therefore, this area was not included in the cost estimate. For the two alternatives of diking Cram Creek, an additional allowance of $\$20/m^3$ of fill material was incorporated into the cost estimate to account for complexities of having to construct dikes within a wetland complex.

Environmental mitigation costs were also incorporated into the analysis. These were assumed to be relatively minor for all options, with the exception of diking Cram Creek where they were assumed to be minor due to the potential impacts of locating the dikes within the wetlands. No allowance for environmental mitigation has been considered in these estimates, with exception of the option of diking Cram Creek.

Table 11.2
Estimated Capital Costs for Raising of the Portage Diversion Dikes for 708 m^3/s

Item	Description	Cost (where pertinent)
Volume of Fill Material for Dikes	350,000 m^3 @ \$20/m^3	\$ 7,000,000
Revegetation	4.83 ha @ \$5,500/ha	\$ 27,000
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	20% of direct costs	\$ 1,405,000
Contingency	30% of direct and indirect costs	\$ 2,529,000
	Total Cost	**\$ 10,961,000**

Table 11.3
Estimated Capital Costs for Raising of the Portage Diversion Dikes for 510 m^3/s

Item	Description	Cost (where pertinent)
Volume of Fill Material for Dikes	115,000 m^3 @ \$20/m^3	\$ 2,300,000
Revegetation	1.63 ha @ \$5,500/ha	\$ 9,000
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	20% of direct costs	\$ 462,000
Contingency	30% of direct and indirect costs	\$ 831,000
	Total Cost	**\$ 3,602,000**

Table 11.4
Estimated Capital Costs for the Purchase of Vulnerable Property

Item	Description	Cost (where pertinent)
Land Acquisition of Private Agricultural Land	1,460 ha @ \$15,000/ha	\$ 21,900,000
Land Acquisition of Private Non-Agricultural Land	1,620 ha @ \$1,000/ha	\$ 1,620,000
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	20% of direct costs	\$ 4,704,000
Contingency	30% of direct and indirect costs	\$ 8,467,000
	Total Cost	**\$ 36,691,000**

Table 11.5
Estimated Capital Costs for Diking Agricultural Land

Item	Description	Cost (where pertinent)
Volume of Fill Material for Dikes (approximately 10.5 km long)	147,000 m^3 @ \$20/m^3	$ 2,940,000
Revegetation	2.43 ha @ \$5,500/ha	$ 13,000
Land Acquisition of Private Agricultural Land	190 ha @ \$15,000/ha	$ 2,850,000
Land Acquisition of Private Non-Agricultural Land	1620 ha @ \$1,000/ha	$ 1,620,000
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	20% of direct costs	$ 1,485,000
Contingency	30% of direct and indirect costs	$ 2,672,000
	Total Cost	**$ 11,580,000**

Table 11.6
Estimated Capital Costs for Diking Cram Creek – Low Cost Alternative

Item	Description	Cost (where pertinent)
Volume of Fill Material (approximately 5 km long)	43,000 m^3 @ \$20/m^3	$ 860,000
Allowance for Miscellaneous Construction Complexities	43,000 m^3 @ \$20/m^3	$ 860,000
Revegetation	0.7 ha @ \$5,500/ha	$4,000
Land Acquisition of Private Non-Agricultural Land	320 ha @ 1,000/ha	$ 320,000
Volume of Riprap for Non-Erodible Failsafe Control Structure	25,000 m^3 @ \$90/m^3	$ 2,250,000
Environmental Mitigation Costs	Minor – 5% of direct costs	$ 215,000
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	20% of direct costs	$ 859,000
Contingency	30% of direct and indirect costs	$ 1,610,000
	Total Cost	**$ 6,978,000**

Table 11.7
Estimated Capital Costs for Diking Cram Creek – High Cost Alternative

Item	Description	Cost (where pertinent)
Volume of Fill Material (approximately 9 km long)	180,000 m^3 @ \$20/m^3	\$ 3,600,000
Allowance for miscellaneous construction complexities	180,000 m^3 @ \$20/m^3	\$ 3,600,000
Revegetation	2.97 ha @ \$5,500/ha	\$ 16,000
Land Acquisition of Private Non-Agricultural Land	100 ha @ 1,000/ha	\$ 100,000
Volume of Riprap for Non-Erodible Failsafe Control Structure	25,000 m^3 @ \$90/m^3	\$ 625,000
Environmental Mitigation Costs	Minor – 5% of direct costs	\$ 478,000
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	20% of direct costs	\$ 1,913,000
Contingency	30% of direct and indirect costs	\$ 3,587,000
	Total Cost	**\$ 15,545,000**

11.2.8 Conclusion

Raising of the PD dikes to the maximum capacity is preferred over other options as it is the only option that would fully eliminate the Failsafe and all damages caused by its activation. Raising dikes on the PD also has the benefit of impacting the least number of stakeholders when compared to the remaining options as land acquisitions would not be necessary and has the lowest potential for environmental impacts. Additionally, the alternate options of Diking Agricultural Land, the Purchase of Vulnerable Property, and Diking Cram Creek are more expensive and, therefore, would be less economically attractive.

Subsequent sections of this report will reveal that expansion of the Portage Diversion is a leading option for reducing flood potential in the region. Regardless of what form that expansion would involve, it would be logical to incorporate the elimination of the Failsafe within it. Due to the relatively low cost of eliminating the Failsafe compared to the overall cost of increasing the Portage Diversion capacity, as well as due to the recurring economic, environmental and social impacts of flooding in recent years from activation of the Failsafe, it has been concluded that any upgrades to the Portage Diversion should inherently eliminate the Failsafe.

11.3 OPTIONS TO INCREASE THE DISCHARGE CAPACITY OF THE PORTAGE DIVERSION

Three concepts have been considered to increase the capacity of the Portage Diversion for this study and are discussed in this section:

1. Widening of Existing PD Channel.
2. Construction of a Parallel Channel West of the Existing PD Channel.
3. Carrying Out Upgrades to the Existing PD ("Retrofit Option").

In addition to these three alternatives, the "Do Nothing" alternative has also been considered. This alternative considers the need to upgrade aging infrastructure regardless of whether or not the capacity is increased. It is required as a frame of reference for any economic analysis.

Each of the options was developed to meet the design criterion of 963 m^3/s (34,000 cfs). This would be the safe design capacity; however, each option would have an emergency capacity greater than 963 m^3/s (34,000 cfs), possibly 142 m^3/s to 283 m^3/s (5,000 cfs to 10,000 cfs), or more depending on the option. To achieve the emergency capacity, significant efforts would likely be required to maintain the integrity of the water retaining and control structures and minimize the risk of failure. This would be similar in concept to the emergency capacity of the existing channel in 2011 and 2014 of approximately 963 m^3/s (34,000 cfs), which was considerably greater than the design capacity of 708 m^3/s (25,000 cfs). The emergency capacity of the channel would be assessed formerly at the next stage of design and could possibly be different for each of the options under consideration. For example, the Channel Widening and Parallel Channel options would likely have more emergency capacity than the Retrofit option due to the already higher water level in the channel and in the reservoir for the Retrofit option under the 963 m^3/s (34,000 cfs) design condition, as discussed in Section 11.3.5. At this conceptual stage of design, further more detailed analyses on the emergency capacity of the options were outside the scope of the study and were not considered.

It should be noted that an option for a larger Portage Diversion was also considered for the study with a design capacity of 1,303 m^3/s (46,000 cfs). This option is discussed in Section 11.4.4 and was based on the same design criteria as those outlined for the 963 m^3/s (34,000 cfs) capacity options.

11.3.1 Dam Safety Considerations

The Portage Diversion Reservoir during flood conditions is a body of water that could cause serious damages if released suddenly through a breach in the retaining structures. The Canadian Dam Association (CDA) has issued widely accepted guidelines (CSA, 2009) for assessing acceptable risks at such structures. It is understood that a dam safety review of the Portage Diversion system (i.e., the Forebay River Control Structure, the Diversion Control Structure, the Channel and the Dikes) has not been carried out to ascertain the extent to which this project complies with the Dam Safety Guidelines. As a result, it is difficult to speculate what changes to the project would be acceptable and which could violate the design norms currently used in Canada.

It is possible that the existing system would not meet the intent of the Dam Safety Guidelines and that modifications would need to be made for the structures to be compliant with the guidelines. Prior to proceeding with any upgrade option an assessment of the Dam Safety Requirements and any deficiencies that may exist should be considered. This type of assessment was beyond the scope of this concept study, but it should be completed before committing to a final upgrade decision. A similar process was undertaken prior to finalizing the scope of work for the Floodway Expansion Project. A number of dam safety deficiencies identified at that time were addressed during the functional design stage.

The so-called Retrofit Option, further discussed in Section 11.3.5, proposes to pass more flow into the diversion than the original design capacity of 708 m^3/s (25,000 cfs). This would require the reservoir level to be raised above the original design level, just as it was during the peak of the 2011 and 2014 floods. For permanent adoption of this strategy, this would require an increase of the crest elevations of the dams and dikes retaining the reservoir by an equal amount (or more if current guidelines require it). This in turn would directly affect the potential risk that this structure imposes on downstream interests. The effect of a breach in this area is uncertain and will need to be assessed as part of the functional design stage of the project.

11.3.2 Existing Design Criteria and Conditions

Design criteria for the three options considered were based on the information provided in the Hatch reports (Hatch, 2011a; Hatch, 2011b) and the current assessments of the expansion requirements.

Options 1 and 2 utilized the original design criteria as the basis of design for the concept development. Information contained in Hatch's infrastructure report (2011a), additional calculations for the current study and the experience of KGS Group engineers during the 2011 Flood were used to develop the concept for Option 3 – the Retrofit Option.

11.3.2.1 Original Basis of Design

For this concept design, the original basis of design was adopted for the expansion alternatives. Review of the criteria and optimization will be considered at subsequent stages of design.

Options 1 and 2 utilized the original design criteria as the basis for concept development.

The Hatch Basis of Design Report (2011b) states that:

1. "The design Diversion Control Structure capacity at the Full Supply Level of El 264.87 m (El 869 ft) is 708 m^3/s (25,000 cfs), with the four (4) gates in their fully open positions. A maximum Assiniboine River flow of 1,274 m^3/s (45,000 cfs) can be accommodated by the River Control Structure and the Diversion Control Structure combined, without causing flooding on the Assiniboine River downstream of the River Control Structure. This maximum design flood flow was estimated to be approximately a 1:300 year flood in magnitude at the time of design. At the design flood level, 708 m^3/s (25,000 cfs) is diverted while the remaining 566 m^3/s (20,000 cfs) passes downstream into the Assiniboine River".

2. "During passage of extreme flood flow conditions, with the reservoir surcharged to El 266.40 m (El 874 ft), a maximum Assiniboine River inflow of 2,549 m^3/s (90,000 cfs) can be accommodated by the combined capacity of the River Control Structure and the Diversion Control Structure. In this case, 1,274 m^3/s (45,000 cfs) is diverted through the Diversion Control Structure and into Lake Manitoba while the remaining 1,274 m^3/s (45,000 cfs) passes through the River Control Structure into the Assiniboine River. This is only the capacity of the structures themselves, as flows of this magnitude would overtop the downstream dykes and flood the downstream reaches".

   "Under open water conditions, the Diversion Channel has the capacity to pass the design flow of 708 m^3/s (25,000 cfs) without overtopping the dykes in the reach from the

> inlet to Drop Structure 1. Downstream of Drop Structure 1, the channel was reduced in capacity to 424.8 m^3/s (15,000 cfs) for economic reasons".

Furthermore, the design capacity of the drop structures and the outlet structure is reported to be 708 m^3/s 25,000 cfs.

11.3.2.2 Infrastructure Assessment Report

The infrastructure report and KGS Group's experience during the 2011 Flood, as well as criteria from other sources, were used to develop most of the criteria for Option 3, the Retrofit Option. Further review of these criteria would be conducted in combination with MIT at the next stage of the design.

The Hatch Infrastructure Assessment Report (2011b) identified the maximum diversion channel capacity, excluding the Failsafe Area, as a discharge of 807 m^3/s (28,500 cfs) that could be conveyed without overtopping the lowest portion of the dikes between the Diversion Control Structure and the Outlet Structure. At this discharge, the low chords of all the bridges were seen to be clear of the modeled water surface for the scenario, with the exception of the former PR 249 and PR 227 bridges. The low chord of the PR 227 Bridge would just begin to be submerged at this flow. Clearance at the other bridges varied between 0.17 m and 0.8 m (0.6 ft and 2.6 ft). The PR 227 Bridge was then replaced after the 2011 Flood.

At a discharge of 807 m^3/s (28,500 cfs), modeled average velocities through the bridges did not exceed 1.7 m/s (5.5 ft/s), with the exception of the CPR Carberry and the former PR 249 bridges. These two bridges are located approximately 100 m (328 ft) downstream of Drop Structure 2 and Drop Structure 1, respectively. Since the flow downstream of the existing structures will not be able to fully expand prior to reaching the bridges, velocities through these bridges would be higher and would be estimated at 2.6 m/s (8.5 ft/s) and 3.1 m/s (10.2 ft/s), for the CPR Carberry and former PR 249 bridges, respectively.

Based on the Hatch report, Drop Structure 1 is expected to have 0.7 m (2.3 ft) of freeboard remaining against the upstream wing walls and a submergence of 0.1 m (0.3 ft) on the downstream basin walls at a discharge of 807 m^3/s (28,500 cfs). Drop Structure 2 is expected to have 0.4 m (1.3 ft) of freeboard remaining against the upstream wing walls and 0.1 m (0.3 ft) of

freeboard above the downstream basin walls. At this flow, the stability of the structures is not expected to be compromised. Tailwater levels should be sufficient to prevent jump sweep out at the Diversion Control Structure and both drop structures for this flow. The Outlet Structure was apparently designed to accommodate jump sweep out and thus is not a concern from a hydraulic performance perspective.

The infrastructure report did not explicitly address the capacity of the drop structures, however, it does imply that these structures should be adequate at a discharge of 807 m^3/s (28,500 cfs). This would need to be confirmed, as well as the ability of the channel to resist erosion at velocities exceeding the original design criteria. Similarly, the adequacy of the dikes would need to be confirmed for levels reported in the Hatch report at this discharge.

Although Hatch has concluded that a discharge of 807 m^3/s (28,500 cfs) could be passed at a level of imminent overtopping, there are a number of channel elements that would need to be verified to ensure they could meet accepted current design criteria at this discharge.

11.3.2.3 Groundwater Salinity Considerations

Farmers in the vicinity of the Portage la Prairie Diversion Channel to Lake Manitoba have raised concerns that some salinity levels in the soil increased in association with higher water levels during the use of the diversion channel and with higher water levels in Lake Manitoba. There are several possible contributors to this increased salinity including:

- Assiniboine River water diversion seeping from the channel into the adjacent aquifers and farm land.
- Very high levels of dissolved salts, primarily within the clay, till and bedrock aquifers, migrating into the overlying sand and gravel aquifers.
- Water level in the channel transmitting as pressure to increase confined saline aquifer groundwater levels in the vicinity.
- Wet periods (snow melt; precipitation) increasing infiltration to the unsaturated zone and dissolving salts within the soil.
- Lake Manitoba water level increases resulting in increased water levels within confined saline aquifers.
- Irrigation practices over time contributing to increased salt levels within the unsaturated zone and water table aquifer.

An overview discussion of these possible salinity contributions is given below. A more rigorous assessment of these potential groundwater impacts is beyond the scope of this study and should be addressed in subsequent design phases.

Soil Strata – In this area, the upper soils deposited between Portage la Prairie and Lake Manitoba are described mainly as low permeability silts and clays, with minor deposits of sands and gravels and silty sands. These stratified deposits were water laiden and vary in lateral continuity, including isolated, discontinuous pockets of sand. The groundwater regime can include upper unconfined aquifers within shallow pervious sands and gravels, which are perched above the silts and clays, and lower confined sand aquifers at depths capped by the silt and clay deposits and underlain by clay till and limestone bedrock. In general, the groundwater table can vary from close to surface to up to several metres in depth, with the overall lateral hydraulic gradient towards Lake Manitoba. The groundwater quality, based on historical monitoring reported by Manitoba Water Resources (Betcher, 1989) indicates considerable variability within the upper 3 to 8 m (10 ft to 26 ft) of overburden (sands and gravels to silts). Total dissolved solids (TDS) ranged from 282 to 2,070 mg/L, and chloride ranged from 15 to 330 mg/L. The clays, tills and underlying limestone bedrock artesian aquifer had very high TDS values of 2,000 to 20,000 mg/L, primarily reflecting the presence of high sulfate levels. In a flood condition, Assiniboine River water quality diverted through the channel is influenced by snow melt and rain dilution effects. For example, during the 2011 Portage Diversion operation April through August 2011 total dissolved solids ranged from 450 to 650 mg/L, chlorides ranged from 15 to 20 mg/L and sulphate ranged from 150 to 225 mg/L. All three parameters were lowest at the beginning of the operation, increasing to the high end of the range in July and August. Local high salinity measured within the surficial sand and gravel aquifers may have resulted from the upward flow of high dissolved solids within the till and bedrock, and possibly buildup related to irrigation practices.

Seepage – When the diversion channel is flowing full, the water surface in the channel could vary up to 1 to 6 m (3 ft to 20 ft) above the ground surface, adjacent to the channel dikes. Direct seepage from the channel could discharge into the surrounding alluvial soils, either via the excavated cut (5 to 6 m deep / 16 ft to 20 ft) into the subsurface strata or through the containment dikes above ground surface, with the former considered the primary pathway. The groundwater table adjacent to the channel would likely be close to ground surface under the wet

conditions (snow melt, precipitation) that necessitate use of the channel. Seepage from the channel would be related to the differential height of the water level in the channel versus the adjacent groundwater table, and the permeability and continuity of the sand and gravel layers or silt and clay zones. This seepage to the groundwater would be localized, influencing conditions a relatively short distance from the channel (possibly 10 to 100 m / 30 ft to 300 ft), and also would depend on the duration of the diversion flow.

Exceptions to this can occur during summer rainfall events originating west of Portage la Prairie. In these cases, groundwater levels in the vicinity of the diversion can be low when the diversion is operated. This scenario would also need to be assessed in future studies.

Groundwater Level Influence – During diversion flow, an average water head surcharge along the channel in the order of 1 m to 6 m (3 ft to 20 ft) could be imposed, coupled with an interconnection to confined aquifer zones. This head increase would dissipate with distance from the channel, likely within a few hundred metres. The incremental increase in groundwater level pressure could force more saline groundwater up to the root uptake zone of crops planted. Rising groundwater levels, also due to the wetter conditions and increased recharge from snow melt and precipitation, could result in more saline groundwater migrating upward, and possibly dissolution of salts that had previously been deposited within the upper unsaturated soil zone.

A more thorough assessment is warranted at subsequent study phases to better delineate the mechanisms influencing salinity impacts on crops in the vicinity of the channel.

11.3.3 Option 1 – Widening of Portage Diversion Channel

Increasing the PD capacity can be achieved by widening the existing channel and increasing the size of the existing bridges and control and drop structures. The concept of the widening option was based on the existing configuration and hydraulic performance of the current PD channel and structures, expanded to accommodate the proposed design flow of 963 m^3/s (34,000 cfs).

11.3.3.1 Diversion Control Structure

The original design of this structure included four 11.58 m (38 ft) wide bays, each with a crest elevation of 260.60 m (855 ft). This discharge per unit width of 15.3 m^3/s / m (~164 cfs / ft) was maintained for the expanded channel by increasing the number of bays to match the required design capacity. It was assumed for this study that new bays will have the same dimensions and crest elevations as the existing bays and that construction of the expanded structure could be achieved without undue difficulties in control of the Assiniboine River during non-flood conditions.

11.3.3.2 Control of Spring Ice Runs

The plan in the 1960s during the original design stage of the existing Portage Diversion was to maintain control of the reservoir at El 264.87 m (El 869 ft) during all flood events, including during spring ice runs. That was found after the project went into operation to result in uncontrollable surges in reservoir level as ice jams were released from the river upstream and entered the reservoir. Rises in reservoir water level often could not be controlled, and overtopping of the dikes, as well as the River Control and Diversion Control Structures during some spring ice runs was a real concern. In recent decades, the reservoir has been held at a lower level (at approximately El 263.65 (El 865 ft)) during the spring ice breakup period, so as to provide a buffer zone in the reservoir storage to absorb the often massive influxes of ice and water (Acres, 1984). It was assumed for the purposes of this study that this same practice would be followed. Only after the ice has dissipated on the river upstream would the reservoir level be raised to approximately the design level of El 264.87 m (El 869 ft).

11.3.3.3 Ice Boom Control

The original design of the Diversion Control Structure included an ice boom intended to restrain and deflect incoming ice. The boom was expected to prevent the ice from advancing to and through the Diversion Control Structure into the Diversion Channel. That ice boom was constructed of timber pontoons and had a limited capacity to restrain ice. It has been unsuccessful many times in restraining the movement of ice and has failed completely in some severe ice run events. The outcome has been the passage of significant volumes of ice through

the Diversion Control Structure (with significant risk of damage to the concrete structure or the control gates) and accumulation as a stationary ice jam at the outlet area of the Portage Diversion. This has caused rises in the water level and excessive flooding through the Failsafe zone. The current view of the operators now is that the release of the ice into the Diversion Channel is undesirable, but the options to prevent that are limited. There is a fear of deflecting the ice into the reservoir because that could result in filling the reservoir with ice, and or releasing ice through the River Control Structure. Downstream release could contribute to jamming and flooding along the Lower Assiniboine River. The inability of the ice boom to perform its originally intended function has been exacerbated by the increased velocities of flow at the low water levels needed to permit safe control of the surges associated with the ice runs, as previously described in Section 11.3.3.2.

Continued frequent release of ice into the Diversion Channel is undesirable and fraught with uncertainty. For example, the Diversion Control Structure and its gates were never designed for passage of large volumes of ice. Although severe damages have fortunately not occurred to date, the potential for that to happen in the future is real and significant.

At this conceptual phase of study, it is impossible to identify a fully satisfactory solution to this complex problem. A detailed hydraulic study of the issue of ice control is required to evaluate the options. One of the leading options for consideration in such a study would be replacement of the existing timber pontoons with more modern, robust steel pontoons filled with durable flotation and a suitable cable and anchor system. This concept is tentatively proposed, along with a detailed hydraulic study of the entire issue of ice control. This new type of design would be capable of restraining ice more effectively. Similar upgrades have been made to many large ice booms, including the one at the Jenpeg Generating Station (Manitoba Hydro), as well as the 2700 m (8,900 ft) long Niagara River and Lake Erie ice boom.

Conceptual analyses and a site visit by one of the leading designers of ice booms in the world (“Geniglace” from Ottawa) have indicated that a new modern boom would have a load bearing capability of at least approximately four times that of the current boom. Clearly that would assist in minimizing ice from entering the Diversion Channel. It is unknown, however, without further study, whether that enhancement would be adequate to significantly improve the performance in reducing the volume of ice that could enter the Diversion Channel. It was also noted by

Geniglace that the boom could be designed to permit a limited release of ice when the forces on the boom reach the peak. After the limited release of ice, and a reduction in the load on the boom, the boom would then re-establish control. The objective would ultimately be a significant reduction in ice volumes passing into the Diversion Channel as compared to the current situation.

It is clear that such a robust boom would increase the tendency for broken ice to reach and pass over the bascule gates in the River Control Structure. If that is proven to be a by-product of the enhanced ice restraint upstream of the Diversion Control Structure, it could be counteracted by installation of a robust modern boom across the reservoir at a strategic location that would restrain it from moving through the River Control Structure. Preliminary discussions with Geniglace indicate that this would likely be entirely feasible (pending further study and analysis to verify). Both booms could also be developed so that a limited passage of ice could be permitted and tolerated.

It is also possible that if the existing boom was fortified and a new boom installed in the reservoir, the large volume of accumulated ice could potentially fill the surface of the reservoir. This could cause an ice jam to progress upstream into the upper end of the reservoir or even into the river upstream. At this point it is not clear without detailed analyses of the extent of potential ice jams whether this would cause a serious problem upstream, or any issues at all. Clearly, detailed analyses that are beyond the scope of the current study are needed to evaluate the options. These studies would have to recognize the on-going effects of sediment deposition in the upper reaches of the reservoir.

There are other options that should also be considered in such a detailed future study of ice control. For example, if enhancement of the booms at the reservoir were shown not to be a fully satisfactory solution, the installation of a river ice control structure in the upper reach of the reservoir could be considered. This concept would involve the ice being jammed intentionally at this location and held to melt in place. The structure could possibly be a simple series of robust piers or concrete blocks in the river that would cause an ice jam to form. This concept has been developed and tested by the US Army Corps of Engineers in the northeastern US. Whether it could function acceptably in the Assiniboine River would of course require further study, along with the potential for flooding that it could cause upstream. An alternative to the piers might be

an overflow weir that would cause an increase in water level and a resulting enhancement in ice restraint. A structure of this type has been in operation for many decades on the St. Lawrence River at Montreal.

The proposed study of ice control should also include a “do-nothing” option, which would consist of continuing with the current operating strategy and equipment.

As outlined above, discussions related to operation and ice management practices have identified a number of complex issues related to the channel and river. These are beyond the scope of this study and should be assessed prior to moving forward with the refurbishment. A number of issues clearly need to be addressed related to the on-going problem.

11.3.3.4 Portage Diversion Channel and Drop Structures

It was assumed for this option that the channel geometry required to safely pass the design flood event must be achieved by widening the channel equally on either side of the PD. Future studies may address economies and difficulties that would come with deepening and/or widening the channel on one side only; however, at this conceptual phase of study, only widening equally on both sides was considered.

It was also assumed at this conceptual phase of design that the existing Failsafe Concept at the downstream end of the PD would be eliminated. Flood mitigation alternatives to reduce or eliminate damages due to overtopping of the Failsafe have been considered for this study and are documented in Section 11.2.

The following list addresses other key assumptions and design parameters applied for the option of widening the PD:

- The flow shear stresses applied to the surface of the channel must be kept within acceptable levels to minimize the risk of erosion under the design flow conditions. Shear stresses should not exceed those of the existing channel at its design discharge of 708 m^3/s (25,000 cfs).
- The channel should be non-aggrading, such that potential sediment deposits occurring during future use do not result in any detrimental impact to the discharge capacity of the channel. Small amounts of deposition that may require periodic minor maintenance

could be acceptable, although are less preferred. This will be a challenge and will require a coordinated maintenance effort to maintain vegetation growth in the channel.

- The final geometry of the expanded PD Channel should be such that it will promote surface runoff and minimize local ponding. Final surface grades should allow for any potential or reasonable changes that could affect runoff (e.g., surface settlements). Based on experience with the Red River Floodway Project, 50:1 lateral slopes were assumed to be necessary to achieve this criterion.
- The channel hydraulic analyses must incorporate appropriate energy losses due to channel roughness, geometry changes (i.e., contraction / expansion) and structures such as bridges.
- Changes in channel geometry should incorporate adequate transition zones to minimize hydraulic losses.
- The design flood profile of flow in the expanded channel should not exceed that of the existing channel at its design magnitude of 708 m^3/s (25,000 cfs).
- The design discharge of the Low Flow Channel should be adequate to pass the local runoff that enters the PD Channel and minimize the risk of local inflows overtopping the Low Flow Channel. The geometry of the existing Low Flow Channel has evolved from its original design state primarily due to erosion. Erosion issues have been aggravated by water ponding upstream of the Drop Structure, which kills vegetation. At the design phase, careful consideration for a stable design will be required to ensure the long term performance of the Low Flow Channel. This may result in the need for some form of erosion protection to achieve this objective. As well, the need to consider irrigation withdrawals and access will need to be addressed. The additional use of the Portage Diversion to convey irrigation water during summer conditions also needs to be considered in establishing the revegetation plan needed for the expansion.
- The Drop Structures and the Outlet Structure must be expanded in proportion to the width increases in the channel and in proportion to the increase in discharge capacity of the channel. Energy dissipation of the stilling basin for each structure should be possible with a submergence of the hydraulic jump that is comparable to the original design objectives of the existing stilling basin.

The offset distance between the top of the expanded Diversion Channel and the base of any transmission towers should allow for maintenance vehicle access. A minimum offset distance of 10 m (33 ft) is proposed for the purposes of this study. Existing transmission towers not complying with this offset criterion would be deemed to require reconstruction. Based on a conceptual review of Manitoba Hydro's existing transmission network, three major transmission lines currently cross the Portage Diversion. On this basis, it is probable that one or more transmission towers will have to be reconstructed to comply with the 10 m (33 ft) offset. However, future studies should identify whether any economies can be achieved by optimizing the channel geometry to minimize disturbance to the existing towers. At this conceptual phase

of analysis, reconstruction costs of transmission towers was estimated to be relatively low and, therefore, covered under the contingency cost allowance.

Assiniboine River Control Structure

The River Control Structure was assumed to require replacement for all options, as further described in Section 11.4.1. Any rehabilitation necessary for the structure, such as that suggested in the Acres Report (Acres, 1997), had not been carried out at the time of this report. The most important of those issues is the flood-proofing of the structure as well as its electrical, mechanical and instrumentation equipment to deal with higher tailwater levels (particularly during passage of ice in spring) and/or higher than anticipated flood Forebay than were envisaged in the original design.

The River Control Structure is currently at or near the end of its useful life and the assumption to replace the structure was based on an assessment of the age and condition of the structure as well as feedback from MIT. At the next stage of design, MIT may consider some level of rehabilitation of the structure or a major rehabilitation to extend its life by 50 plus years.

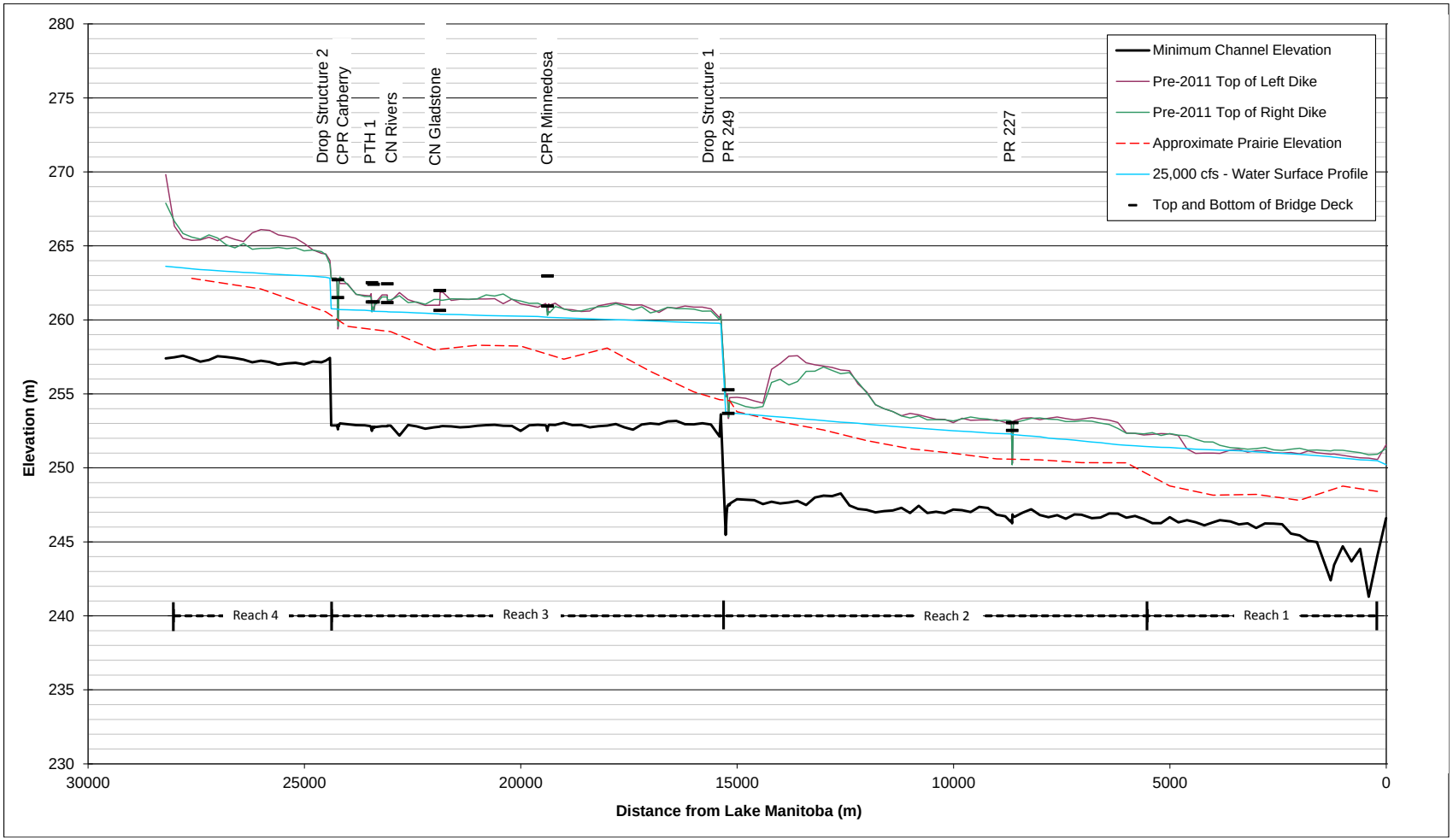
11.3.3.5 Design Details

Based on the design criteria outlined in Sections 11.3.3.1 to 11.3.3.5, a conceptual channel widening design was applied to the PD to increase the capacity of the PD Channel to 963 m^3/s (34,000 cfs).

Channel Widening

Since the existing channel geometry of the PD channel varies with location, a single and constant design upgrade template could not be applied to the entire channel. For the purposes of this study, the PD Channel was, therefore, subdivided into four reaches. The location of the four reaches is shown on Figure 11.2.

Figure 11.2
Profile of Portage Diversion

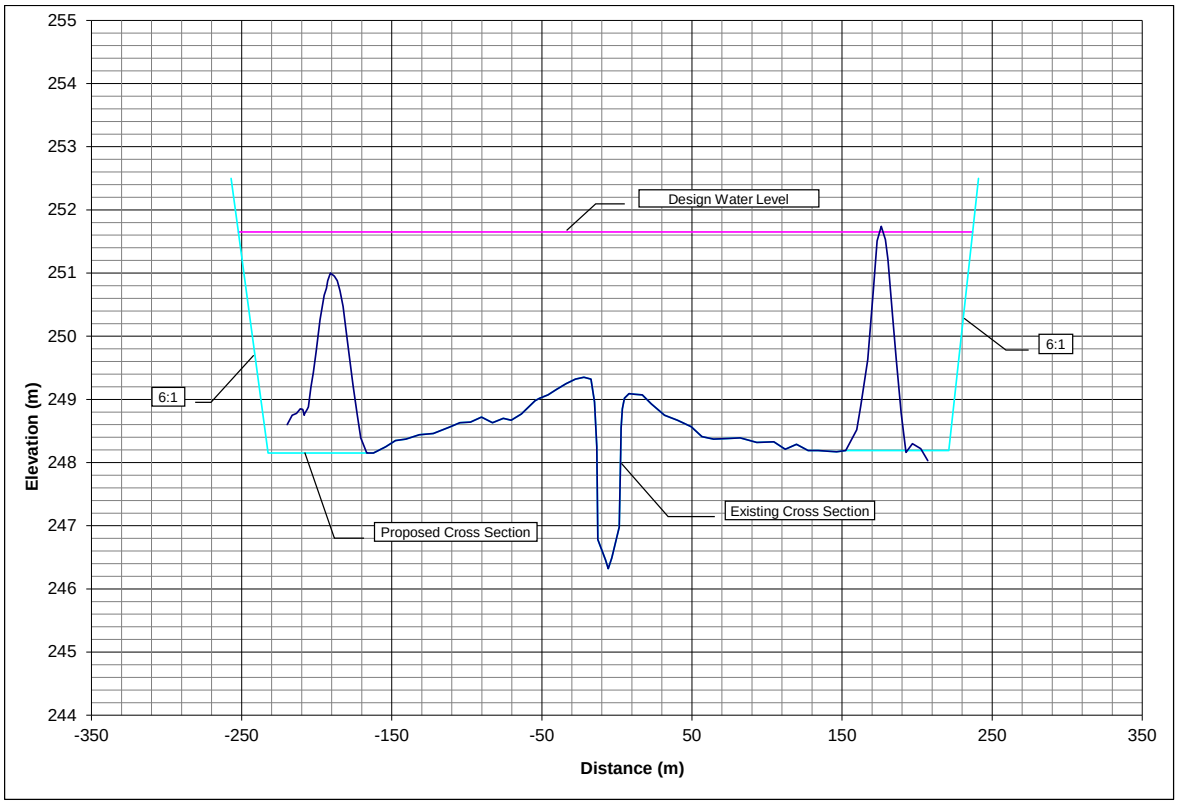


The PD channel consists of three main components: the low flow channel, the main channel and the west and east dikes. At the design flow of 708 m^3/s (25,000 cfs), the water elevation in the PD exceeds the top of the main channel and is contained by the dikes. The top of the main channel is also approximately equivalent to the existing prairie elevation on either side of the channel. The width of the channel varies depending on the reach. For example, the approximate distance between the east and west dike centreline in Reaches 1, 2 and 4 is approximately 400 to 425 m (1,300 ft to 1,400 ft). In Reach 3, the distance is approximately 180 m (590 ft). The offset between the dikes and the edge of the main channel also varies. For example, in Reach 3 it is approximately 60 m (200 ft) compared to Reaches 2 and 4 where the offset varies between approximately 100 m and 140 m (330 ft and 460 ft). In Reach 1, the original design of the PD did not include the excavation of a main channel; and, therefore, the entire flow is contained by the dikes. The profile of the Portage Diversion (Figure 11.2) shows the invert elevation of the low flow channel, the approximate prairie elevation (approximately equivalent to the top of bank of the main channel) and the top of dikes.

For the design of the dikes, an average 6:1 slope was assumed along Reach 1 based on a review of the existing dike geometry. Along Reaches 2, 3 and 4, the existing dikes appeared generally less steep than those in Reach 1. A conservative 8:1 slope was, therefore, assumed for the remainder of the PD.

As mentioned above, on Reach 1 the original design of the PD did not include the excavation of a main channel. Therefore, expansion of the PD would be achieved by setting back the dikes only. It was assumed that the existing ground elevation at the toe of the existing dikes would extend horizontally to the new location of the dikes. Lateral slopes of the channel invert to promote surface runoff were not incorporated into the design along this reach at this conceptual phase as the existing channel invert was located within marsh land, which naturally has less surface runoff. Furthermore, maintenance activities are currently planned along this reach by the Province of Manitoba to re-establish the 425 m^3/s (15,000 cfs) capacity of the channel at the location of the Failsafe. A detailed analysis along this reach, which considers surface runoff, as well as the effects of the proposed maintenance activities to the design are reserved for future studies, should this concept be selected to move forward to a subsequent phase of analysis. A typical cross section showing the existing channel along Reach 1 and the conceptual design of the proposed expansion to 963 m^3/s (34,000 cfs) is provided on Figure 11.3.

Figure 11.3
Typical Cross Section of the Channel Widening Option along Reach 1



To widen the channel along Reaches 2 and 4, excavation would begin from the toe edge of the existing main channel. Due to the relatively shallow depth of excavation (approximately 1 m or 3 ft and less), applying a uniform lateral slope of 50:1 to the proposed main channel invert would cause the channel excavation to daylight to the existing ground level too quickly, which would yield insufficient flow area to achieve the hydraulic criteria outlined previously. Instead, applying flatter 100:1 to 125:1 average lateral slopes to the main channel invert was necessary. Typical cross sections showing the existing channel and conceptual design along Reaches 2 and 4 are provided on Figures 11.4 and 11.5. To achieve proper drainage and promote surface runoff, a series of “steps” with alternating 50:1 lateral slopes could be incorporated into the design. Runoff would then be diverted back into the low flow channel at regular intervals along the PD. The potential for the “steps” to fill with sediments, as well as how best to manage drainage, would have to be further considered at the final stage of design. This concept is shown on the cross sections.

Figure 11.4
Typical Cross Section of the Channel Widening Option along Reach 2

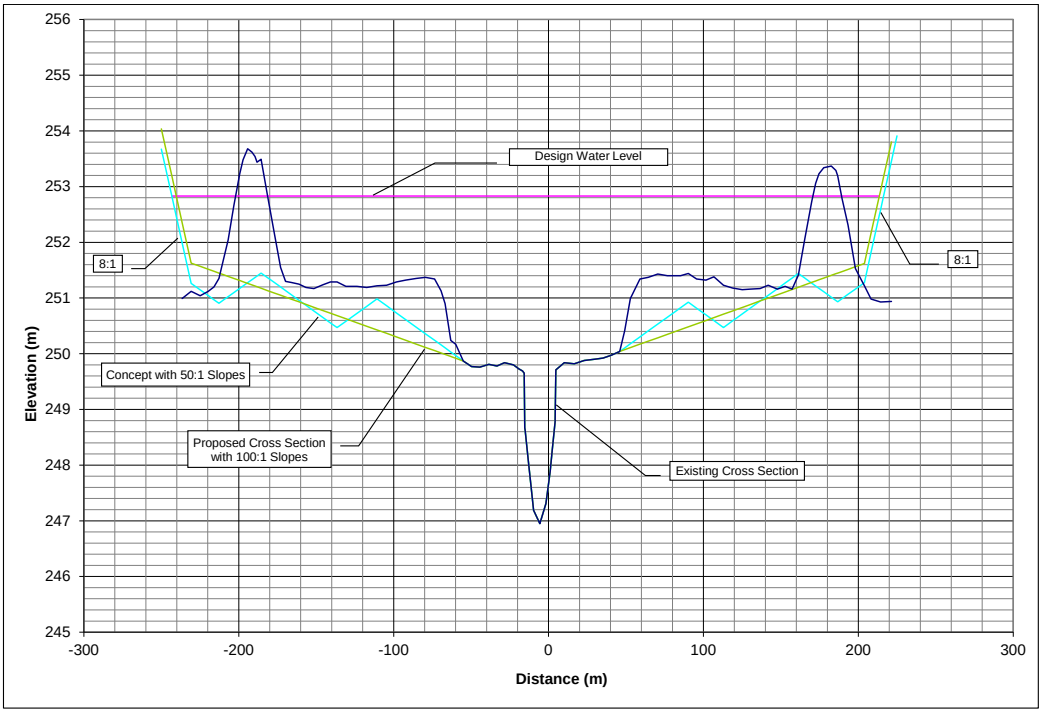
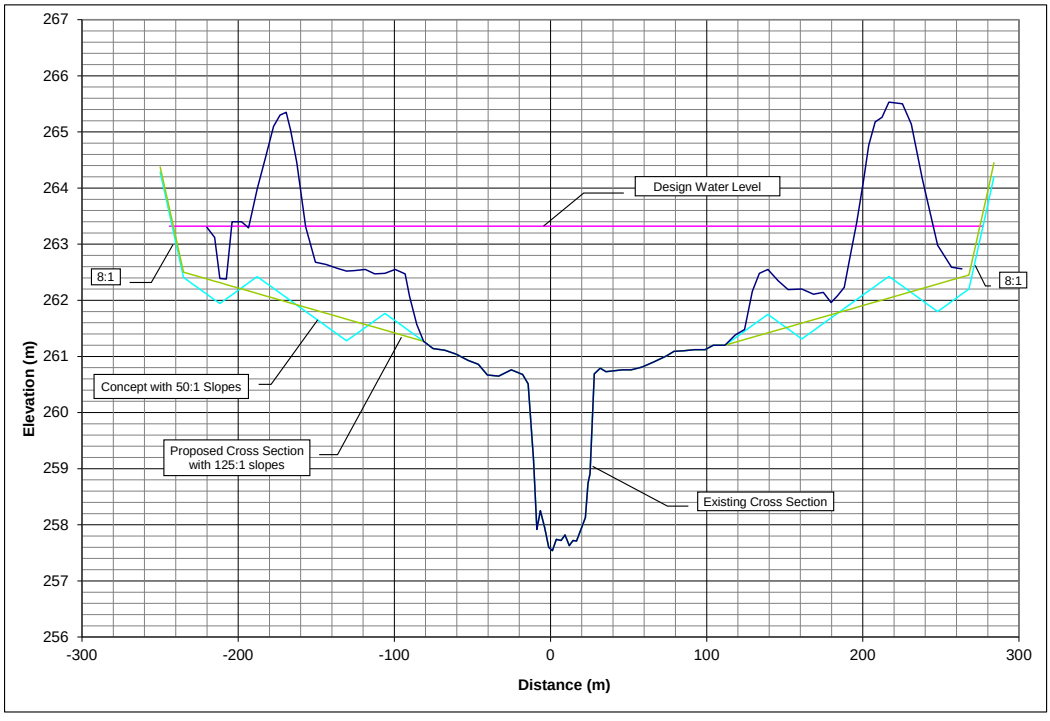
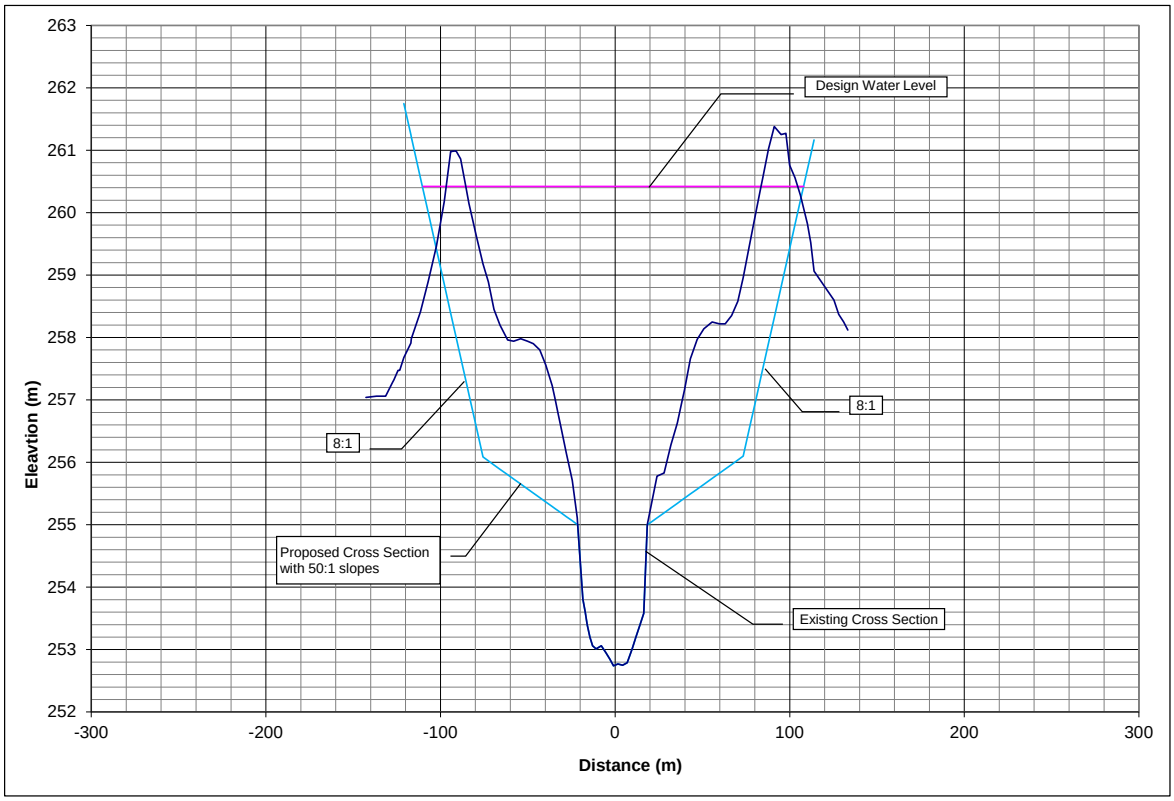


Figure 11.5
Typical Cross Section of the Channel Widening Option along Reach 4



Along Reach 3, the depth of excavation would be deeper and, therefore, a uniform lateral slope of 50:1 to the proposed main channel invert would be achievable. A typical cross section showing the existing channel and the conceptual design of the proposed expansion to 963 m^3/s (34,000 cfs) along this reach is provided on Figure 11.6.

Figure 11.6
Typical Cross Section of the Channel Widening Option along Reach 3



It should be noted that the east outside drain of the Portage Diversion currently needs improvement to better allow the drain to discharge into Lake Manitoba. This issue would be addressed as part of the Channel Widening Option, and it was assumed that the cost would be covered with the contingency of the option.

Channel Structures

As discussed in Section 11.3.3.4, expansion of the PD at the two drop structures, the River Control Structure and the Outlet Structure would be achieved by maintaining the existing

discharge per unit width at the structures. Table 11.8 summarizes the proposed width of the structures for the Channel Widening Option.

Table 11.8
Proposed Width of Control and Drop Structures for Channel Widening Option

	Proposed Width (m)
Outlet Structure	114.0
Drop Structure 1	62.2
Drop Structure 2	114.0
Diversion Control Structure	75.2

Bridge Crossings

The same methodology was applied for the seven existing bridges along the diversion. Table 11.9 lists all of the PD bridges and their proposed lengths for the Channel Widening Option.

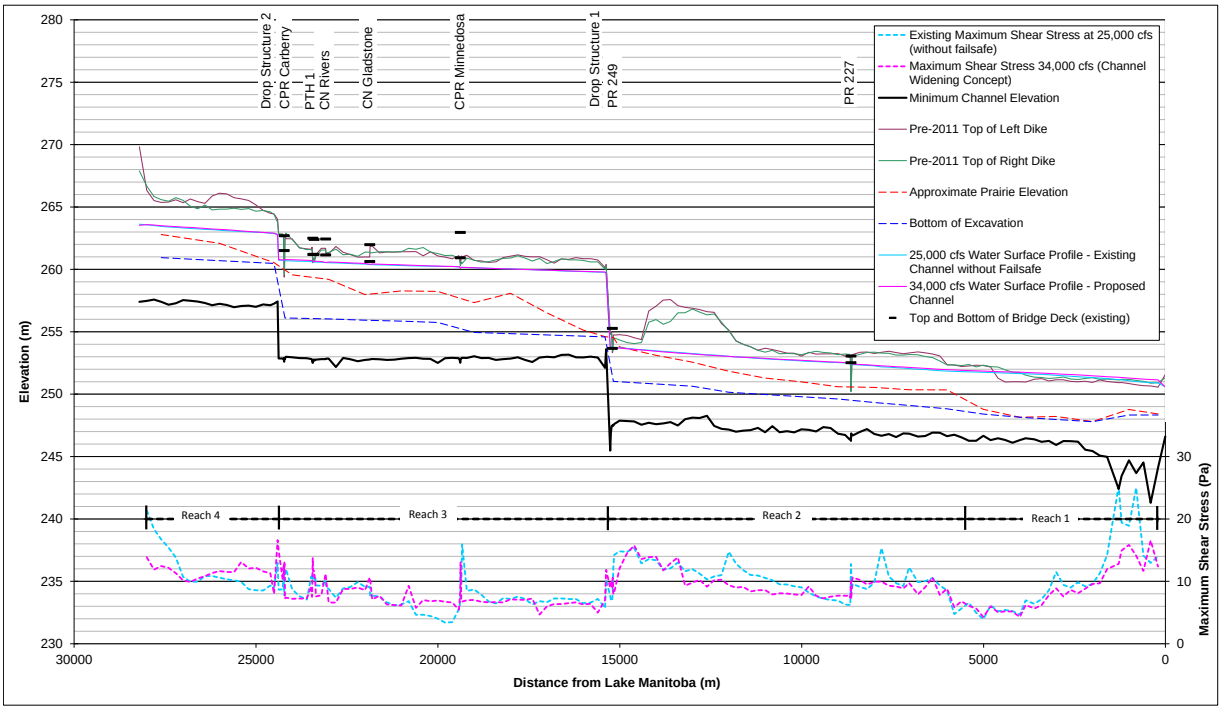
Table 11.9
Proposed Length of Bridges for Channel Widening Option

Bridge ID	Proposed Bridge Length (m)
CP Carberry	145
PTH 1 East & West	171
CN Rivers	233
CN Gladstone	235
CP Minnedosa	177
PR 249	173
PR 227	235

Hydraulic Profile

The HEC-RAS backwater model described in Section 8.3.4 was used to develop the channel widening concept and geometry described within this section. In the model, the same hydraulic coefficients and parameters that were used for the existing PD were applied for the proposed expansion alternatives. The proposed expansion of the drop structures, the Diversion Control Structure and the Outlet Structure previously discussed were included in the model. A hydraulic profile comparing the proposed 963 m^3/s (34,000 cfs) conditions to the existing 708 m^3/s (25,000 cfs) conditions in the PD is provided on Figure 11.7.

Figure 11.7
Hydraulic Profile of the Channel Widening Option

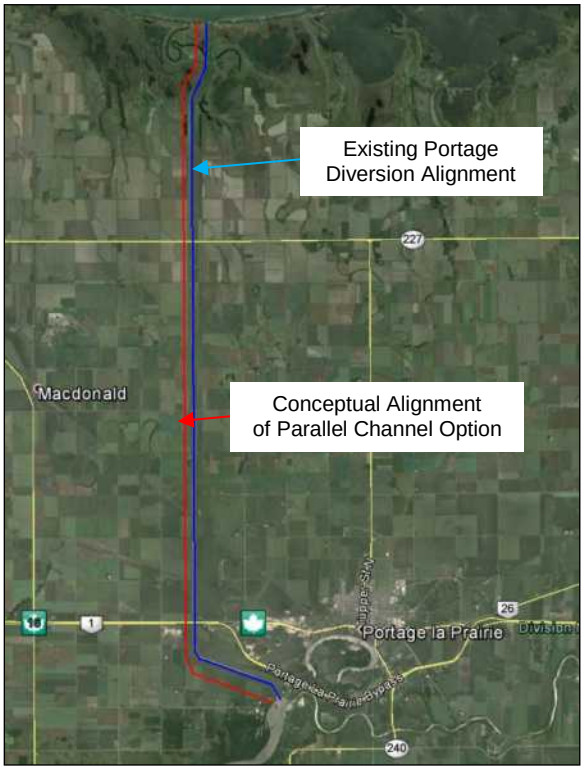


In the model, some adjustments at the bridge crossings were incorporated into the geometry to ensure that the bridges did not negatively impact the backwater results. However, the adopted bridge geometry outlined above was not finalized in the model as it was not necessary at this conceptual phase of analysis to obtain a high level cost estimate and proof of concept. Optimization of the backwater model is reserved for future studies once a final PD expansion concept has been selected.

11.3.4 Option 2 – Construction of a Parallel Channel

Increasing the capacity of the PD can also be achieved by constructing a new diversion channel extending from the Assiniboine River to Lake Manitoba. The Parallel Channel Concept consists of adding a secondary channel to the existing PD, which would be located immediately beside the existing channel. At this conceptual phase of study, the west side of the existing channel was selected for the analysis, however, future studies should identify whether this is a better alternative than an eastern alignment. The Parallel Channel considered for this study has a design discharge capacity of 255 m^3/s (9,000 cfs). Its conceptual alignment is shown on Figure 11.8. The existing configuration and hydraulic conditions of the current PD channel and structures formed the basis of the proposed Parallel Channel Option. Another factor not considered at this concept level of study was the potential environmental effects associated with a separate channel. Intuitively, greater public and environmental resistance from a second channel would be anticipated, as opposed to widening the existing channel. This factor has not been evaluated at this stage of study.

Figure 11.8
Conceptual Alignment of Parallel Channel Option



11.3.4.1 Diversion Control Structure

The original design of the Diversion Control Structure of the existing PD Channel had a discharge per unit width of 15.3 $m^3/s/m$ (165 cfs/ft). This discharge per unit width was maintained for the Diversion Control Structure of the new channel by adjusting the number of bays to match the required 255 m^3/s (9,000 cfs) design capacity. It was assumed for this study that the inlet structure of the new channel would have a similar design to the Diversion Control Structure of the existing PD channel (i.e., same dimensions and crest elevations), and that construction of the new structure could be achieved without undue difficulties in control of the Assiniboine River during non-flood conditions. The proposed Diversion Control Structure of the new channel would share the same reservoir as that of the existing channel.

11.3.4.2 Control of Spring Ice Runs

The existing design level of the PD reservoir of 264.87 m (869 ft) was maintained for the new channel. It was assumed that the same practices of holding the reservoir at a lower elevation of approximately 263.65 m (El 865 ft) during the spring ice breakup period, as discussed in Section 11.3.3.2, would be followed.

11.3.4.3 Ice Boom Control

The Parallel Channel was assumed to incorporate ice mitigation measures such as an ice boom to restrain incoming ice and prevent it from advancing through the Diversion Control Structure.

In the existing PD Channel, the current timber pontoon ice boom has not performed to its originally intended function, as discussed in Section 11.3.3.3. At this conceptual phase of study, it is proposed to replace the existing timber pontoons with more modern, robust steel pontoons filled with durable flotation.

11.3.4.4 Portage Diversion Channel and Drop Structures

The existing configuration and hydraulic conditions of the current PD Channel and structures formed the basis of the proposed Parallel Channel Option. It was assumed for this alternative

that the invert and slope of the new channel would equal that of the existing PD main channel. The locations of bridges and drop structures were also maintained for the new channel. It was also assumed that the excavated material would be spoiled on the west side of the channel to form the West Dike of the new channel. The West Dike of the existing channel would form the East Dike of the new channel. Future studies may address economies and difficulties that would come with constructing a deeper channel or whether design improvements could be made by selecting an alignment to the east of the existing PD channel instead of the west.

It was also assumed at this conceptual phase of design that by constructing the new parallel channel, the existing Failsafe Concept at the downstream end of the PD would be eliminated. Flood mitigation alternatives to reduce or eliminate damages due to overtopping of the Failsafe have been considered for this study and are documented in Section 11.2.

The following list addresses other key assumptions and design parameters applied for the Parallel Channel Option:

- The flow shear stresses applied to the surface of the channel must be kept within acceptable levels to minimize the risk of erosion under the design flow conditions. Shear stresses should not exceed those of the existing channel at its design discharge of 708 m^3/s (25,000 cfs).
- The channel should be non-aggrading, such that potential sediment deposits occurring during future use do not result in any detrimental impact to the discharge capacity of the channel. Small amounts of deposition that may require periodic minor maintenance could be acceptable, although are less preferred.
- The final geometry of the new channel should be such that it will promote surface runoff and minimize local ponding. Final surface grades should allow for any potential or reasonable changes that could affect runoff (e.g., surface settlements). Based on experience with the Red River Floodway Project, 50:1 lateral slopes were assumed necessary to achieve this criterion.
- The proposed geometry of the new channel should consider requirements to convey local runoff during non-flood periods. At this conceptual phase, it was assumed that this could be achieved with a low flow channel similar to the one in the existing PD channel. Future studies should identify whether this is the best solution to convey local runoff, and should be considered when selecting the final surface grades and establishing the revegetation plan for the new channel.
- The channel hydraulic analyses must incorporate appropriate energy losses due to channel roughness, geometry changes (i.e., contraction / expansion) and structures such as bridges.

- Changes in channel geometry should incorporate adequate transition zones to minimize hydraulic losses.
- The design flood profile of flow in the new channel should not exceed that of the existing channel at its design magnitude of 708 m^3/s (25,000 cfs).
- The design of the Drop Structures and the Outlet Structure will be based on those of the existing channel. The existing footprint and crest elevation of the current structures will be maintained for the new channel. The required structure width will be based on the 255 m^3/s (9,000 cfs) discharge capacity of the channel. Energy dissipation of the stilling basin for each structure should be possible with a submergence of the hydraulic jump that is comparable to conditions in the existing channel.

The offset distance between the top of the expanded Diversion Channel and the base of any transmission towers should allow for maintenance vehicle access. A minimum offset distance of 10 m (33 ft) has been proposed for the purposes of this study. Existing transmission towers that do not comply with this offset criterion would be deemed to require reconstruction. Based on a conceptual review of Manitoba Hydro's existing transmission network, three major transmission lines currently cross the Portage Diversion. On this basis, it is probable that one or more of the transmission towers will have to be reconstructed to comply with the 10 m (33 ft) offset. However, future studies should identify whether any economies can be achieved by optimizing the new channel geometry to minimise disturbance to the existing towers. At this conceptual phase of analysis, the reconstruction costs of transmission towers were estimated to be relatively low and, therefore, covered under the contingency cost allowance.

11.3.4.5 Assiniboine River Control Structure

As described in Section 11.3.3.5, it was assumed that the River Control Structure would be replaced for all of the options considered.

11.3.4.6 Design Details

The option of construction of a parallel channel was considered to increase the capacity of the PD channel to 963 m^3/s (34,000 cfs) based on the design criteria outlined in Sections 11.3.4.1 to 11.3.4.5.

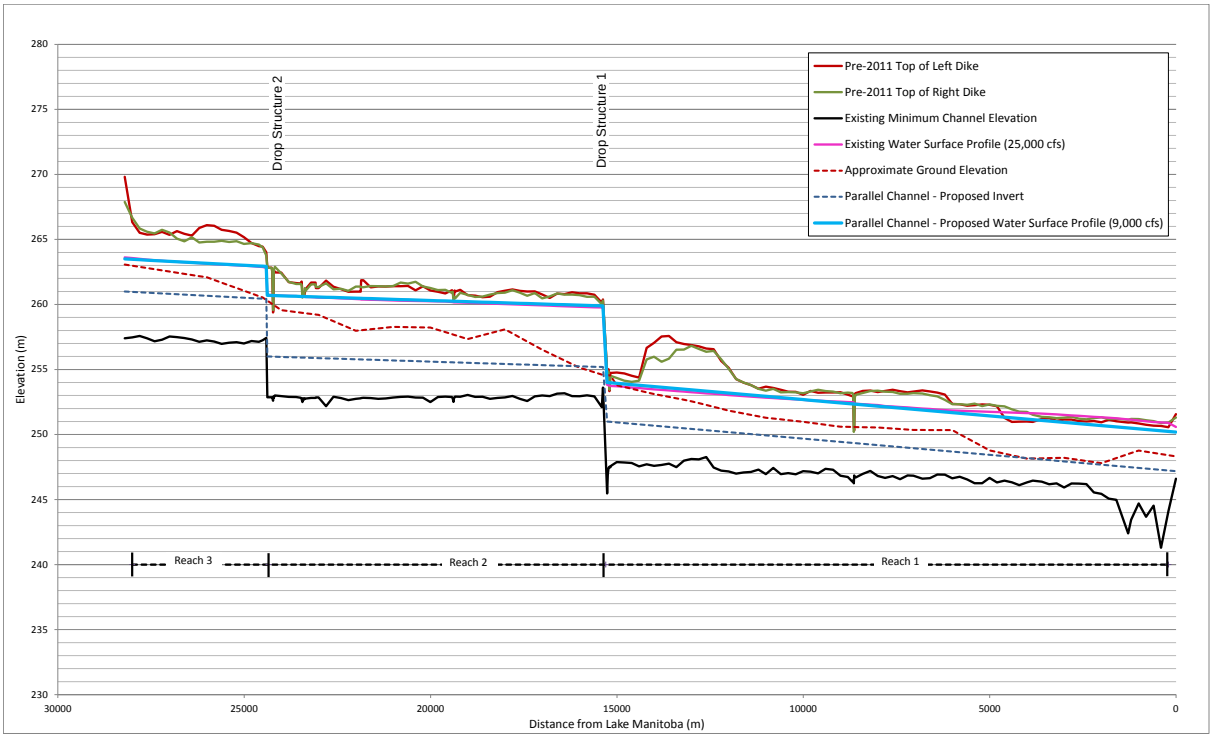
Parallel Channel

As previously mentioned, it was assumed for this study that the new channel would be located on the west side of the existing PD. To achieve the design criteria outlined above, the parallel channel was subdivided into three reaches. A summary of the conceptual hydraulic design of the parallel channel in each of the three reaches is provided in Table 11.10. A Hydraulic Profile of the option, which also shows the location of the three reaches, is provided on Figure 11.9.

Table 11.10
Hydraulic Design Summary for Parallel Channel Option
Q = 255m^3/s (9,000 cfs)

Description	Reach 1	Reach 2	Reach 3
Base Width	70 m	48 m	130 m
Side Slopes	6:1 (h:v)	6:1 (h:v)	6:1 (h:v)
Depth of Flow	3 m	4.7 m	2.5 m
Average Velocity	1.0 m/s	0.7 m/s	0.7 m/s
Approx. Reach Length	15,300 m	9,000 m	3,800 m
Channel Slope	0.025%	0.009%	0.015%
Assumed manning's "n"	0.03	0.03	0.03

Figure 11.9
Hydraulic Profile of the Parallel Channel Option

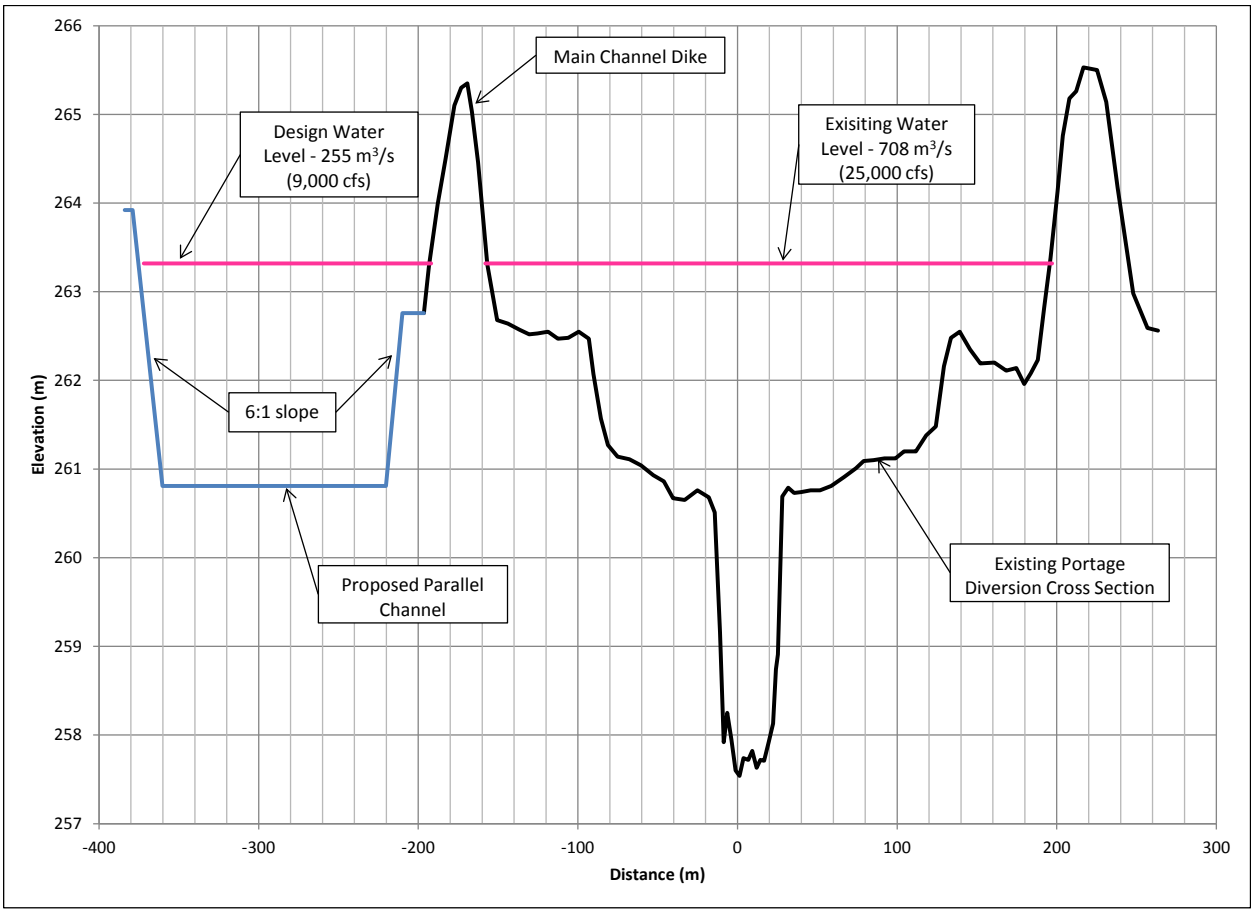


A flat bottom to the channel was assumed at this conceptual level of analysis. Future studies would incorporate a low flow channel and lateral slopes for local surface runoff, as per the design criteria discussed in Section 11.3.4.4.

Also, the material from the channel excavation would be spoiled on the west side of the new channel to form the west dike. It was assumed that the new dike would have 6:1 side slopes and a minimum height equal to the design water surface profile with 0.6 m (2 ft) of freeboard.

A typical cross section of the Parallel Channel Option is provided in Figure 11.10.

Figure 11.10
Typical Cross Section of the Parallel Channel Option along Reach 3



Channel Structures

The two drop structures, as well as the Diversion Control Structure and the Outlet Structure, were designed based on the existing structures. Table 11.11 summarizes the proposed width for the Parallel Channel Option.

Table 11.11
Proposed Width of Control and Drop Structures for Parallel Channel Option

	Proposed Width (m)
Outlet Structure	35
Drop Structure 1	20
Drop Structure 2	45
Diversion Control Structure	20

Bridge Crossings

For the seven bridges along the diversion, it was assumed that the bridge lengths would span over 70% of the top width of the channel. The 70% value was selected based on average conditions in the existing PD channel. Table 11.12 lists all of the PD bridges, and their proposed lengths for the Parallel Channel Option.

Table 11.12
Proposed Length of Bridges for Parallel Channel Option

Bridge ID	Proposed Bridge Length (m)
CP Carberry	73
PTH 1 East & West	73
CN Rivers	73
CN Gladstone	73
CP Minnedosa	73
PR 249	74
PR 227	74

11.3.5 Option 3 – Retrofit of Existing Channel

The Retrofit Option would essentially consider upgrading the existing channel to reliably pass 963 m^3/s (34,000 cfs). Although 963 m^3/s (34,000 cfs) was passed in 2011, it is recognized that

a number of temporary measures were utilized to pass the discharge without the failure of one of the components.

From a design criteria basis, recognition of the increased risk of this design concept will need to be acknowledged. The water surface profile at the original design discharge of 708 m^3/s (25,000 cfs) in the PD channel was above the surrounding prairie ground level (by about 2 m, or 6.5 ft, on average and as much as 5 m, or 16 ft in some areas). This exposes the dikes / spoil piles on each side of the channel to differential water pressures and requires them to serve as water retaining structures. The increase in the design flow to 963 m^3/s (34,000 cfs) would exacerbate this condition and raise the design water surface profile even further above the surrounding landscape (by up to 1 m, or 3 ft).

11.3.5.1 Control of Spring Ice Runs

Requirements for the ice boom and the control of spring ice runs would be similar in concept to that described for Options 1 and 2. Operational requirements would need to be modified to accommodate the higher design forebay levels associated with Option 3.

11.3.5.2 Channel and Structure Upgrade Requirements

The upgrade requirements have been defined with the assumption that concerns associated with the increased risk of the Retrofit Option could be overcome.

At this conceptual level of design with the development of the Retrofit Option, it was assumed that the following considerations would need to be addressed:

1. Theoretical information suggests that a headwater level of approximately El 265.63 m (El 871.5 ft) is required to pass 963 m^3/s (34,000 cfs) through the existing Diversion Control Structure. The nominal crest elevation of the water retaining structures is El 266.4 m (El 874 ft). This permits a freeboard of only 0.75 m (2.5 ft). This is 0.75 m (2.5 ft) less than what had been selected as a requirement for the original design, and it is not yet known to what extent it exceeds the recommended guidelines established by the Canadian Dam Safety Guidelines (CDA, 2009). This would need further investigation and would suggest a possible requirement of increasing the height of all the water retaining structures on the reservoir, or adding a bay onto the Diversion Control Structure, or some combination, depending on the optimum economic approach. The assumption utilized was that the original freeboard 1.5 m (5 ft) as a minimum would need

to be maintained. This would require raising the dikes around the perimeter of the PD reservoir by 0.75 m (2.5 ft).

2. The energy dissipation at the Diversion Control Structure must be examined for the increased flow and upstream water level. There was apparently a significant loss of riprap during the 2001 event. Nevertheless, the successful operation in 2011 at a flow approaching 990 m^3/s (35,000 cfs) suggests that the energy dissipation may have been adequate.

3. The original design of the PD required two drop structures to restrict the velocities in the channel to a magnitude that would not risk erosion of the channel bed. It appears that the operation in 2011 was successful; and with the exception of erosion in the pilot channels, there were no reports of serious erosion.

4. Similar to Point 3, velocities and shear stresses should also be checked at bridge crossings and approaching the drop structures and outlet structure to ensure that erosion would not be a significant risk. Riprap protection may be required to meet current standards.

5. The rockfill added for erosion protection at the drop structures in 2011 should be reviewed to ensure that the stone diameters installed were adequate to meet current standards. Other areas near the structures may also require additional rockfill for erosion protection and would require further study.

6. The pier heights at most of the bridges over the PD Channel are inadequate during passage of the target flow of 963 m^3/s (34,000 cfs). The bridge girders would either be partially submerged or would have a freeboard that is inadequate compared to modern design standards. Modifications of bridges that are already approaching the end of their useful life may be economically unwise, when viewed from a life cycle perspective. It is likely that full replacement of the bridges would be justified. For the purpose of this assessment, the assumption is that the bridges crossing the channel would need to be replaced.

7. Energy dissipation at the drop structures and the outlet structure should be verified to confirm that with the proposed channel hydraulic conditions at 963 m^3/s (34,000 cfs), there is adequate tailwater depth to ensure a stable hydraulic jump that has no risk of causing erosion in the riverbed below the structures. If the elevation of the stilling basin floor is too high to ensure a stable hydraulic jump, deepening of the basin should be considered. To assess this concern, a cursory comparison of the hydraulic performance of Drop Structure 1 with 708 m^3/s (25,000 cfs) versus 963 m^3/s (34,000 cfs) was conducted.

   - At 708 m^3/s (25,000 cfs) - HW El 259.6 m (El 851.7 ft), TW El 253.5 m (El 831.7 ft) (based on HEC-RAS model calculations). This provides a velocity into the stilling basin of 15.5 m/s (51 ft/s) and complies reasonably closely with the United States Bureau of Reclamation (USBR). This is consistent with the prudent design by the original design team.

   - At 963 m^3/s (34,000 cfs) - HW El 261 m (El 856.3 ft), TW El 254 m (El 833.3 ft) (again from the HEC-RAS model). This provides a velocity of 17.6 m/s (57.6 ft/s) entering the stilling basin, which is well over the recommended velocity for this type

of stilling basin. Under these conditions, it is likely that the basin was on the verge of a jump sweepout in 2011. This is far more risky than what should be adopted for a reliable stilling basin.

This check would suggest that, if a design flow of 963 m^3/s (34,000 cfs) were to be seriously considered, the stilling basin should be deepened, lengthened, and changed in form to a USBR-type stilling basin (chute blocks with a dentated end sill similar to SaskPower's EB Campbell) to be compatible with the USBR recommendations. Clearly, the assessment of all elements of the channel is beyond the scope of this concept study. However, this calculation demonstrates that all of the components will need to be evaluated.

8. The freeboard at the tie-in structures at each drop structure and the outlet structure should be assessed, and if found to be inadequate, a long-term solution for water retention should be developed. Similarly, adequate freeboard along the dikes adjacent to the channel should be assessed and confirmed as meeting suitable design criteria.

9. Stability of all the structures along the diversion channel, including the earthfill dikes along each side of the channel, should be assessed for the new conditions with a design flow of 963 m^3/s (34,000 cfs). Safety factors that are acceptable to the CDA Dam Safety Guidelines would be required. Modifications to the structures will likely be required to achieve this. As well, the dikes along the channel would need to be raised to accommodate increased water levels.

10. Similar to Item 9, stability of all the water retaining structures around the periphery of the reservoir, including the Control Structures, must be verified to meet suitable standards for the increased water level that may be needed for the expansion scheme selected.

11.3.5.3 Assiniboine River Control Structure

As described in Section 11.3.3.5, it has been assumed for all of the options considered that the long term plan for the River Control Structure is to be replaced. The hydraulic and mechanical requirements of the River Control Structure should also be assessed, depending on the solution that is adopted to make the Diversion Control Structure safe. If, for example, the headwater level for the design condition of the Retrofit option increases from El 264.87 m (El 869 ft) as it now exists, the increased head differential at the River Control Structure must be assessed:

- Is the desired outflow capacity (linked to the solution selected for the Assiniboine Dikes downstream) capable of being controlled with the control gates?
- Does the flow over the raised gates impinge on critical elements of the structure?
- Does the increased head differential, combined with the known degradation of the downstream channel and the reduction in the tailwater depth at the stilling basin, allow for suitable stability of the hydraulic jump?

The issue is that the two pressures from the decreasing tailwater level and the increasing headwater level may exceed the safe limits of operation of the stilling basin. If so, adjustments to the stilling basin may be required (deepening / lengthening). As well, additional modifications to the control structure and bridge may be required. If necessary, an additional cost allowance for replacement of the River Control Structure for the Retrofit Option should be considered at the next stage of design to address these issues. For the purpose of this study, the River Control Structure replacement costs for each option have been assumed to be the same.

11.3.5.4 Experience from 2011

In addition to the items identified above, the experiences of the 2011 Flood provided some insight as to the deficiencies that need to be addressed. Upgrade requirements for the Portage Diversion infrastructure were based on the KGS Group report, *"Geotechnical Emergency Engineering Services 2011 Flood Wrap Up Report"* (2013b).

- Bridge stability downstream of the Diversion Channel was one of the determining factors limiting the maximum flow rate to 963 m^3/s (34,000 cfs). Many of the bridges would not meet the freeboard criteria adopted for the Floodway Expansion Project (criterion was 0.6 m (2 ft) in the centre of the channel; with the camber of the bridge, this only permitted 0.3 m (1 ft) at each side). Scouring also occurred at some of the bridge piers in 2011.
- In 2011 seepage, boils and wet conditions were detected at the toe areas of some reshaped dry side slopes of the dikes. Canadian military crews placed geotextile and sandbags over these localized areas as a temporary emergency measure.
- Any dike breaches north of the CPR Minnedosa railroad, which encompasses about ¾ of the 29 km (18 mi) PD length, would flow north to Lake Manitoba and be confined to a width of 1.6 km (1 mi) on the east side and 4.0 km (2.5 mi) or less on the west side.
- Temporary measures at the Drop Structures to increase freeboard, improve stability and enhance erosion protection were implemented in 2011, but they had to be strategically added / removed as flows increased / decreased. This approach would not be reliable for a permanent capacity of 963 m^3/s (34,000 cfs) and would need some form of permanent retrofit.

The KGS report (2013b) is consistent with the list provided in the "*2011 Flood: Technical Review of Lake Manitoba, Lake St. Martin and Assiniboine River Water Levels*" (Manitoba, 2013a) report but provides additional background information. The technical review report identifies a number of emergency activities undertaken to increase the capacity of the Portage Diversion and to ensure that the diversion could continue to operate even with very high flows

and water levels. The enhanced Portage Diversion eventually conveyed flows of up to 983 m^3/s (34,700 cfs). Temporary measures to improve the Portage Diversion included:

- Raising the south dike of the Portage Reservoir.
- Raising the diversion channel dikes on both sides by approximately three feet.
- Placing additional erosion protection around the drop structures in the channel.
- Adding rock on the drop structures and around the bridges crossing the diversion channel to increase stability.
- Installing a gas generator to ensure the diversion gates could be operated if electrical power was affected by rising waters.
- Installing back-up hydraulic pumps to lift the bascule gates on the structure in the Assiniboine River in the event the mechanical room was flooded.
- Welding metal plates to the Assiniboine River control structure's control room floor and walls to allow the structure to remain operational once water levels in the reservoir exceeded the floor elevation.

11.3.5.5 Design Details

The Retrofit Option considered increasing the capacity of the PD channel to 963 m^3/s (34,000 cfs) and is based on the design criteria outlined in Section 11.3.5.1 to 11.3.5.4.

The final geometry of the expanded PD Channel, with the exception of the dikes will be essentially the same as the existing channel.

The upgrade design would be developed to methodically address the deficiencies experienced in 2011 and the criteria described in Section 11.3.5.1 to 11.3.5.4. Given the nature of the Retrofit Option, the requirements for this alternative have been less rigorously defined than the other two options. Should this option be accepted as a viable alternative, additional effort would be required to finalize the criteria and design requirements.

Reservoir and Channel Retrofit

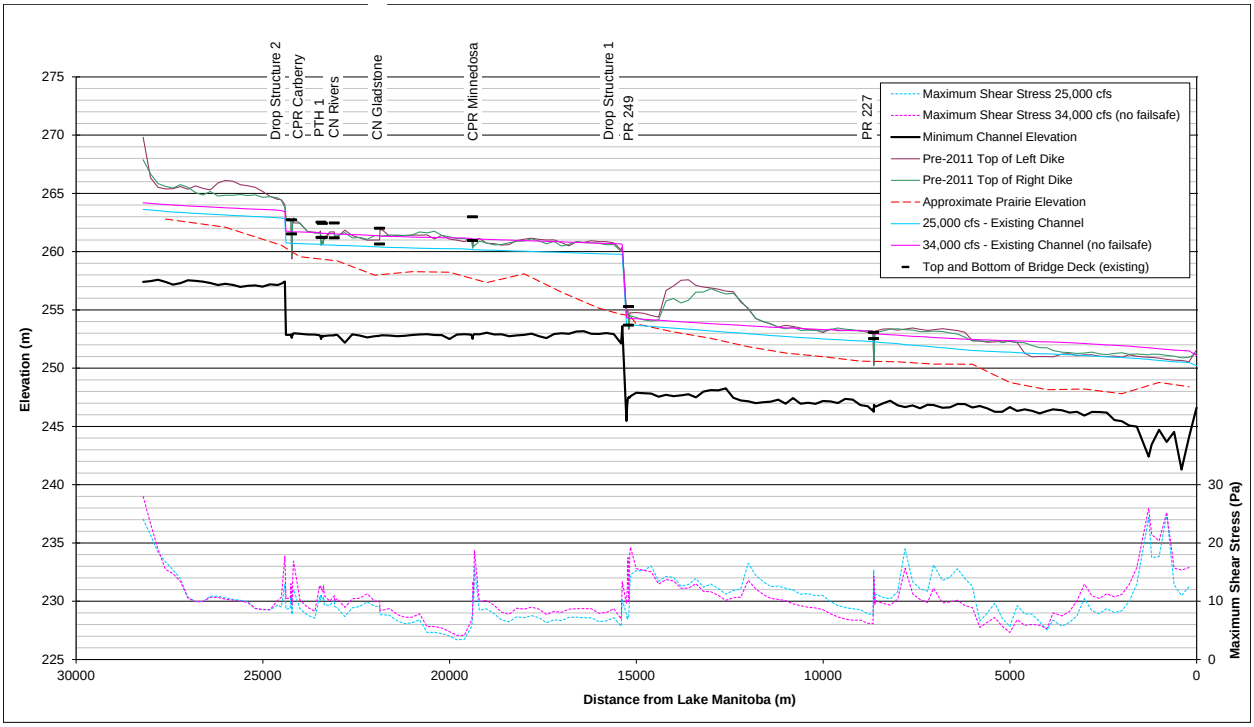
The dike upgrades for the reservoir and channel proposed for this concept require detailed geotechnical design that is beyond the scope of this study. Proposed dike improvements along

the PD Channel and reservoir were, therefore, based on those outlined in the Hatch Report, "*Final Investigation and Design Review Report for Geotechnical Design Review – Portage Diversion Channel Dikes, Portage la Prairie, Manitoba"* (Hatch, 2013). The proposed dike improvements outlined in Hatch 2013 were concentrated in areas that needed immediate remediation work within the channel. At this conceptual phase, it was assumed that the proposed dike improvements along the remainder of the channel and within the reservoir would have the same slopes (6H:1V) and crest width (5 m or 16 ft) as those proposed by Hatch. For the section which required immediate remediation, the 8 m (26 ft) wide and 1.5 m (5 ft) high toe berm to be placed at the downstream toe of the dikes recommended by Hatch was included. At the next stage of development, the design of the channel dikes would be more rigorously assessed based on additional surveys and data along the entire channel.

To be consistent with the Channel Widening and Parallel Channel options, the Retrofit Option assumed that the existing Failsafe area would be eliminated by raising the dikes along the channel. Flood mitigation alternatives to reduce or eliminate damages due to overtopping of the Failsafe have been considered and are documented in Section 11.2. Along the channel, the top of the dike elevation was set based on the 963 m^3/s (34,000 cfs) water surface profile plus 0.6 m (2 ft) of freeboard. In the reservoir, it was assumed that the entire length of dikes would be raised by 0.75 m (2.5 ft) to achieve the freeboard criteria.

A hydraulic profile of the Retrofit Option at its design discharge of 963 m^3/s (34,000 cfs) is provided on Figure 11.11 and is compared to the 708 m^3/s (25,000 cfs) hydraulic profile of the existing PD Channel. On the figure, only the "pre-2011" top of dike elevation is shown. Though the existing dikes were raised in 2011 and again in 2014, both times under emergency conditions, the proposed dike improvements outlined above are based on the pre-2011 dike profile as this was the only data available for the study at the time of the assessment. Existing freeboard deficiencies along the PD dikes for a flow of 963 m^3/s (34,000 cfs) are provided in Hatch, 2013. Using the "pre-2011" dike profile was deemed sufficiently accurate at this conceptual phase of study since the majority of the fill volume necessary to make the channel dikes more reliable comes from material required to flatten the dike slopes or to eliminate the Failsafe area. It is anticipated that using a "post-2011" dike profile would decrease the volume of fill material necessary for the dike improvements only by a minimal amount and, therefore, would likely be negligible.

Figure 11.11
Hydraulic Profile of the Retrofit Option



It should be noted that the east outside drain of the Portage Diversion currently needs improvement to better allow the drain to discharge into Lake Manitoba. This issue would be addressed as part of the Retrofit Option, and it was assumed that the cost would be covered with the contingency of the option.

Structure Requirements

Upgrading the PD at the two drop structures, the Diversion Control Structure and the Outlet Structure would be achieved by evaluating the requirements for replacement or retrofit. Based on a comparison of the hydraulic performance of Drop Structure 1 with 708 m^3/s (25,000 cfs) vs 963 m^3/s (34,000 cfs), as described in Section 11.3.5.2, it was concluded that the Drop Structure 1 basin was on the verge of a jump sweepout in 2011. This is more risky than what should be adopted for a reliable stilling basin, and it was assumed this structure would require replacement. A similar evaluation of the energy dissipation requirements of all the channel structures at the increased flows would be required.

Bridge Crossings

At the proposed design flow of 963 m^3/s (34,000 cfs), only one of the seven existing bridges meet modern design criteria for bridge clearance (assumed 0.6 m or 2 ft). This bridge is PR 227, which was replaced after the 2011 Flood. On this basis, it was assumed that the remaining bridges along the channel would need to be replaced. At this conceptual phase, it was assumed that the new bridges would span the same length as the existing bridges. Table 11.13 lists all of the PD bridges that need replacement and their proposed widths for the Retrofit Option.

Table 11.13
Proposed Length of Bridges for Retrofit Option

Bridge ID	Existing Bridge Length (m)
CP Carberry	107
PTH 1 East & West	125
CN Rivers	171
CN Gladstone	173
CP Minnedosa	130
PR 249	127

11.4 COST ESTIMATES FOR CHANNEL CAPACITY INCREASE OPTIONS

11.4.1 34,000 cfs Capacity

Cost estimates for the PD upgrade alternatives discussed in this document were based on the unit measures and prices outlined in Section 7.8 of this report.

Excavation quantities for the Channel Widening Option were calculated using the Channel Design / Modification tool in HEC-RAS. For the Parallel Channel Option, excavation quantities were calculated using estimated ground elevations determined from a combination of LiDAR data and cross sections from the existing PD. Since the Channel Widening option requires setting back the dikes on both sides of the PD, the volume of excavation for this option is significantly larger than that of the Parallel Channel Option. This makes the Channel Widening

Option more costly at this level of design. As discussed in Section 11.3.3, future studies may address economies and difficulties that would come with deepening or widening the channel on one side only for the Channel Widening Option.

The volume of impervious fill material necessary to improve the dikes for the Retrofit Option was based on data from Hatch (2013) for the channel dikes, and on the original PD design drawings for the Reservoir dikes. The quantities calculated were then increased by 25% to account for various losses during construction. An allowance for tree and stump removal along the dikes was also incorporated into the estimate.

The area of land required for acquisition was estimated based on land ownership and Right-of-Way (ROW) maps along the PD, and the conceptual footprint of the proposed Channel Widening and Parallel Channel options. For the Channel Widening Option, it was assumed that the new dikes would have a similar footprint area as the existing dikes. For the Parallel Channel Option, the footprint area of the dikes was based on the volume of channel excavation. An allowance of 30 m was added on the outside of the dikes for both options for local drainage and additional ROW. Purchase of land was assumed to be unnecessary for the Retrofit Option.

For the Parallel Channel Option, the purchase of an approximately 600 m (1,970 ft) length by 250 m (820 ft) wide parcel of land would be necessary. An assumed lump-sum value of $15 million was added to the estimate. Future studies should identify whether this additional cost can be avoided with an optimization of the Parallel Channel design and alignment in the area. As discussed in Section 11.3.4, future studies should also identify whether the proposed western alignment is a better alternative than an eastern alignment.

Revegetation costs to provide appropriate weed control and a vegetation cover was estimated based on the footprint area of the options.

Bridge costs were based on the footprint area of the bridges determined from the proposed bridge lengths summarized in Section 11.3 and the existing width of the bridges obtained from the HEC-RAS backwater model. An allowance of $3 million per structure was added to the costs for the rework of the bridge approaches and for traffic control during construction.

The costs for the two Drop Structures, as well as the Diversion Control Structure and the Outlet Structure were also based on the footprint area of the proposed structures determined from the widths and the existing lengths obtained from Hatch (2011b). Since the Diversion Control Structure also incorporated a bridge crossing and construction activities would require a cofferdam, temporary traffic control, and relocation of at least one of the existing abutments and approaches, the cost of upgrades for this structure would be larger. Therefore, a premium was added to the unit price for this structure.

For the Retrofit Option, based on a cursory analysis of the structures, it was assumed that Drop Structure #1 would need to be replaced and the other drop structure upgraded at a cost of 50% of the equivalent replacement cost. For the Diversion Control Structure, it was assumed that this structure was functionally adequate and that nominal upgrade costs of 10% of the replacement value would be required to accommodate the higher flows. Since the outlet structure is already at the end of its useful life, it was assumed that it would be replaced. In all cases, refurbishment refers to rehabilitation with a 50-year life extension in 2044.

For the Channel Widening Option, it was also assumed that the outlet structure would be replaced. For the remaining structures, it was assumed that only the incremental area of the structures necessary to achieve the 963 m^3/s (34,000 cfs) design condition would be charged to the capital cost of the project. However, for both Retrofit and Channel Widening options, it was assumed that the existing structures would have to be refurbished in 30 years at a cost of 25% of the total replacement cost. For the present valuation calculation, a 7% interest rate and 3% inflation rate were assumed. These assumptions are consistent with those of the “Do Nothing Option” as outlined in Section 11.4.2.

The River Control Structure was assumed to need replacement for all options, as it is currently at or near the end of its useful life. This was based on an assessment of the age and condition of the structure as well as feedback from MIT. This replacement assumption was, therefore, assumed for all options. In practice, MIT will likely need to consider some level of rehabilitation of the structure in the short term or a major rehabilitation of the structure to extend its life by 50 plus years. The River Control Structure rehabilitation option would be evaluated in greater detail at the next stage of design when the preferred option is selected and design parameters have been established.

As discussed in Section 11.3, a replacement ice boom is proposed for the PD. An individual cost item for the ice boom was not included in the estimates, but rather assumed to be covered under the contingency of the options. The contingency was also assumed to cover the cost of improvements to the east outside drain.

As discussed in Section 11.3.2.3, there are some concerns with salinity levels in the soils in the vicinity of the Portage Diversion. A thorough assessment to better delineate the mechanisms influencing salinity impacts on crops in the vicinity of the channel is warranted at subsequent study phases. The cost to address this issue will depend on potential mitigation measures if adopted. At the current conceptual level of analysis, it was assumed that this cost was covered under the contingency of the options.

Environmental costs were also included in the estimates. Minor mitigation efforts were assumed for the channel widening option due to the increased flows and the larger footprint, which could affect landowners. Minor mitigation efforts were also assumed for the retrofit option due to the increased flows, water levels and velocities. The Parallel Channel Option was assumed to have moderate mitigation efforts due to the increased flows, the larger increase in the project footprint, and the potential impacts on the First Nations near the inlet.

In addition to the direct costs outlined above, an allowance of 20% of the total direct costs was added to account for indirect costs such as engineering, construction supervision and owner's costs. An allowance of 30% of direct and indirect costs was added for contingency.

A summary of the costs for the PD expansion alternatives is provided in Tables 11.14 to 11.16. Quantities where applicable as well as unit rates are summarized in the tables. Note that Options 1 and 3 include costs for refurbishment of the structures in 30 years to allow economic comparison of the options. Incremental costs for economic comparison are based on the cost for these options minus the "Do Nothing" costs. Since the parallel option did not consider refurbishment of the existing structures, the "Do Nothing" costs have not been subtracted from this option.

Table 11.14
Summary of Costs for Channel Widening Option (#1)

Item	Description	Cost (Million)
Volume of Excavation	16,670,000 m^3 @ \$10/m^3	\$ 166.7
Revegetation	970 ha @ \$5,500/ha	\$ 5.4
Land Acquisition	110 ha @ \$10,000/ha	\$ 1.1
Hydraulic Structures (Replacement)	2,600 m^2 @ \$10,000/m^2 (Outlet Structure) 2,400 m^2 @ \$17,000/m^2 (River Control Structure)	\$ 66.3
Hydraulic Structures (Upgrades for Capacity)	2,200 m^2 @ \$10,000/m^2 (Drop Structures) 700 m^2 @ \$17,000/m^2 (Diversion Control Structure)	\$ 33.9
Hydraulic Structures (Refurbishment in 30 yrs)	PV [25% of 6,100 m^2 @ \$10,000/m^2 (Drop Structures)] PV [25% of 1,700 m^2 @ \$17,000/m^2 (Diversion Control Structure)]	\$ 7.0
Bridge Structures (Replacements)	10,600 m^2 @ \$7,000/m^2 (total of 7 bridges) & \$3.0M / structure (approaches & traffic control)	\$ 97.9
Environmental Mitigation	Minor – estimated at 5% of construction costs	\$ 18.9
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	Estimated at 20% of construction cost	\$ 75.7
Contingency	30% of direct and indirect costs	\$ 141.9
	Total Cost	**\$ 614.8**

Note: PV is the present value of expenditure assumed to occur in 2044.

Table 11.15
Summary of Costs for Parallel Channel Option (#2)

Item	Description	Cost (Million)
Volume of Excavation	3,528,000 m^3 @ \$10/m^3	\$ 35.3
Revegetation	400 ha @ \$5,500/ha	\$ 2.2
Land Acquisition	250 ha @ \$10,000/ha Additional Adjacent Land – \$15 M Lump Sum	\$ 17.5
Hydraulic Structures	3,800 m^2 @ \$10,000/m^2 (Drop / Outlet Structures) 700 m^2 @ \$17,000/m^2 (Diversion Control Structure)	\$ 49.0
Bridge Structures	4,100 m^2 @ \$7,000/m^2 (total of 7 bridges) & \$3.0M / structure (approaches & traffic control)	\$ 52.8
Environmental Mitigation	Moderate – estimated at 10% of construction costs	\$ 15.7
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	Estimated at 20% of construction cost	\$ 31.3
Contingency	30% of direct and indirect costs	\$ 61.1
	Total Cost	**\$ 264.9**

Table 11.16
Summary of Costs for Retrofit Option (#3)

Item	Description	Cost (Million)
Dike Improvements	2,389,000 m^3 @ \$20/m^3 (Channel Dikes) 109,000 m^3 @ \$20/m^3 (Reservoir Dikes)	\$ 50.0
Revegetation	190 ha @ \$5,500/ha (Channel Dikes) 10 ha @ \$5,500/ha (Reservoir Dikes)	\$ 1.1
Miscellaneous Allowance	Tree & Stump Removal – \$ 0.1 Million Lump Sum	\$ 0.1
Hydraulic Structures (Upgrades for Capacity)	50% of 3,200 m^2 @ \$10,000/m^2 (Drop Structure 2) 10% of 1,700 m^2 @ \$17,000/m^2 (Diversion Control Structure)	\$ 18.9
Hydraulic Structures (Replacements)	4,800 m^2 @ \$10,000/m^2 (Drop Structure 1 & Outlet Structure) 2,400 m^2 @ \$17,000/m^2 (River Control Structure)	\$ 88.8
Hydraulic Structures (Refurbishment in 30 yrs)	PV [25% of 3,200 m^2 @ \$ 10,000/m^2 (Drop Structure 2)] PV [25% of 1,700 m^2 @ \$ 17,000/m^2 (Diversion Control Structure)]	\$ 4.8
Bridge Structures (Replacement)	7,800 m^2 @ \$7,000/m^2 (All but PR 227 – total of 6 bridges) & \$3.0M / structure (approaches & traffic control)	\$ 65.9
Environmental Mitigation	Minor – estimated at 5% of construction costs	\$ 11.5
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	Estimated at 20% of construction cost	\$ 45.9
Contingency	30% of direct and indirect costs	\$ 86.1
	Total Cost	**\$ 373.1**

Note: PV is the present value of expenditure assumed to occur in 2044.

11.4.2 Do Nothing Alternative

The costs for the "Do Nothing" alternative are required for the economic analyses. The "Do Nothing" alternative represents the present value of the costs required to replace or refurbish the structures at the end of their design life. For the purpose of this assessment, the following assumptions have been made for the relevant structures below:

- Remaining Bridges – need replacement in 30 years except for PR 227, which was replaced in 2012.
- Drop Structures – need refurbishment in 30 years.
- Diversion Control Structure – needs refurbishment in 30 years.
- Outlet Structure– needs replacement prior to the project implementation.

- River Control Structure – needs replacement prior to the project implementation.
- Dam and Dikes – annual maintenance allowances adequate to maintain the dikes in a reliable state.
- Ice Boom – needs immediate replacement.
- East Outside Drain – needs improvements to allow the drain to discharge into Lake Manitoba.

For the present valuation calculation, a 7% interest rate and a 3% inflation rate were assumed.

A summary of the estimated "Do Nothing" costs is provided in Table 11.17. Quantities where applicable as well as unit rates are summarized in the table.

Table 11.17
Summary of Costs for Do Nothing Option

Item	Description	Cost (Million)
Hydraulic Structures (Refurbishment in 30 yrs)	PV [25% of 6,100 m^2 @ \$10,000/m^2 (Drop Structures)] PV [25% of 1,700 m^2 @ \$17,000/m^2 (Diversion Control Structure)]	\$ 7.0
Hydraulic Structures (Replacement)	1,900 m^2 @ \$10,000/m^2 (Outlet Structure) 2,400 m^2 @ \$17,000/m^2 (River Control Structure)	\$ 59.5
Bridge Structures (Replacements in 30 yrs)	PV [6,400 m^3 @ \$7,000/m^3 (All but PR 227 – total of 7 bridges) & \$3.0M / structure (approaches & traffic control)]	\$ 20.4
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	Estimated at 20% of construction cost	\$ 17.4
Contingency	30% of direct and indirect costs	\$ 31.3
	Total Cost	**\$ 135.5**

Note: PV is the present value of expenditure assumed to occur in 2044.

For the economic analysis, the project costs are based on the total cost of a given alternative minus the "Do Nothing" costs. This allows the state of the existing infrastructure to be accounted for in the assessment of alternatives and the calculation of the benefit / cost ratio. For example, all of the existing structures and bridges will need to be replaced or refurbished in the next 30 to 50 years regardless of whether or not the expansion proceeds, as discussed above. The incremental costs for comparison purposes and economic analysis are therefore the total cost of the alternative, minus a credit for the present value of those costs that would have been

required in the next 30 to 50 yrs (the "do-nothing costs"). Since the Parallel Channel Option is independent of the existing PD, there are no replacement or refurbish credits to consider in the Parallel Channel cost assessment. Table 11.18 summarizes the items considered in the incremental cost calculations. Table 11.19 summarizes the costs of the four options as well as their incremental costs.

Table 11.18
Summary of Items Considered for Incremental Costs

Component		Channel Widening (Option 1)	"Do Nothing"	Parallel Channel (Option 2)	"Do Nothing"	"Retrofit" (Option 3)	"Do Nothing"
Excavation / Dike Improvements		+		+		+	
Revegetation / Land Acquisition		+		+		+	
Bridge Structures	(New or replacement)	+		+		+	
	(Replacement in 30 years)		-				-
Outlet Structure	(New or replacement)	+	-	+		+	-
Drop Structures	(New or replacement)			+		+	
	(Upgrades for capacity)	+				+	
	(Refurbishment in 30 years)	+	-			+	-
Diversion Control Structure	(New or replacement)			+			
	(Upgrades for capacity)	+				+	
	(Refurbishment in 30 years)	+	-			+	-
River Control Structure	(New or replacement)	+	-			+	-

Notes:
"+" : item included in the cost estimate of the option.
"-" : item subtracted from the cost estimate for the incremental cost calculation.

Table 11.19
Summary of Cost of Options and Incremental Costs
for Portage Diversion Expansion Options

Alternative	Total Costs (Million)	Incremental Costs (Million)
Channel Widening	$ 614.8	$ 479.3
Parallel Channel	$ 264.9	$ 264.9
Retrofit Option	$ 373.1	$ 237.6
Do Nothing Option	$ 135.5	n/a

Note that the capital funds required to undertake a given option will need to subtract the costs allocated for the Present Value of future refurbishment.

11.4.3 Operating and Maintenance Costs

Operating and maintenance (O&M) costs are required for the economic analysis of the options. For the purposes of this assessment, the following assumptions have been made with respect to the O&M costs:

- Average annual O&M costs for the Parallel Channel Option equal to 1% of Total Capital Costs.
- Average annual O&M costs for the existing PD ("Do Nothing" option) equal to 25/9 times the O&M costs of the Parallel Channel. The ratio of 25/9 is based on the design flow capacity of the channels.
- Average annual O&M costs for the Channel Widening Option equal to 34/9 times the O&M costs of the Parallel Channel. The ratio of 34/9 is based on the design flow capacity of the channels.
- Average annual O&M costs for the Retrofit Option is 10% larger than the O&M costs of the existing PD.

For the economic analysis, the O&M costs are based on the average annual O&M costs of a given option minus the existing O&M costs ("Do Nothing" costs). This allows the state of the existing infrastructure to be accounted for in the assessment of alternatives and the calculation of the benefit / cost ratio. Since the Parallel Channel Option is independent of the existing PD, the existing O&M costs are not discounted from the Parallel Channel costs. Table 11.20 summarizes the O&M costs of the four options as well as their incremental costs.

Table 11.20
Summary of Operating and Maintenance Costs
for Portage Diversion Expansion Options

Alternative	Average Annual O&M Costs (Million)	Incremental Average Annual O&M Costs (Million)
Channel Widening	$ 9.8	$ 2.6
Parallel Channel	$ 2.6	$ 2.6
Retrofit Option	$ 7.9	$ 0.7
Do Nothing Alternative	$ 7.2	n/a

11.4.4 46,000 cfs Capacity

A cost estimate was also prepared for the option of increasing the capacity of the Portage Diversion to 1,303 m^3/s (46,000 cfs). As later discussed in Sections 22.0 and 23.0 of the report, without upgrades to the Lower Assiniboine River capacity, a 963 m^3/s (34,000 cfs) Portage Diversion on its own does not achieve the 1:200 year flood protection level criteria on the Lower Assiniboine River. A larger capacity Portage Diversion was, therefore, considered for the study to compare to other flood mitigation options such as increasing the capacity of the Lower Assiniboine River or the construction of large storage reservoirs for example.

It was assumed that the increase in capacity to 1,303 m^3/s (46,000 cfs) would be achieved with the construction of a new 595 m^3/s (21,000 cfs) parallel channel. The alternative option of channel widening was not considered in detail at this conceptual phase as it would require deepening the channel invert and/or increasing the water levels and velocities in the channel to convey the additional flow, which was not consistent with the design criteria outlined in Section 11.3.2. The Retrofit option was not considered beyond the 963 m^3/s (34,000 cfs) capacity as it would further increase the water levels in the diversion channel and reservoir above the surrounding landscape, which was not desirable. However, a hybrid option of widening and retrofitting the existing PD could be considered at the next stage of design if selected to proceed. This level of optimization was not considered at the conceptual level as it fell beyond the scope of the study.

The cost estimate for the 595 m^3/s (21,000 cfs) parallel channel was based on the same design criteria and unit costs as those outlined for the 283 m^3/s (9,000 cfs) Parallel Channel Option

discussed in Sections 11.3.4 and 11.4.1, but with quantities increased based on the larger capacity. Due to the greater level of uncertainty for this option, an additional allowance of 10% was added to the contingency. The estimated cost for the option is summarized in Table 11.21.

Table 11.21
Summary of Costs for 46,000 cfs Portage Diversion

Item	Cost (Million)
Volume of Excavation	$ 87
Revegetation	$ 4
Land Acquisition	$ 18
Control Structures	$ 106
Bridge Structures	$ 81
Environmental Mitigation	$ 30
Indirect Costs (Engineering, Administration, Approvals, Owner's Costs)	$ 59
Contingency	$ 155
Total Cost	**$ 540**

11.5 CONCLUSIONS AND RECOMMENDATIONS FOR CHANNEL CAPACITY INCREASE OPTIONS

At this level of concept design, the retrofit option was identified as the preferred alternative to upgrade the Portage Diversion capacity, since it had the lowest cost. The cost is estimated to be $238 million more than the "Do Nothing" option, but is $27 million less than the combined cost of the parallel channel option and the "Do Nothing" option. However, the Retrofit Option requires careful consideration of the dam safety issues and the additional risk associated with this option in the overall decision making process. Additionally, the parallel channel will almost certainly have greater environmental and socio-economic impacts. This is primarily due to the additional land acquisition requirements and expansion of the overall project foot print.

The Province is currently evaluating some of the issues associated with the "retrofit" that have been identified in the report, such as the adequacy of the Portage Diversion dikes raised in 2011. Should the Province wish to pursue the retrofit option, this would best be done at a

functional design stage where the criteria for the preferred options could be established and a more robust comparison of the alternatives could be undertaken.

DRAWINGS

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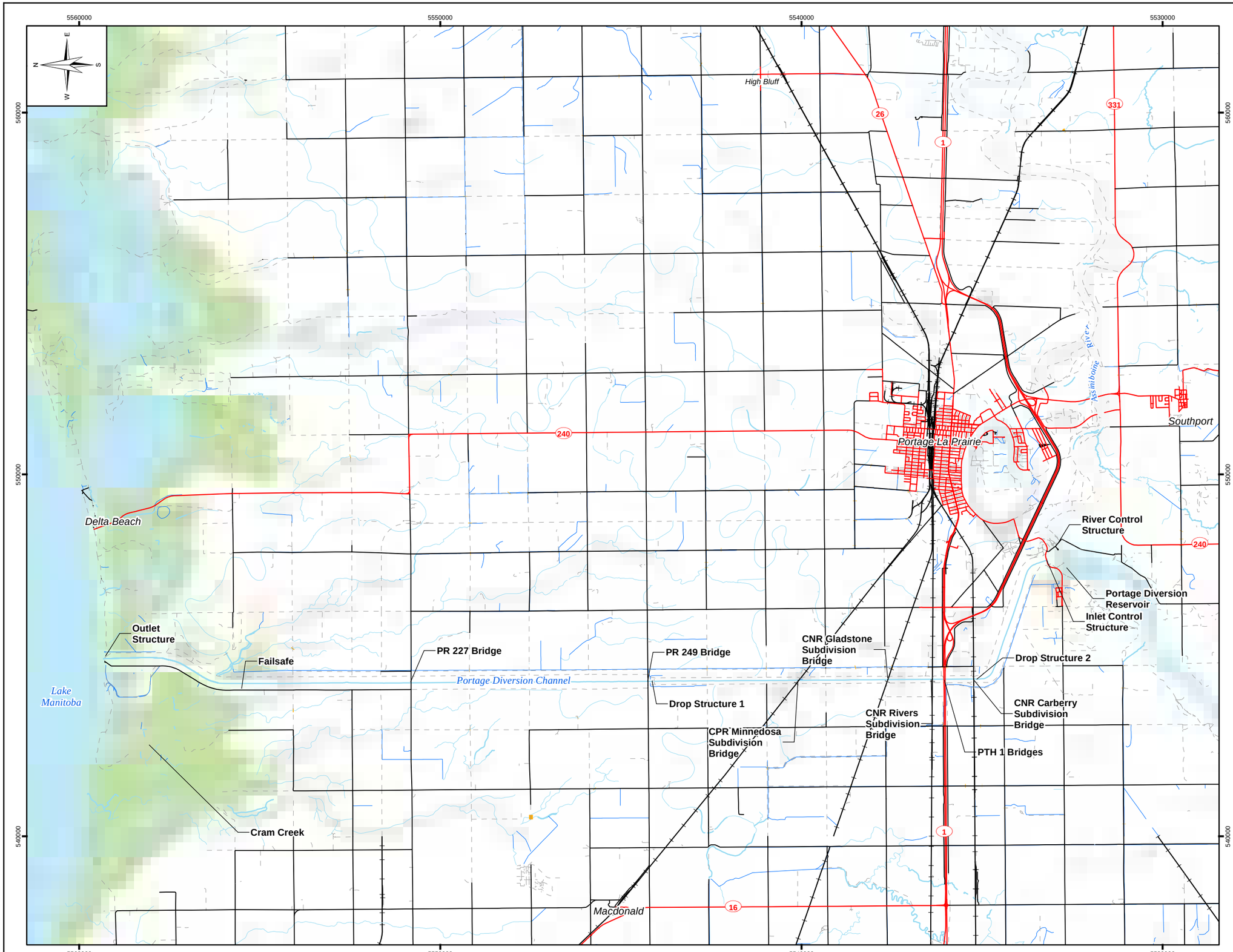

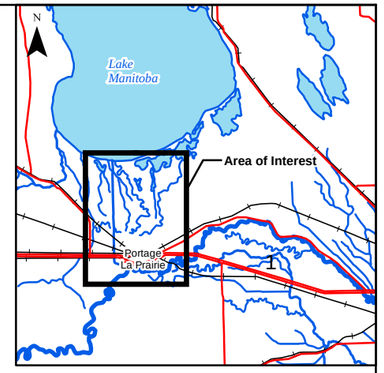

LEGEND:

- Road (Paved)
- Road (Gravel)
- Road (Unclassified/Trails)
- Railway
- Ditch
- River/Stream, Indefinite
- River/Stream
- Dugout; Pond;
- Island
- Marsh; Swamp
- Lake
- First Nation

1 0 1 2 3 4
Kilometres

SCALE: 1:100,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

GENERAL PLAN OF PORTAGE DIVERSION

JANUARY 2016 | DWG 11.1 | REV: 0

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11"x17" PLOT SCALE 1:1

12.0 SOURIS RIVER

12.1 INTRODUCTION

The Souris River is the largest tributary contributor to the Assiniboine River Basin. It originates in Saskatchewan and flows south into North Dakota and then back north into Manitoba. Flows in the upstream portion of the basin are mostly controlled by three large reservoirs: the Rafferty and Alameda Reservoirs in Saskatchewan and Lake Darling in North Dakota. In Manitoba, there are a number of tributaries that can also contribute a significant volume of water to the river. In 2011, a significant volume of water originating mostly from the basin's headwaters caused unprecedented flooding throughout the basin. In Manitoba, emergency diking was required in the communities of Souris, Melita and Wawanesa.

For the communities of Souris and Melita, commitments are currently in place to increase the level of protection to the highest flood on record (2011 Flood). However, this level does not exceed the 1:200 year FPL. Upgrades to the diking system in these two communities were, therefore, examined and have been discussed in Section 12.4. In Wawanesa, the current diking system exceeds both the highest flood on record (2011 Flood) and the 1:200 year flood event, but not the 1:500 year flood event. Therefore, flood protection upgrades have also been evaluated for this community and are discussed in Section 12.4.

12.2 SUMMARY OF RESULTS FROM PAST STUDIES

The report prepared by the Souris River Basin Board in the late 1970s (SRBB, 1978) was a strong basis for assessment of flood planning options along the Souris River for the current study. That report was the culmination of several years of study of the full spectrum of water resource management in that watershed. It addressed nine sectors including water quality, recreational use, agriculture and flood protection. The current study focused on the analyses and reported results in the sector of flood protection.

The studies for the Souris River Basin Board provided a comprehensive review and quantification at that point in time of flood damages both in terms of monetary losses as well as

in numbers of people affected. The studies addressed both structural and non-structural means of flood protection.

Recommendations of the study were numerous and far-ranging. The key recommendations, which related to flood protection that proved useful for the current studies, were as follows:

- Zoning and land use regulations should be developed and implemented as soon as possible to prevent further development in flood risk areas.
- A program of relocation should be initiated for flood prone buildings and developments and incentives should be considered for this purpose.
- Flood-proofing should be encouraged where relocation would not be practical.
- Flood damage reduction measures recommended for implementation included:
  - A maintenance program for removal of obstructions such as beaver dams and debris.
  - Land drainage projects should be allowed to proceed provided that increased flooding problems caused were offset by appropriate mitigation measures, or by compensation.
  - Specific measures including diking and flood proofing at Weyburn, River Park near Estevan, Roche Percee, Beaver Park near Oxbow, between the Yellow Grass Marsh and Weyburn, Melita, Souris and Wawanesa.

In addition to the recommendations, there were a number of findings and conclusions that were of significant value in guiding the current studies. These included the results of studies of a variety of structural measures including channel improvements, reservoirs, and diking of agricultural interests. These were in general shown to be uneconomical, largely due to the excessive costs of construction relative to the modest reductions in damages that could be attributed. The only marginally effective schemes included diking at the communities of Melita, Souris and Wawanesa.

12.3 STORAGE RESERVOIR(S)

12.3.1 Description of Alternatives

Between 1956 and 1979, the Prairie Farm Rehabilitation Administration (PFRA) investigated three potential sites on the Souris River mainstem. The sites included the High Souris Dam

located in NE 28-7-21W1, the Langvale Dam in 9-6-18W1, and the Nesbitt Dam in 1-7-18W1. The High Souris Dam and the Nesbitt Dams were selected for review in this study as they were the largest reservoirs and had the greatest potential to provide flood reduction benefits downstream on the Souris and Assiniboine Rivers.

Both the High Souris and Nesbitt Dam Projects were studied in 1968 for the Saskatchewan-Nelson Basin Board (SNBB) and are documented in *"Water Supply for the Saskatchewan-Nelson Basin"* (SNBB, 1972). Further studies on this project were also done by Prairie Farm Rehabilitation Administration (PFRA) in the late 1980s and are documented in the "*Reconnaissance Study of Potential Storage Development on Assiniboine River Tributaries"* (PFRA, 1989). Information from SNBB, 1972 and PFRA, 1989 formed the basis of the concepts outlined in this report. The conceptual location of the two reservoirs is shown on Drawing 12.1.

12.3.2 High Souris Dam

The High Souris Dam would be located on the Souris River about 3 km (2 mi) upstream from the Town of Souris. The reservoir would extend upstream a distance of 48 km (30 mi) inundating 1,400 ha (3,500 ac) of cultivated land and bush at the full supply level.

Based on SNBB, 1972, a CPR bridge would have to be replaced and sections of the railway would need to be raised and protected. Two highway bridges would have to be replaced and embankments improved. The embankments on two other highways would have to be improved, as well. Power and telephone lines would be relocated as would three farmyards in the reservoir area. The Hartney Dam would be flooded and abandoned.

The embankment of the proposed High Souris Dam would be a zoned earthfill dam with slopes of 3:1 and 4:1 upstream and 2.5:1 and 4:1 downstream. A downstream sloping and horizontal filter and an upstream impervious blanket would control seepage through the dam and foundation.

The spillway would be comprised of a gated 9-bay, 110 m wide (360 ft) overflow structure with a crest at El 422.15 m (El 1,385 ft). The spillway gates would be vertical lift gates, 12.22 m wide by 5.18 m high (40.1 ft x 17 ft).

A 24 m long reinforced concrete conduit, with a horseshoe shaped cross-section, would pass the bank full discharge at the reservoir full supply level (FSL).

The general arrangement of the High Souris Dam concept is shown on Figure 12.1. A typical cross section of the dam embankment is shown on Figure 12.2.

Figure 12.1
General Arrangement of High Souris Dam Concept

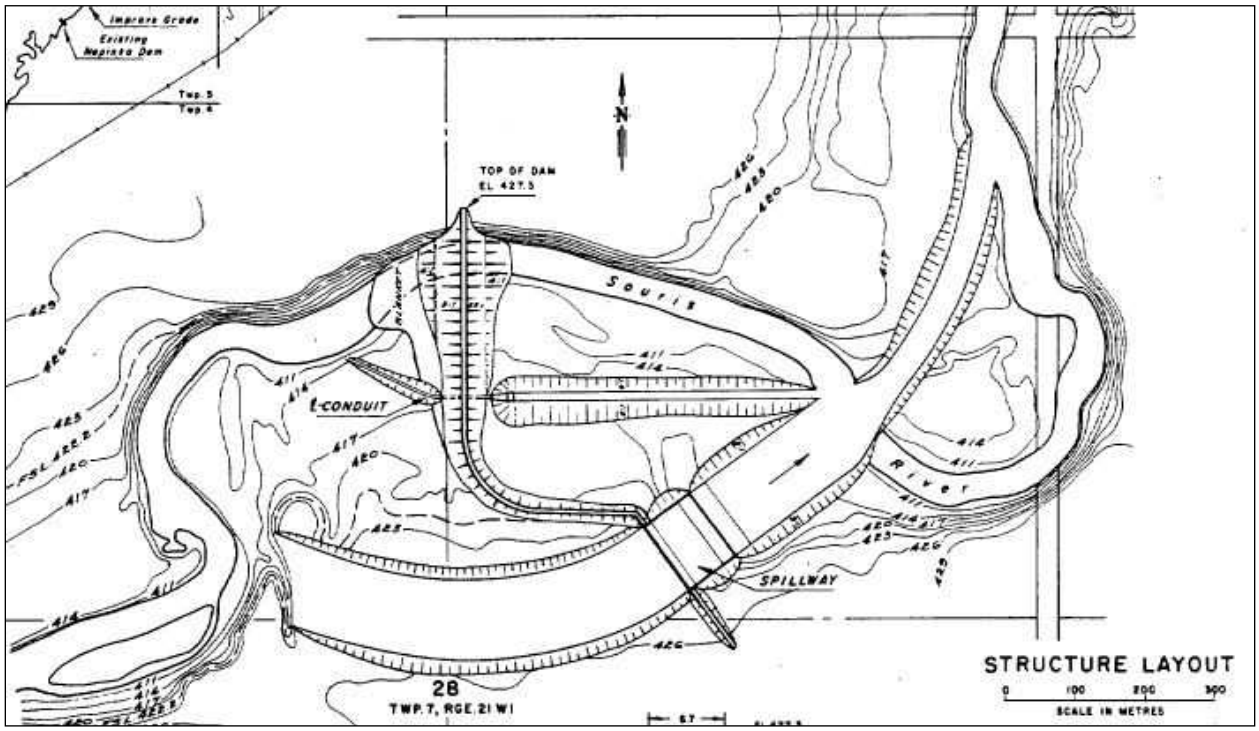
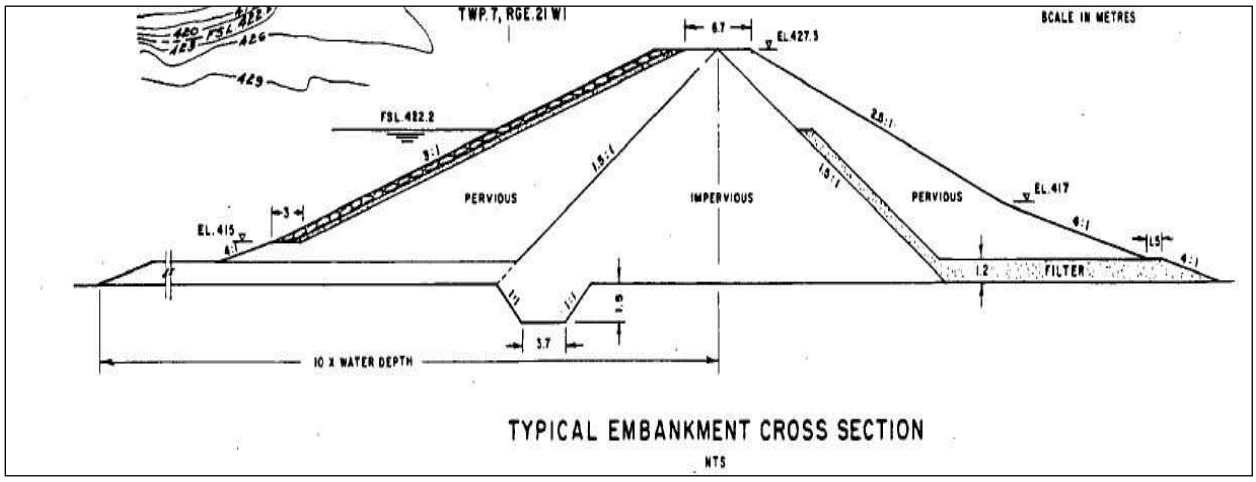
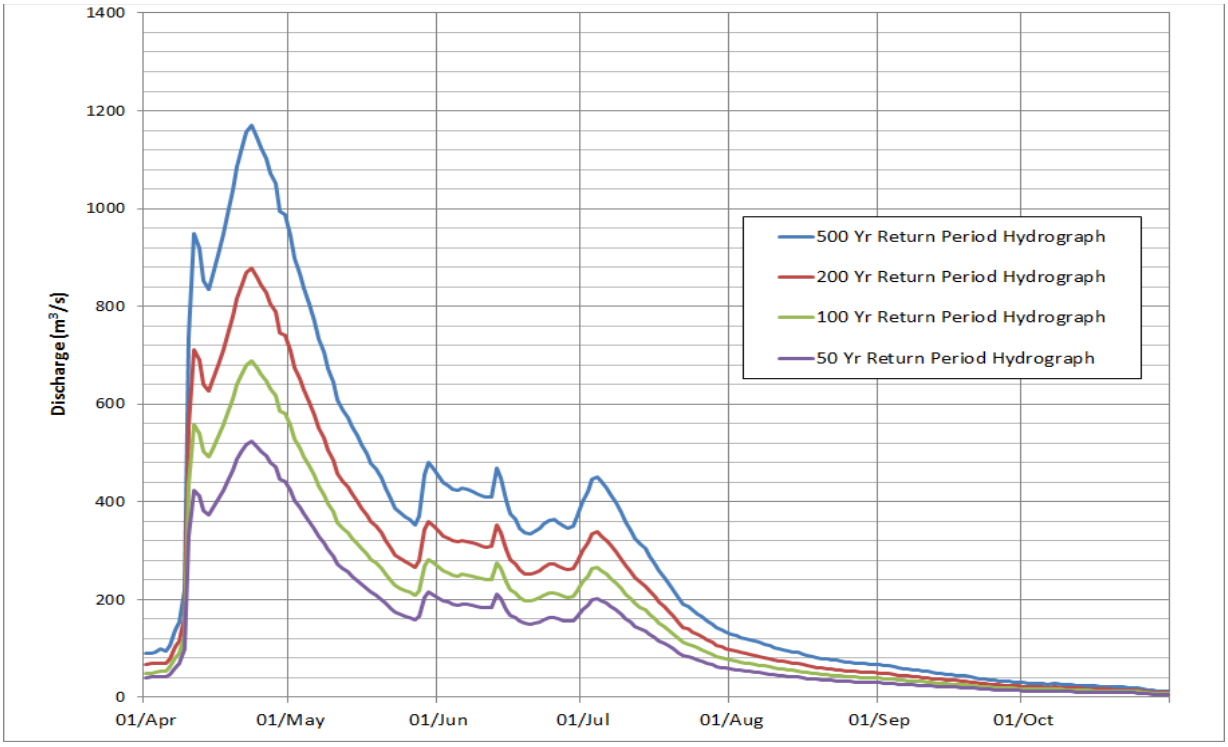


Figure 12.2
Typical Cross Section of Embankment



Design hydrographs of the Souris River were developed at Wawanesa based on a discharge frequency curve provided by MIT and a volume frequency curve prepared for the current study, as documented in Section 5.4 of this report. The hydrographs were then scaled by prorating the design hydrographs at Wawanesa to the drainage area at the dam site. The design inflow hydrographs for the High Souris Reservoir, for the return periods from 50 years to 500 years are shown on Figure 12.3.

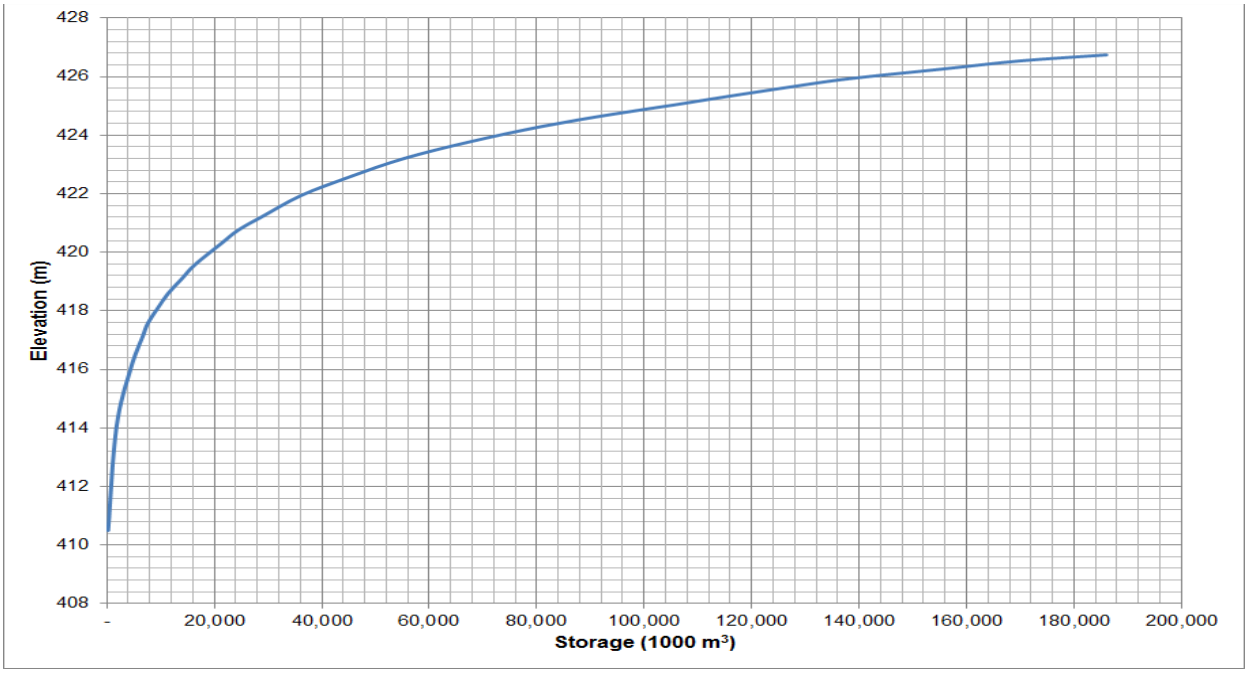
Figure 12.3
Design Hydrographs of Souris River at High Souris Dam



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

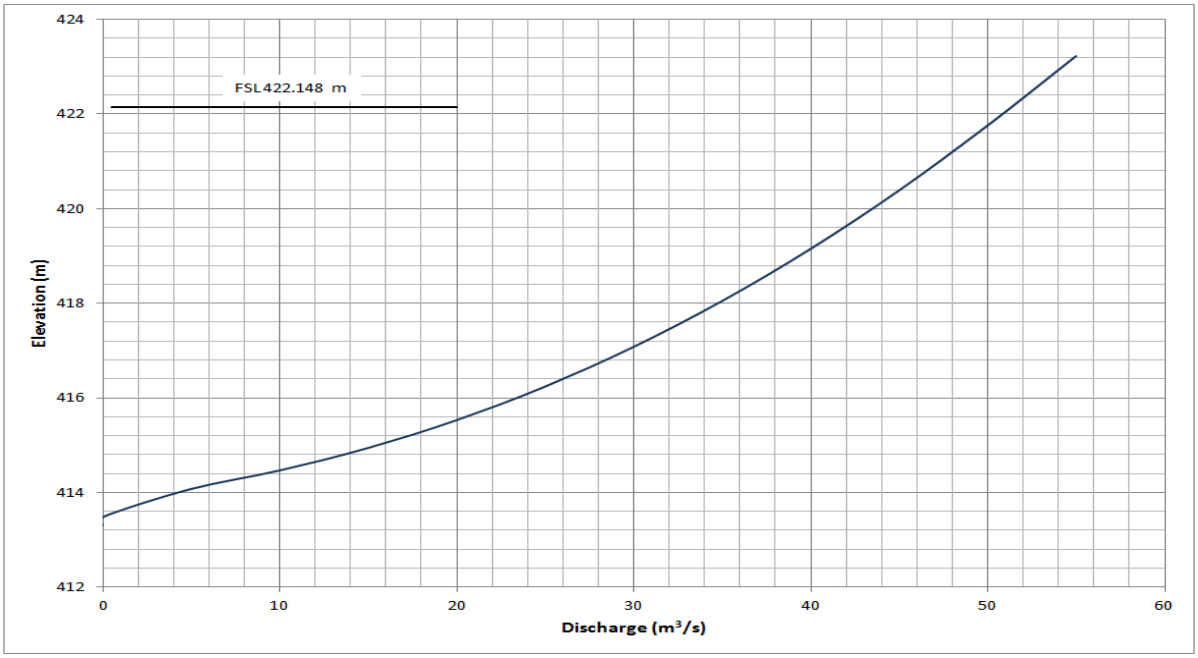
The stage-storage curve for the High Souris Dam is shown on Figure 12.4 and was taken from PFRA, 1989.

Figure 12.4
High Souris Reservoir Storage Curve



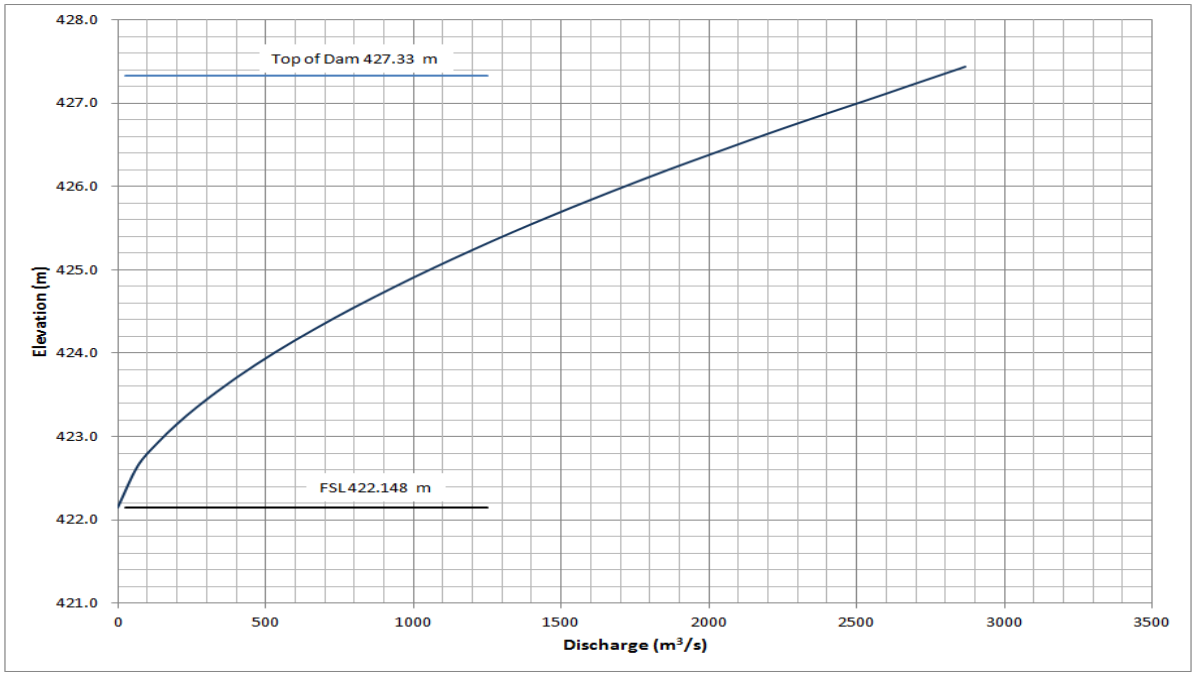
Stage-discharge rating curves for the low level conduit and the overflow spillway are illustrated in Figures 12.5 and 12.6, respectively.

Figure 12.5
Low Level Conduit Rating Curve



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 12.6
Overflow Spillway Rating Curve



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

12.3.3 Nesbitt Dam

The Nesbitt Dam would be located on the Souris River about 10 km (6 mi) upstream of the Town of Wawanesa. The reservoir would extend upstream a distance of 49.9 km (31 mi) inundating 2,900 ha (7,170 ac).

Based on SNBB, 1972, the reservoir would flood PR 346 and PTH 10. PR 346 would be closed and its bridge removed while PTH 10 would be raised above the maximum flood level. PR 348 Bridge would also have to be raised, as well as the power line along PTH 10. In addition, operation of the reservoir would raise the water level at the Town of Souris. SNBB, 1972 states that a total of fourteen buildings in the vicinity of the town would need relocation and two bridges on Plum Creek would be replaced. It is likely that additional infrastructure would also be affected due to development along the river since the original analysis.

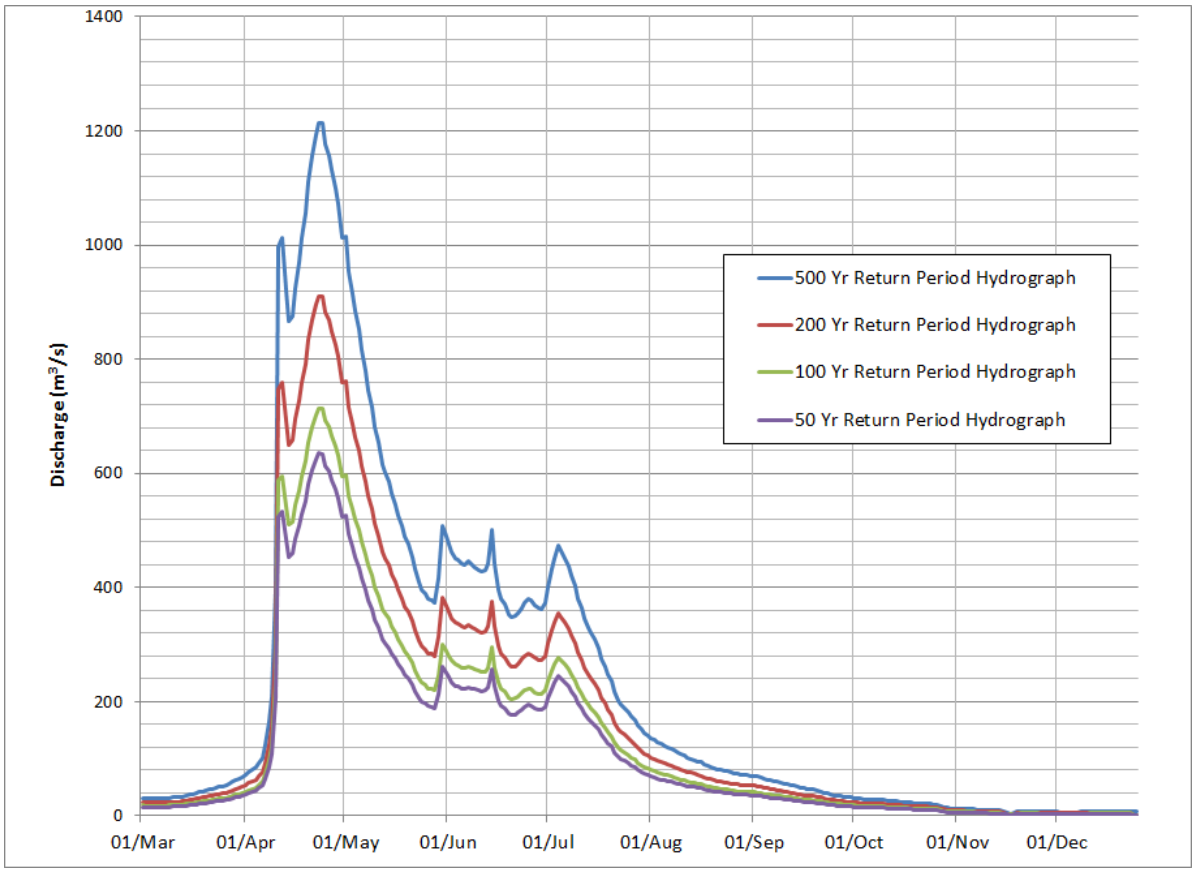
At the proposed dam site, the valley is 45 m (150 feet) deep and 550 m (1,800 feet) wide. The river has a well-defined channel with a width of 20 m (60 feet). The valley is incised into bedrock, which is a hard, cemented, siliceous, grey shale fractured to a considerable depth. Both abutments are mantled with lean and medium plastic glacial till. In the valley bottom, weak clay shale overlies the bedrock. Alluvial soil lies on top of the clay.

The embankment of the proposed Nesbitt Dam would be a zoned earthfill structure with very flat slopes averaging 9:1 to avoid problems with clay shale. A downstream sloping and horizontal filter and a positive cutoff would control seepage. Aside from the flat slopes, the embankment cross section would look very similar to that of the High Souris Dam. A typical cross section of the High Souris Dam is shown on Figure 12.2.

A 9-bay, 74.4 m (244 ft) wide gated concrete chute spillway would be used to pass the design flood. A 619 m (2,030 ft) long concrete conduit, with a horseshoe shaped cross section, would be used to pass the riparian flow. The general arrangement of the proposed project would be very similar to that of the conceptual arrangement for the High Souris Dam as shown on Figure 12.1.

The design hydrographs of the Souris River at Wawanesa documented in Section 5.4 were used to represent the Nesbitt Dam inflows. The hydrographs for the return periods from 50 years to 500 years are shown on Figure 12.7.

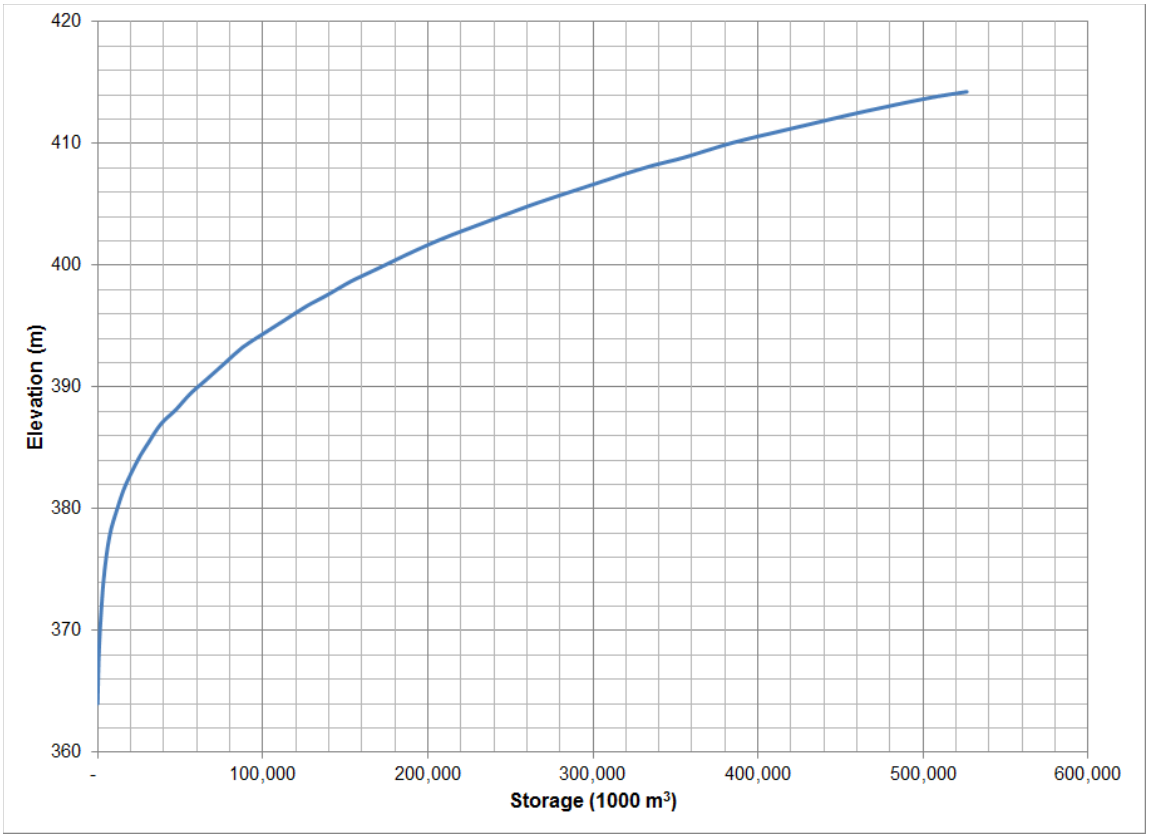
Figure 12.7
Design Hydrographs for Nesbitt Dam



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

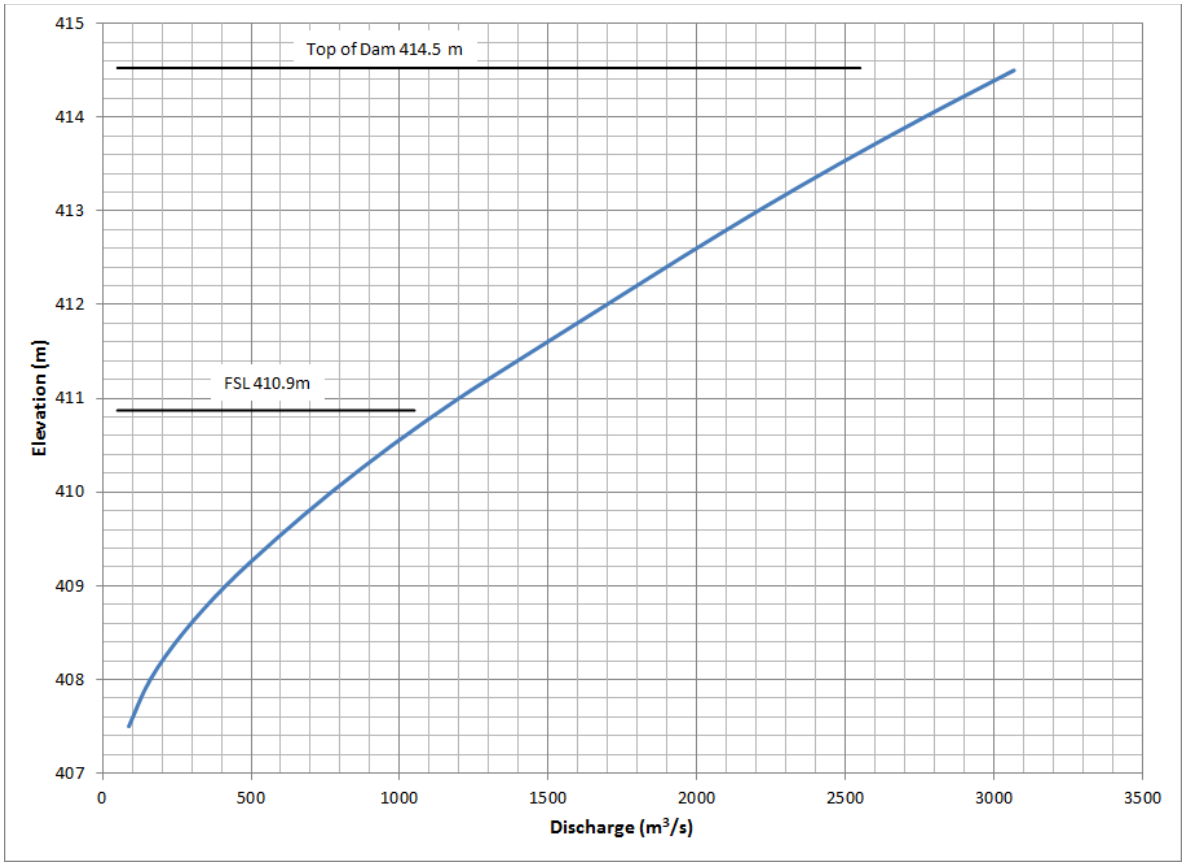
The stage-storage curve for the Nesbitt Dam shown on Figure 12.8 was taken from PFRA, 1989.

Figure 12.8
Nesbitt Dam Stage-Storage Curve



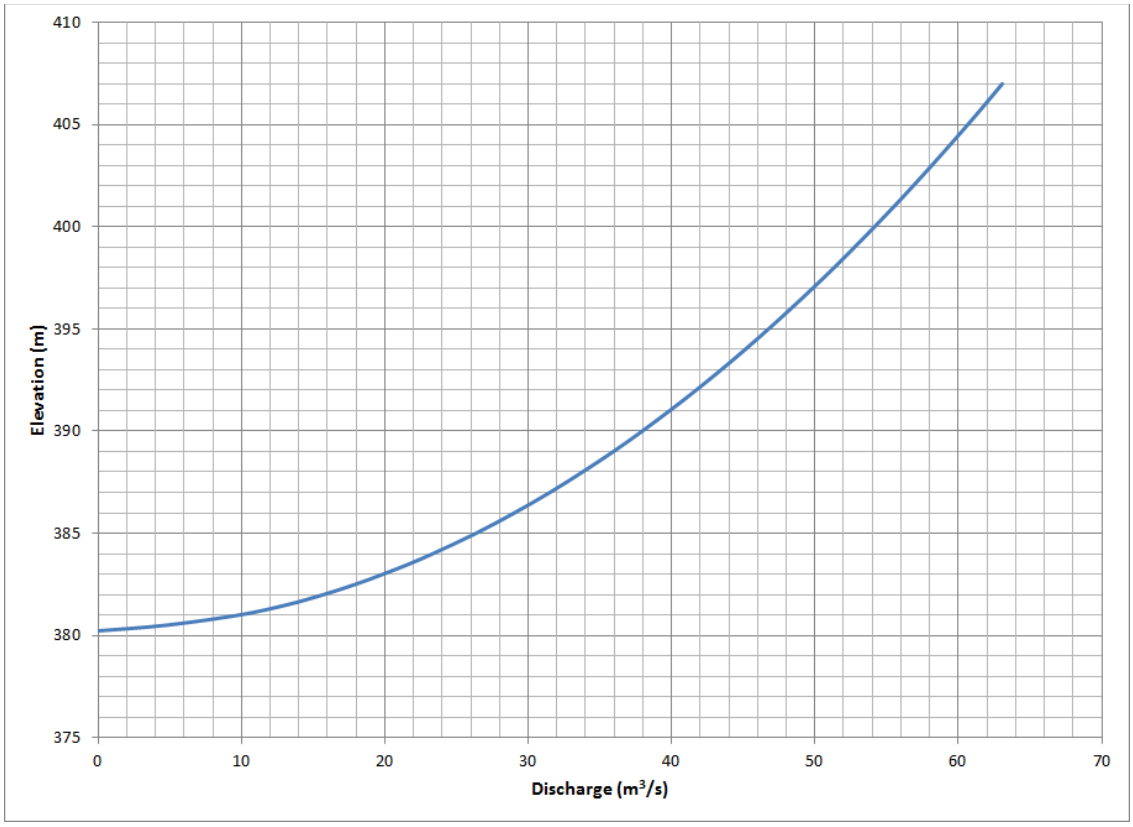
The stage-discharge rating curves for the overflow spillway and low-level conduit are shown in Figures 12.9 and 12.10, respectively.

Figure 12.9
Nesbitt Dam Overflow Spillway Stage-Discharge Curve



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Figure 12.10
Nesbitt Dam Low Level Conduit Spillway Rating Curve



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

12.3.4 Model Results

12.3.4.1 High Souris Dam

Results at High Souris Dam

The design hydrographs of the Souris River at Souris for return periods of 1:50 years to 1:500 years were routed through the High Souris Reservoir. Reservoir routing was performed using a level pool routing program and was based on the outlet rating curves and storage curves described in Section 12.3.2. The reservoir outflows were then routed down the Souris River and the Assiniboine River using the Muskingum models discussed in Section 8.2.3.

The peak discharge of the Souris River in the design hydrographs occurs earlier than that of the Assiniboine River. If the gates at the High Souris Dam are regulated solely for the purpose of reducing the peak flow on the Souris River, subsequent releases from the reservoir would be greater than the natural flow at the later time. The higher flows, if coinciding with the peak on the Assiniboine River would result in an increased peak discharge at Portage. To reduce peak flows in the Assiniboine River, operation of the gates on the High Souris Dam would be delayed until after the Souris River peak had passed, such that the outflow holdback would coincide with the time of the peak on the Assiniboine River at Portage la Prairie.

The travel time of the outflow peak from the High Souris Reservoir to Portage was estimated to be four days. The magnitude of the outflow holdback was determined through trial and error, so that the peak at Portage was minimized while the level of the High Souris Reservoir was less than the maximum permissible with appropriate freeboard. The holdback operation utilized the extra storage of the reservoir that was otherwise not used, and adjusted the timing of the outflow peak that otherwise coincided with the peak of the Assiniboine River.

The operation of outflow holdback was based on the design hydrographs on the Assiniboine River at Portage and on the Souris River. In reality, these discharges would not be known with any certainty. To achieve maximum storage benefits, accurate forecasting of discharges on the Souris River and Assiniboine Rivers would be required. Since this can be difficult, the storage benefits derived in the study are conservative from a project viability perspective and are the most optimistic estimations of the maximum benefit for the High Souris Dam.

The results of the routing for the 50-year, 100-year, 200-year and 500-year flood events are shown in Figures 12.11 to 12-14.

Figure 12.11
Results of the 500-Year Flood Event Routed through the High Souris Dam

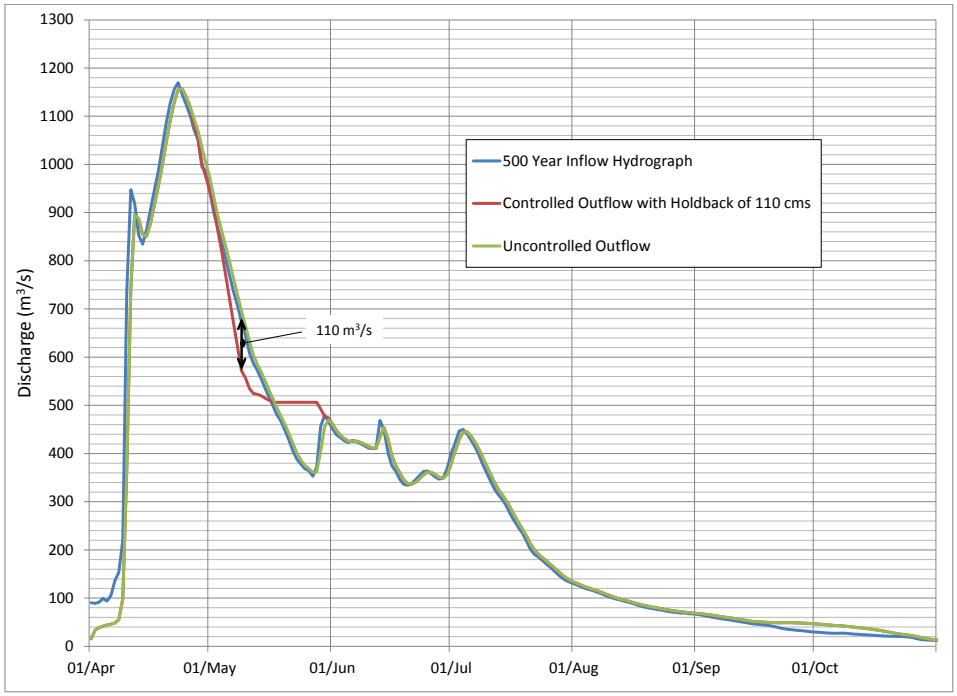


Figure 12.12
Hydrographs at High Souris Reservoir Outlet for a 200-Year Inflow Event, with and without Outflow Holdback

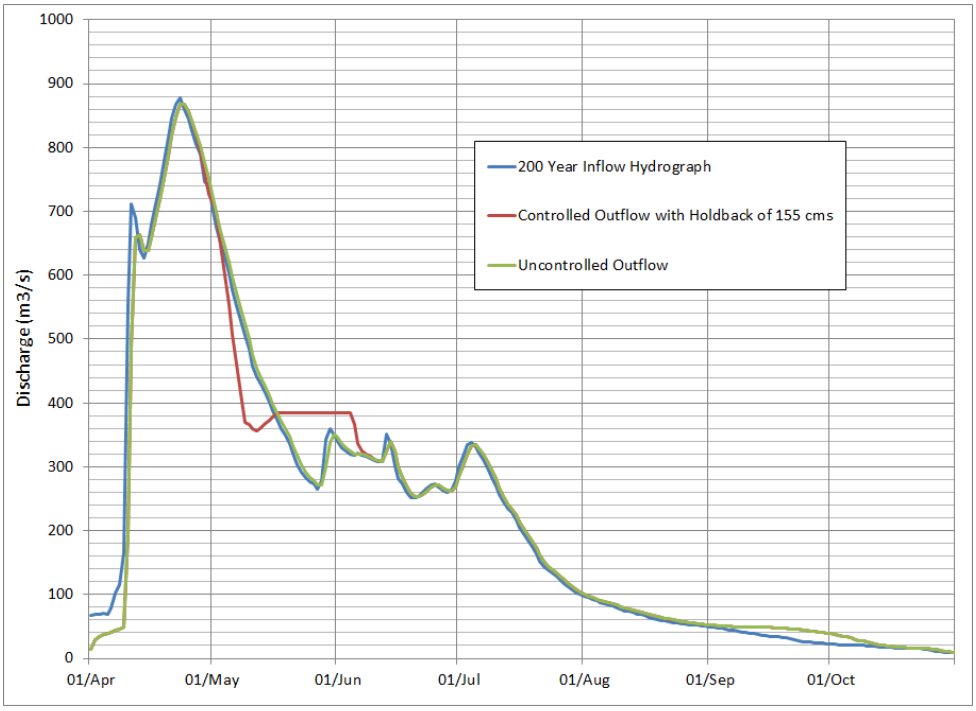


Figure 12.13
Hydrographs at High Souris Reservoir Outlet for a 100-Year Inflow Event, with and without Outflow Holdback

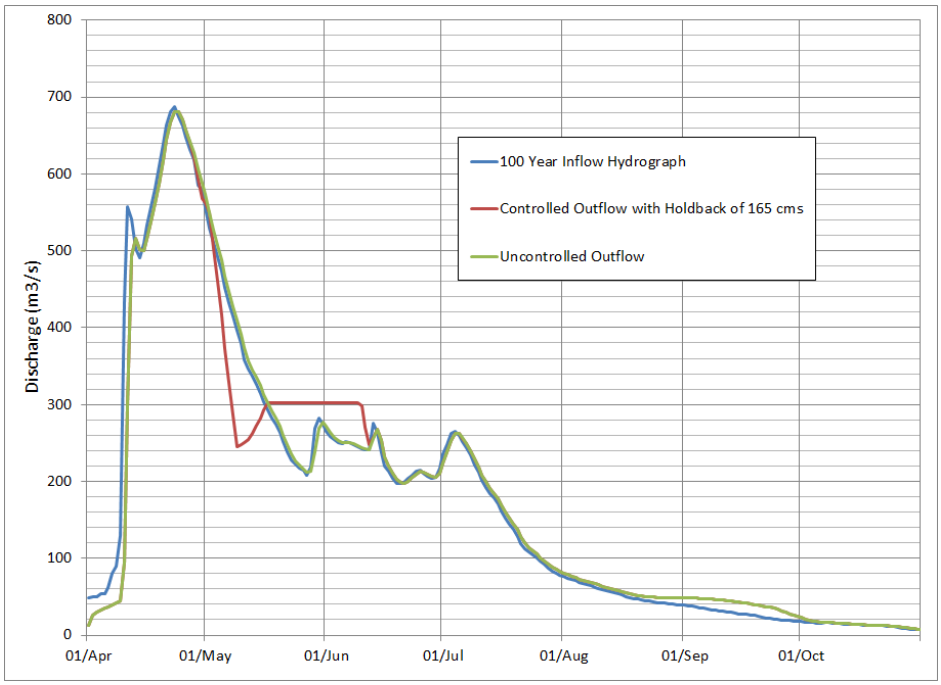
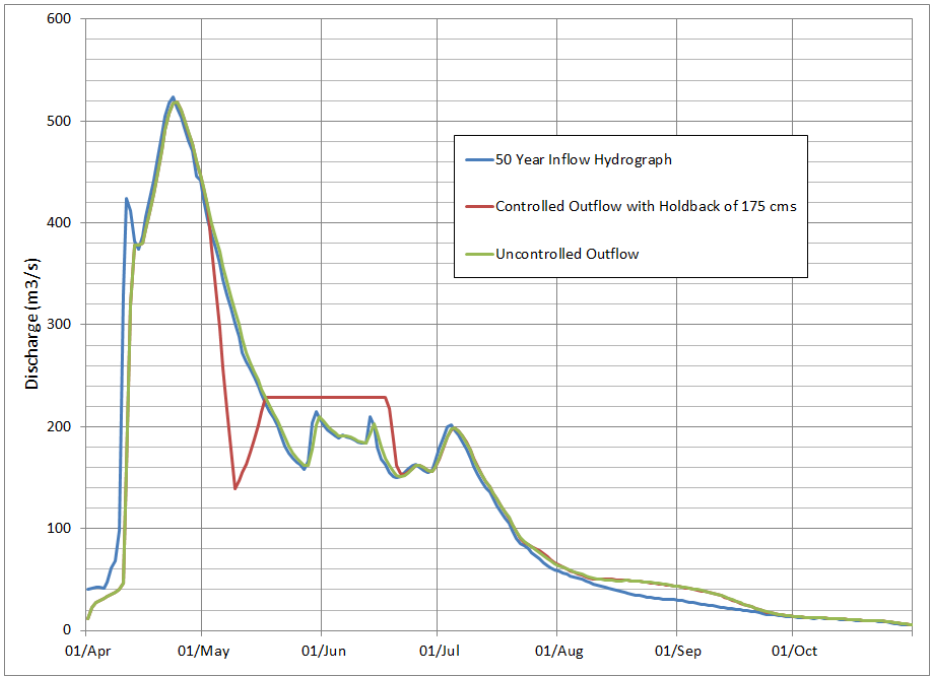


Figure 12.14
Hydrographs at High Souris Reservoir Outlet for a 50-Year Inflow Event, with and without Outflow Holdback



As shown in Figure 12.11, the outflow holdback resulted in a maximum outflow reduction of 110 m^3/s (3,880 cfs) to the 500-year flood hydrograph at the reservoir, and occurred during the time of peak in the Assiniboine River.

The maximum water level on the High Souris Reservoir, with controlled holdback, was computed to be 426.11 m (1,398 ft), or 1.2 m (4 ft) below the top of the dam during the 500-year event, as shown on Figure 12.15. This is equivalent to a water level rise of approximately 10 m (33 ft) in the reservoir during the flood. Figures 12.16 to 12.18 also illustrate the computed reservoir water levels for the 50-year, 100-year and 200-year flood events.

Figure 12.15
Computed Water Level during the 500-Year Event in the High Souris Dam Reservoir

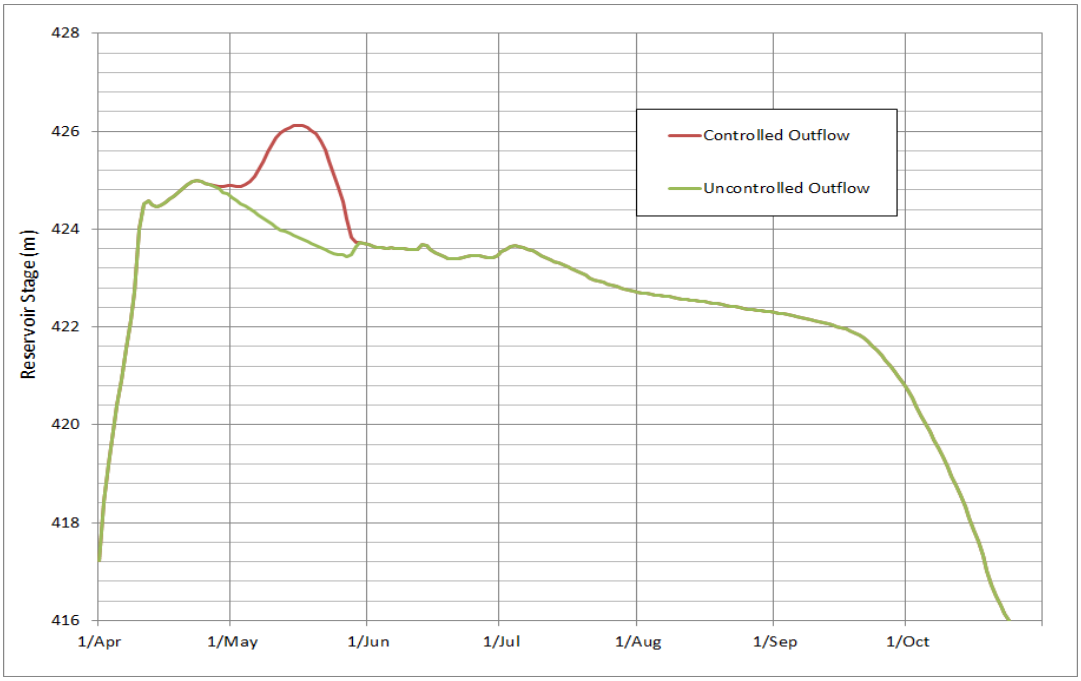


Figure 12.16
Water Elevation of High Souris Reservoir with and without Outflow Holdback, under 200-Year Inflow Event

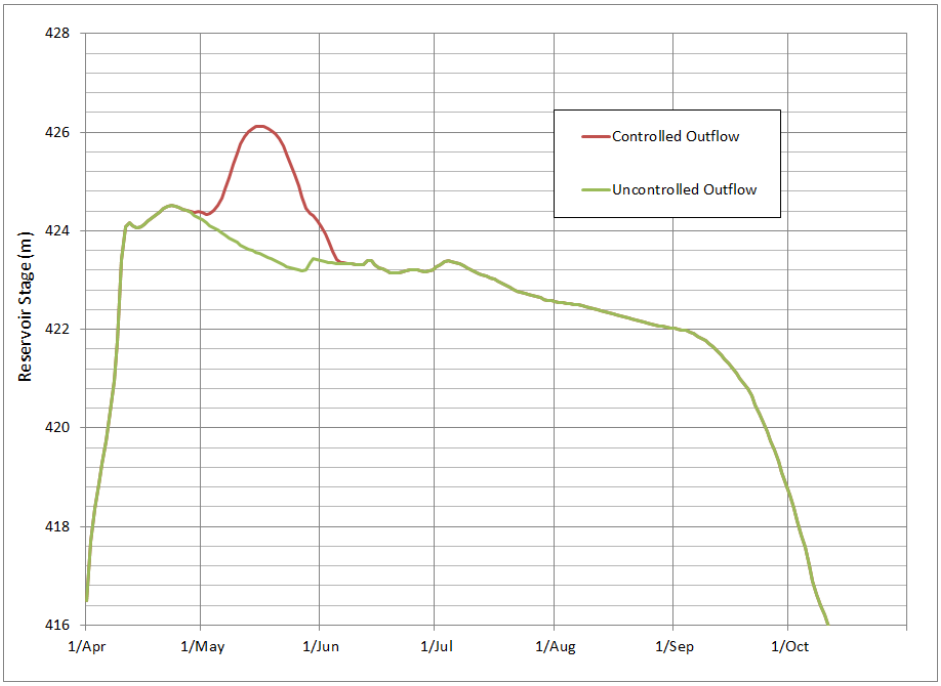


Figure 12.17
Water Elevation of High Souris Reservoir with and without Outflow Holdback, under 100-Year Inflow Event

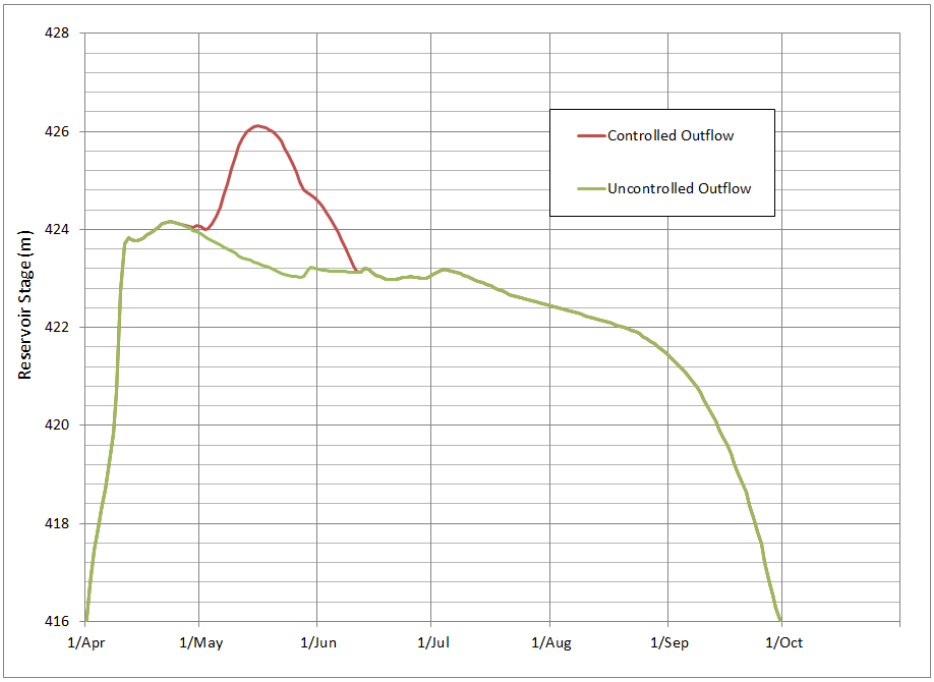
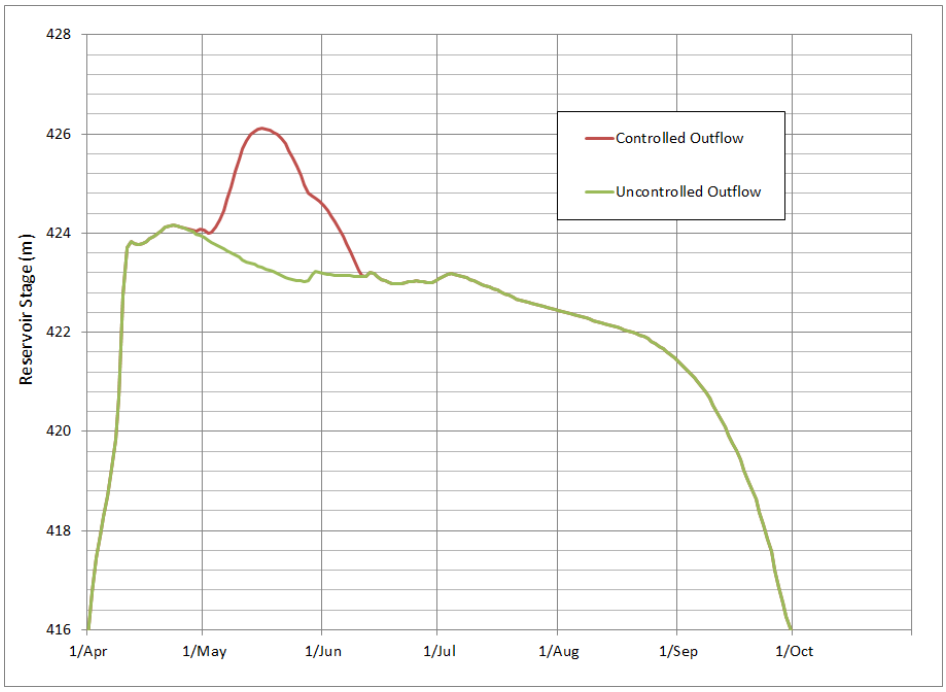


Figure 12.18
Water Elevation of High Souris Reservoir with and without Outflow Holdback, under 50-Year Inflow Event



Results at Portage la Prairie

The outflow from the High Souris Reservoir was routed to Portage la Prairie using the Muskingum River routing models of the Souris River and the Assiniboine River discussed in Section 8.2.3. Figures 12.19 to 12.22 illustrate the computed hydrograph at Portage with and without the effect of the High Souris Dam during the various flood events.

Figure 12.19
Muskingum Routing Results of a 500-Year Event at Portage with and without High Souris Dam

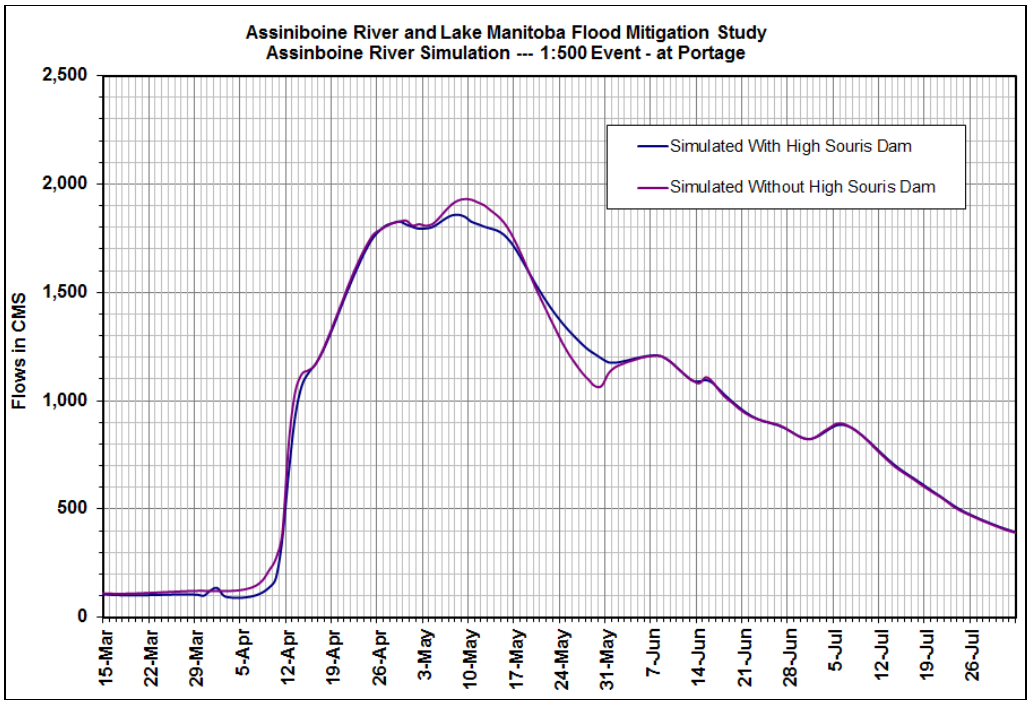


Figure 12.20
Muskingum Routing Results of a 200-Year Event at Portage with and without High Souris Reservoir

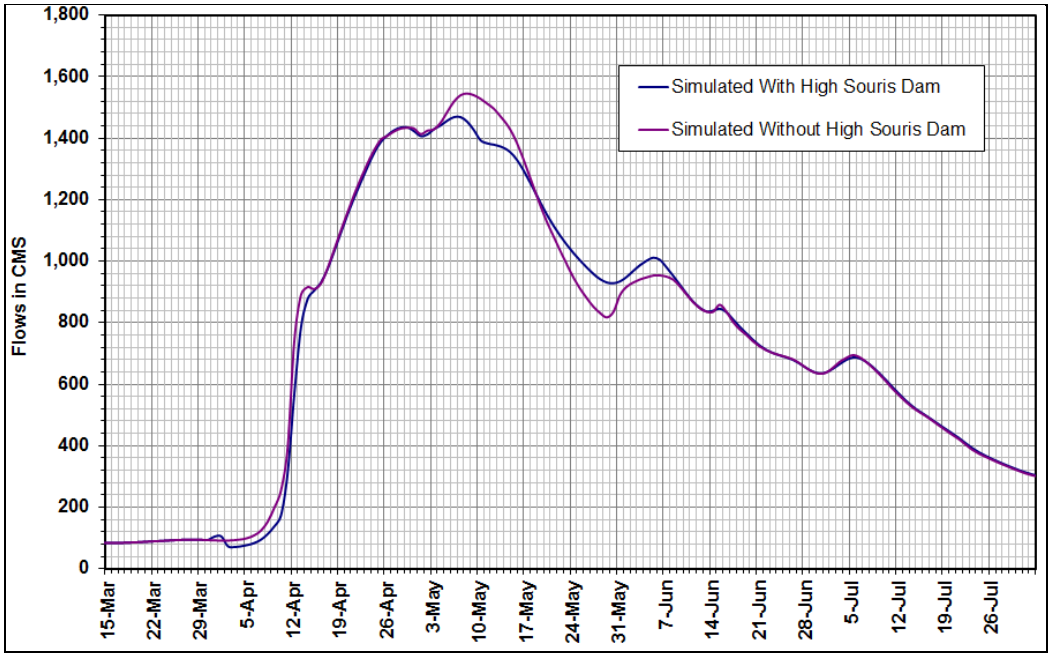


Figure 12.21
Muskingum Routing Results of a 100-Year Event
at Portage with and without High Souris Reservoir

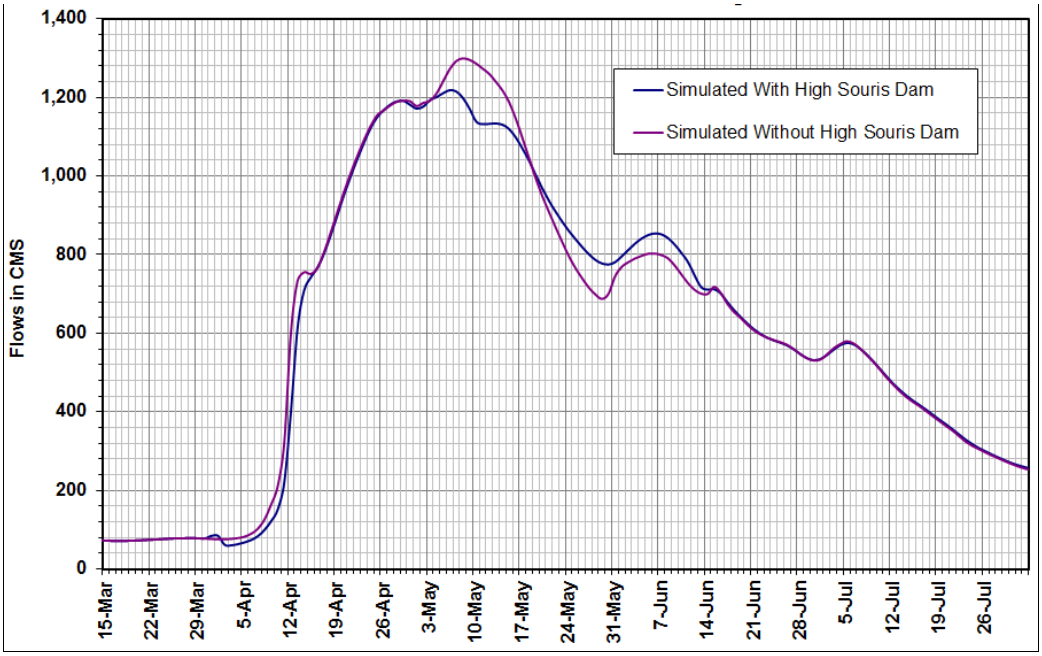
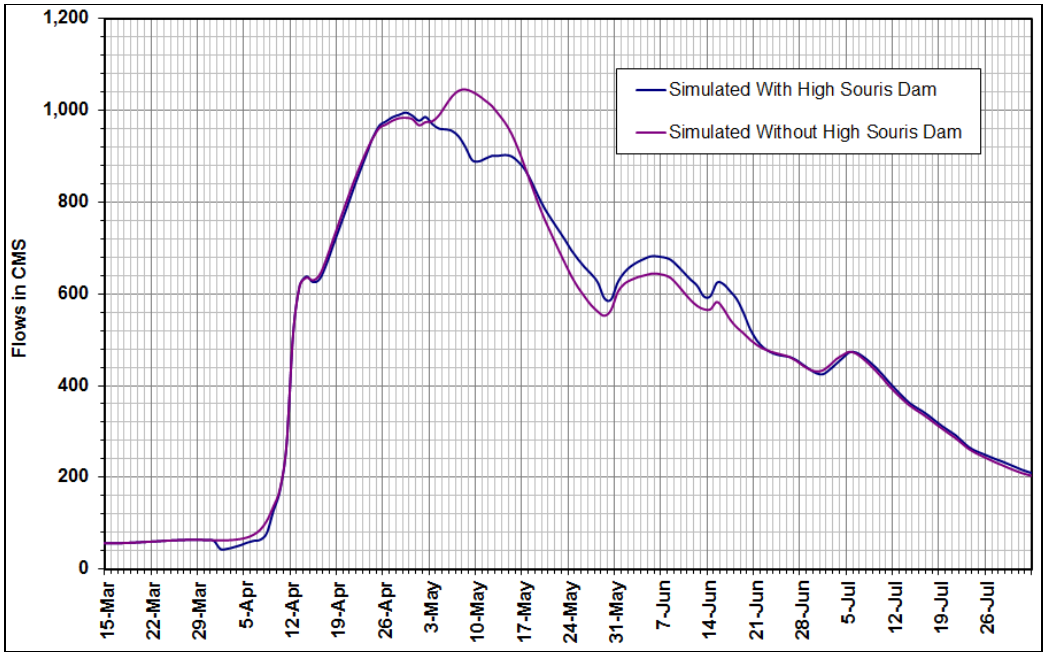


Figure 12.22
Muskingum Routing Results of a 50-Year Event
at Portage with and without High Souris Reservoir



As shown on Figure 12.19, the simulated peak discharge of the Assiniboine River at Portage la Prairie without the dam for the 1:500 year event is 1,931 m^3/s (68,193 cfs). The corresponding peak discharge with the High Souris Dam, with outflow holdback operation, is 1,859 m^3/s (65,652 cfs). Based on the results of the routings, the peak flow reduction capacity of the High Souris Dam was calculated for the 1:50 to 1:500 year events.

As discussed previously, the discharge reduction values are conservative and are considered to be the most optimistic estimations of the maximum benefit that the High Souris Dam can provide based on the assumed holdback operation. Furthermore, to achieve the maximum benefits with the gate control outlined above, it is necessary that the reservoir be completely empty at the beginning of the flood. It was assumed that sediment trapping and retention within the reservoir would not significantly affect live storage. Additionally, at this conceptual phase, optimization of the reservoir outflows and levels after the flood was not incorporated into the analysis. Careful consideration would be required to determine what operating conditions are necessary to empty the reservoir without causing significant artificial flooding downstream during the summer, fall and winter months due to increased outflows from the dams. Based on experience with the Shellmouth Dam, it was evident that a strict set of operating rules would be necessary for the High Souris Dam (and any other large storage reservoir), which would limit the maximum benefits of the dam. On this basis, it was estimated based on engineering judgment that 75% of the maximum peak flow reduction that could be achieved by the dam would best represent the actual peak flow reduction capability of the reservoir. Further studies beyond this conceptual phase analysis would be necessary to determine if this 75% value is achievable. The results of the High Souris Dam analyses at Portage are summarized in Table 12.1.

Table 12.1
Peak Flow Reduction – High Souris Dam at Portage

Return Period (year)	Reduction in Peak at Portage due to High Souris Dam (m^3/s)	Percent Reduction in Peak at Portage due to High Souris Dam (%)
50	38	4
100	58	5
200	58	4
500	54	3

Note: Flows can be converted to cfs by multiplying metric values by 35.3.

12.3.4.2 Nesbitt Dam

Results at Nesbitt Dam

The design hydrographs of the Souris River at Nesbitt Dam for return periods of 1:50 years to 1:500 years were routed through the Nesbitt Dam Reservoir. Reservoir routing was performed using a level pool routing program and was based on the outlet rating curves and storage curves described in Section 12.3.3. The reservoir outflows were then routed down the Souris River and the Assiniboine River using the Muskingum routing models discussed in Section 8.2.3.

Similar to the High Souris Reservoir, the gates on the Nesbitt Dam were operated to create an outflow holdback to reduce peak flows in the Assiniboine River. The travel time of the outflow peak from Nesbitt Dam to Portage was estimated to be two days. The magnitude of the outflow holdback was determined based on the same trial and error approach used for the High Souris Reservoir discussed in Section 12.3.4.1.

The results of the routings for the 500-year, 200-year, 100-year and 50-year floods through the reservoir are shown on Figures 12.23 to 12.26. Figures showing the results for routing of the floods of other return periods are also provided below.

Figure 12.23
Results of the 500-Year Flood Event Routed through the Nesbitt Dam

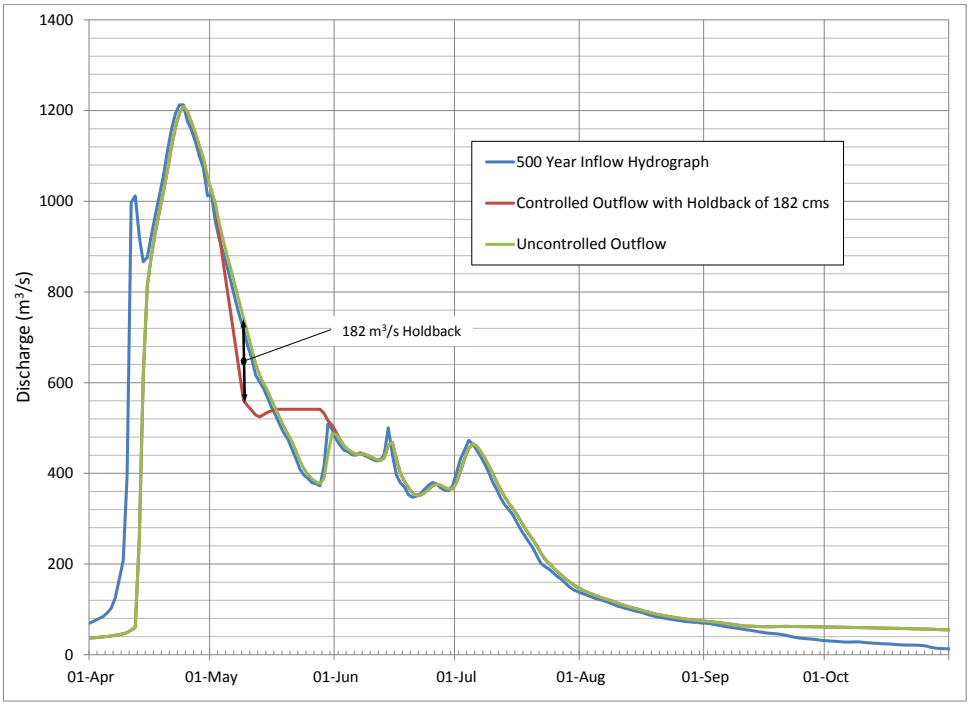


Figure 12.24
Results of the 200-Year Flood Event Routed through the Nesbitt Dam

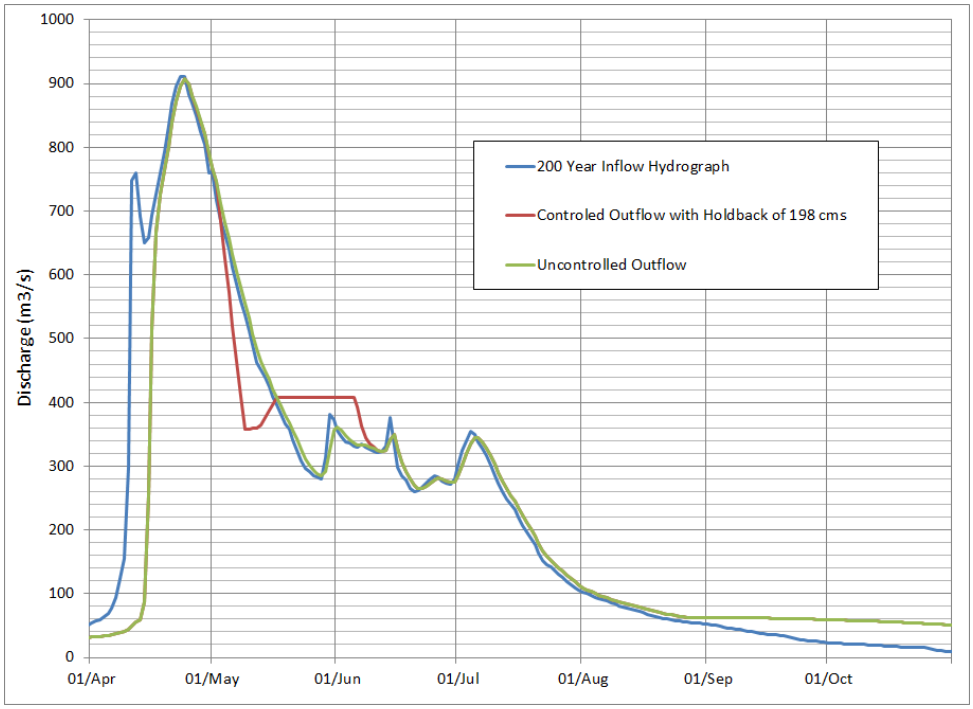


Figure 12.25
Results of the 100-Year Flood Event Routed through the Nesbitt Dam

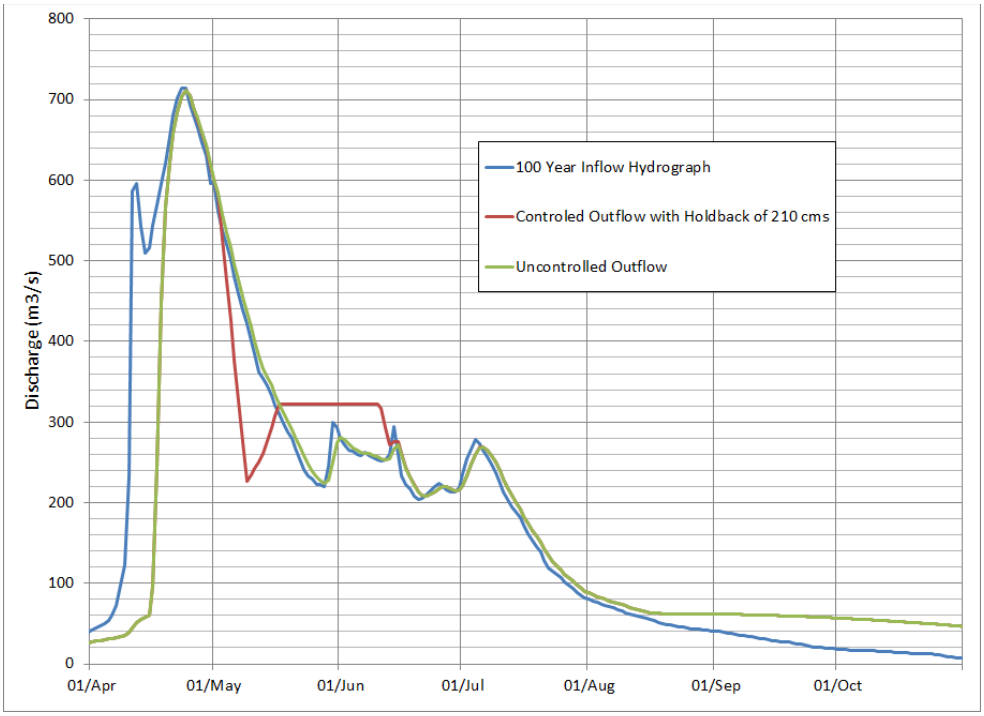
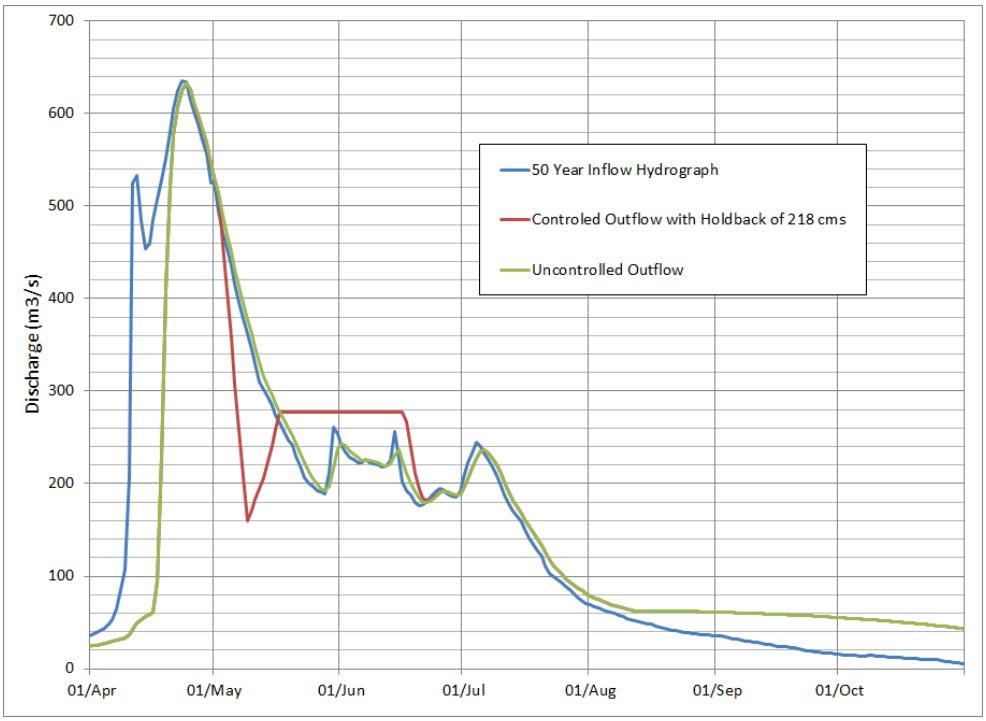


Figure 12.26
Results of the 50-Year Flood Event Routed through the Nesbitt Dam



As shown in Figure 12.23, the outflow holdback resulted in a maximum outflow reduction of 182 m^3/s (6,430 cfs) to the 500-year flood hydrograph at the reservoir, and occurred during the time of peak in the Assiniboine River.

The maximum water level on the Nesbitt Dam Reservoir, with controlled holdback, was computed to be 413.11 m (1,356 ft), or 1.2 m (4 ft) below the top of the dam for the 500-year event, as shown on Figure 12.27. This is equivalent to a water level rise of approximately 30 m (100 ft) in the reservoir during the flood. Figures 12.28 to 12.30 illustrate the computed reservoir water levels for the 200-year, 100-year and 50-year flood events.

Figure 12.27
Computed Water Level during the 500-Year Event in the Nesbitt Dam Reservoir

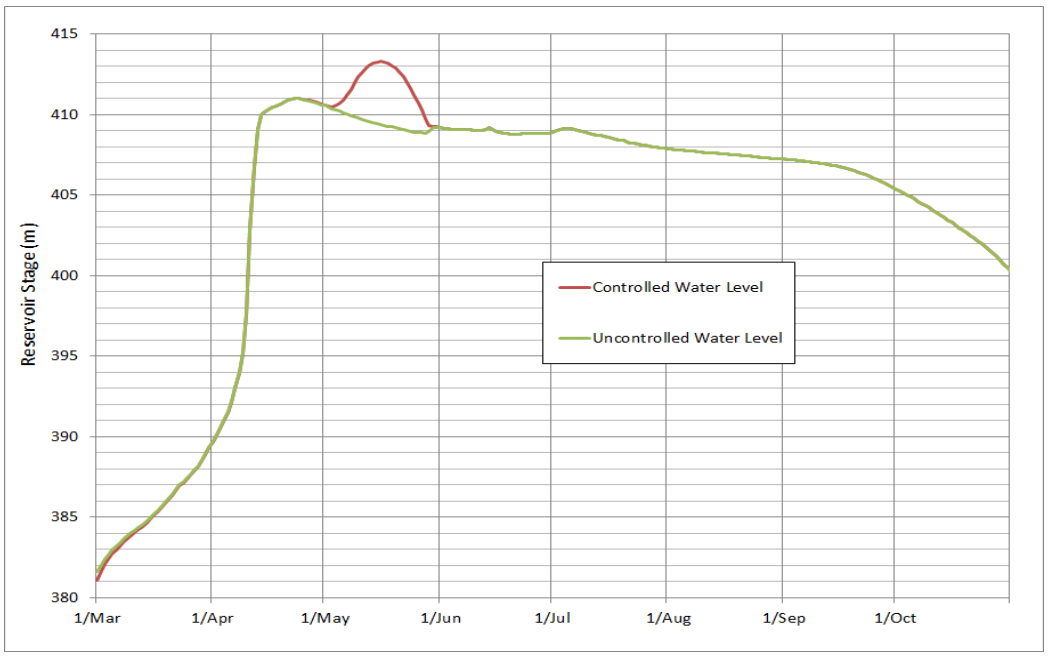


Figure 12.28
Computed Water Level during the 200-Year Event in the Nesbitt Dam Reservoir

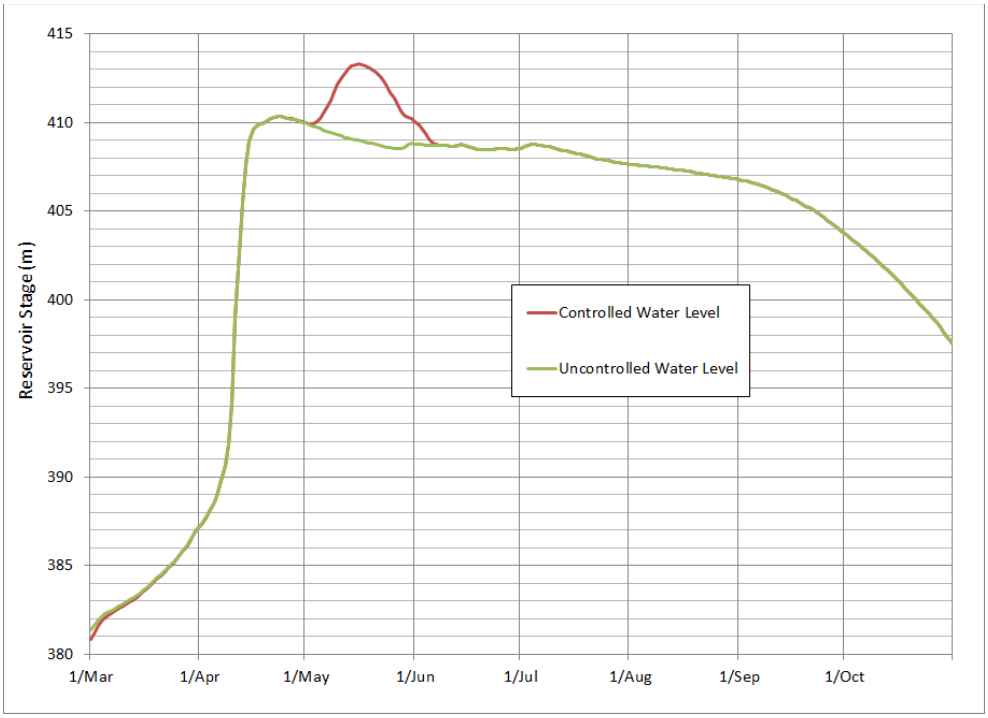


Figure 12.29
Computed Water Level during the 100-Year Event in the Nesbitt Dam Reservoir

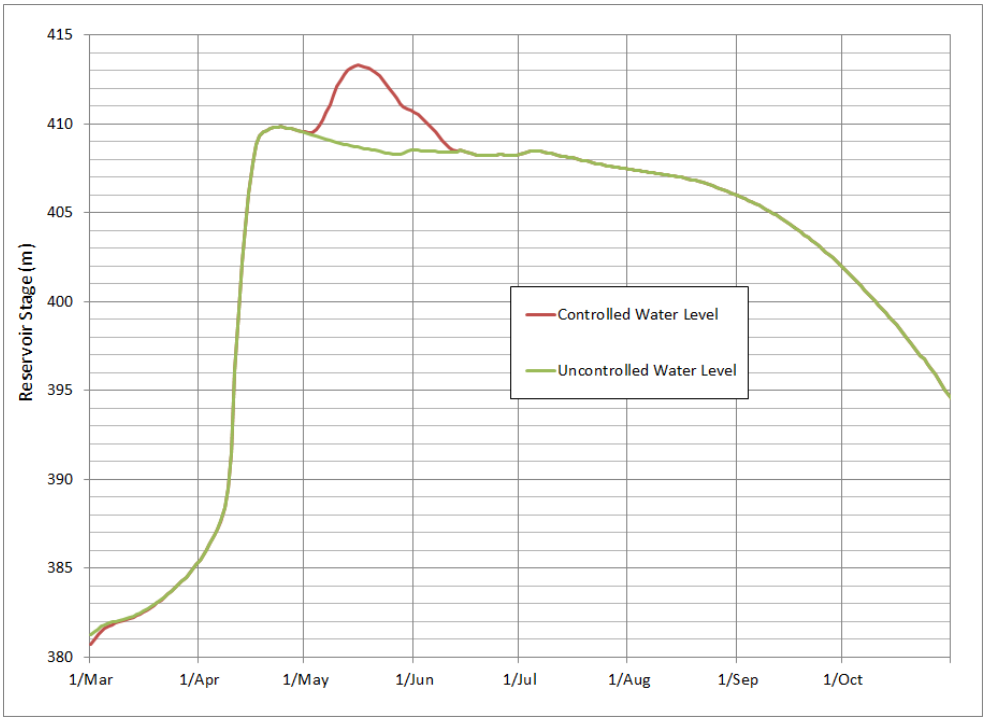
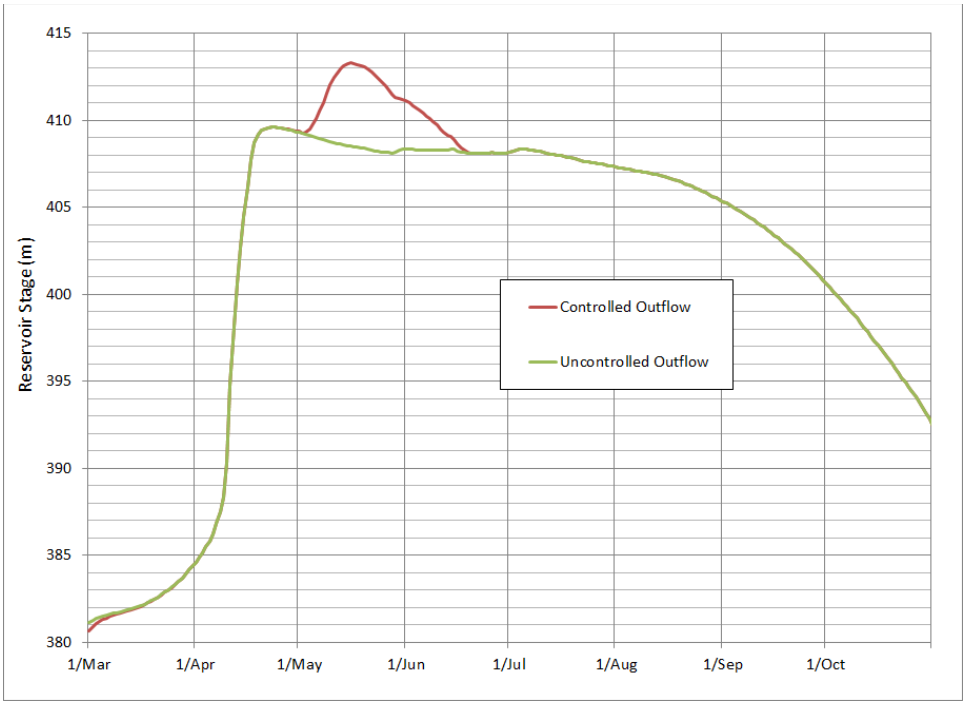


Figure 12.30
Computed Water Level during the 50-Year Event in the Nesbitt Dam Reservoir



Results at Portage la Prairie

The outflow from the Nesbitt Reservoir was routed to Portage la Prairie using the Muskingum River routing models of the Souris River and Assiniboine River discussed in Section 8.2.3. Figures 12.31 to 12.34 illustrate the computed hydrograph at Portage with and without the effect of the Nesbitt Dam during the 500-year, 200-year, 100-year and 50-year flood events.

Figure 12.31
Muskingum Routing Results of a 500-Year Event at Portage with and without Nesbitt Dam

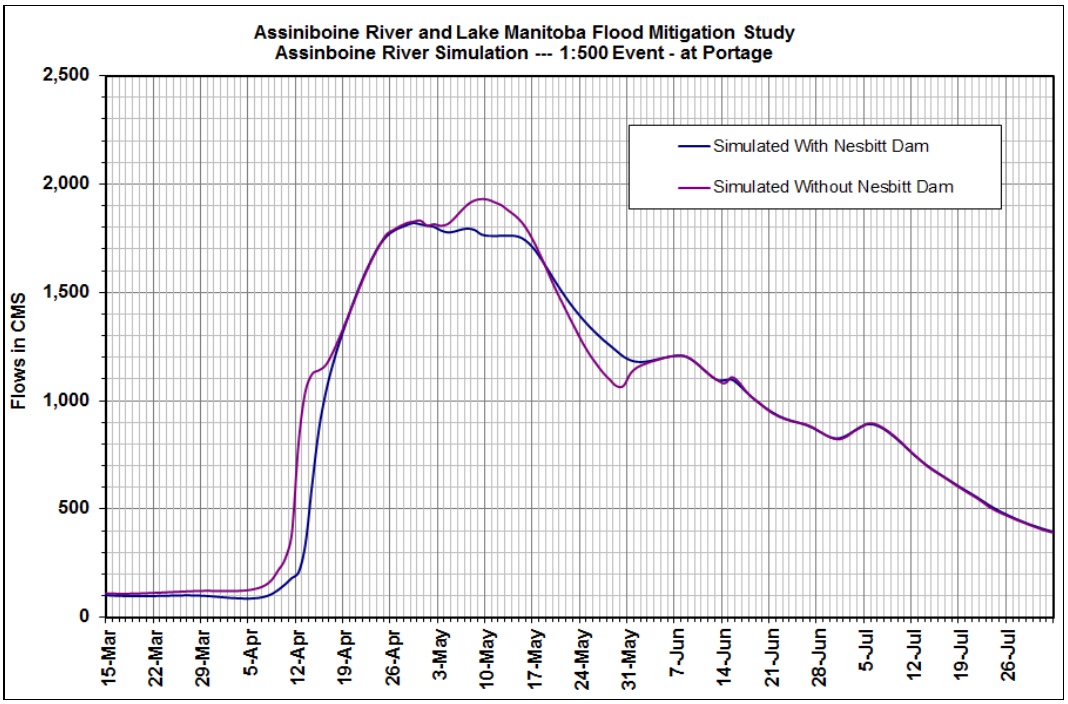


Figure 12.32
Muskingum Routing Results of a 200-Year Event at Portage with and without Nesbitt Reservoir

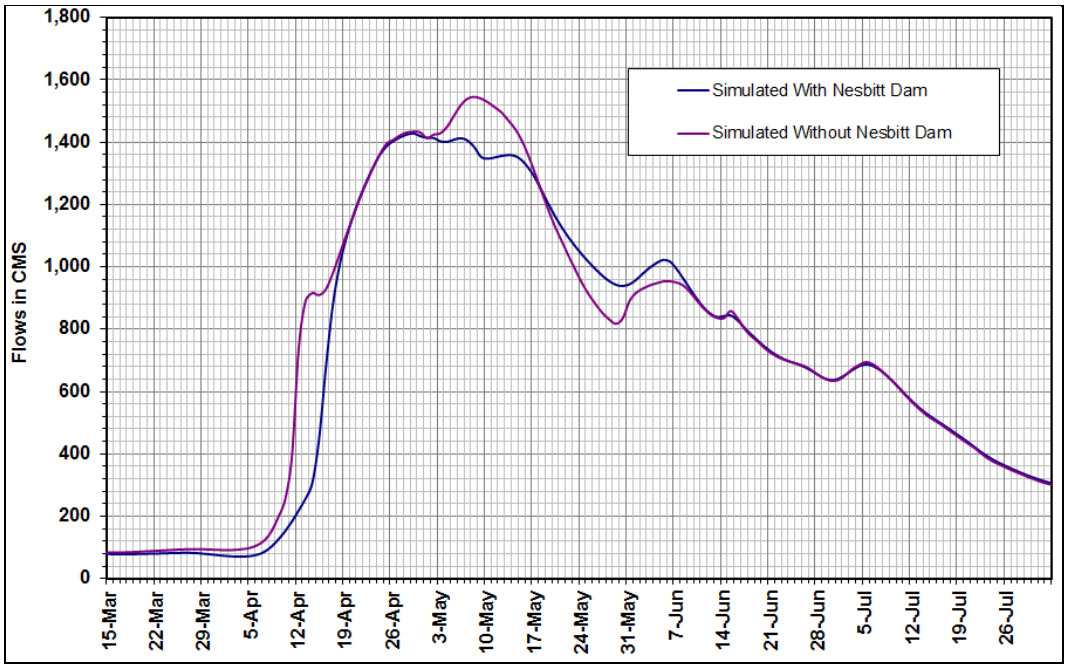


Figure 12.33
Muskingum Routing Results of a 100-Year Event
at Portage with and without Nesbitt Reservoir

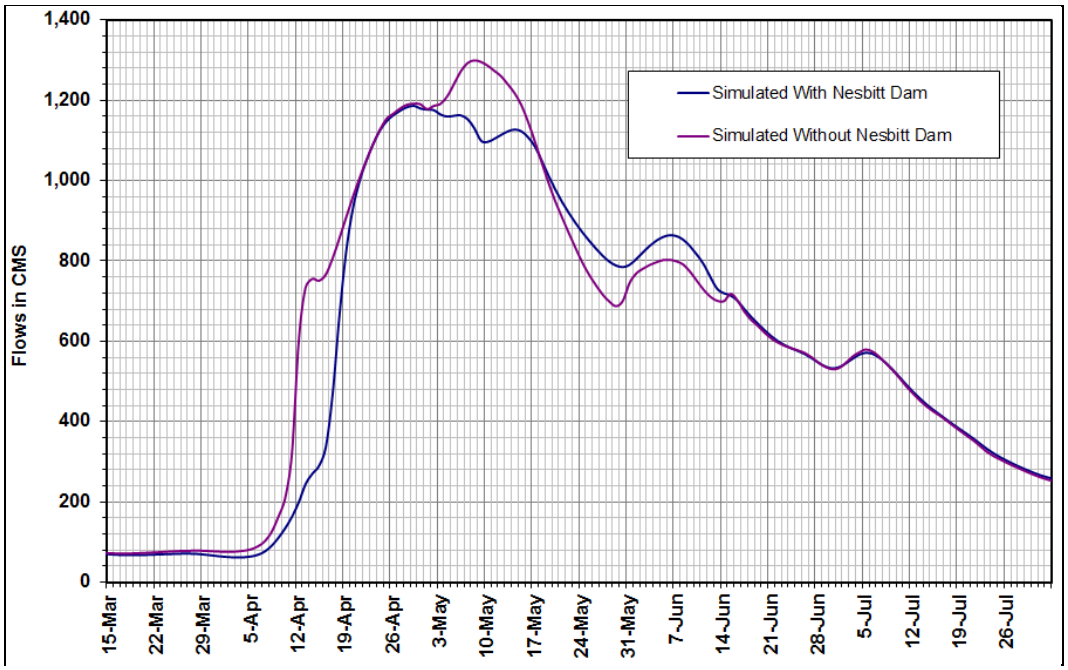
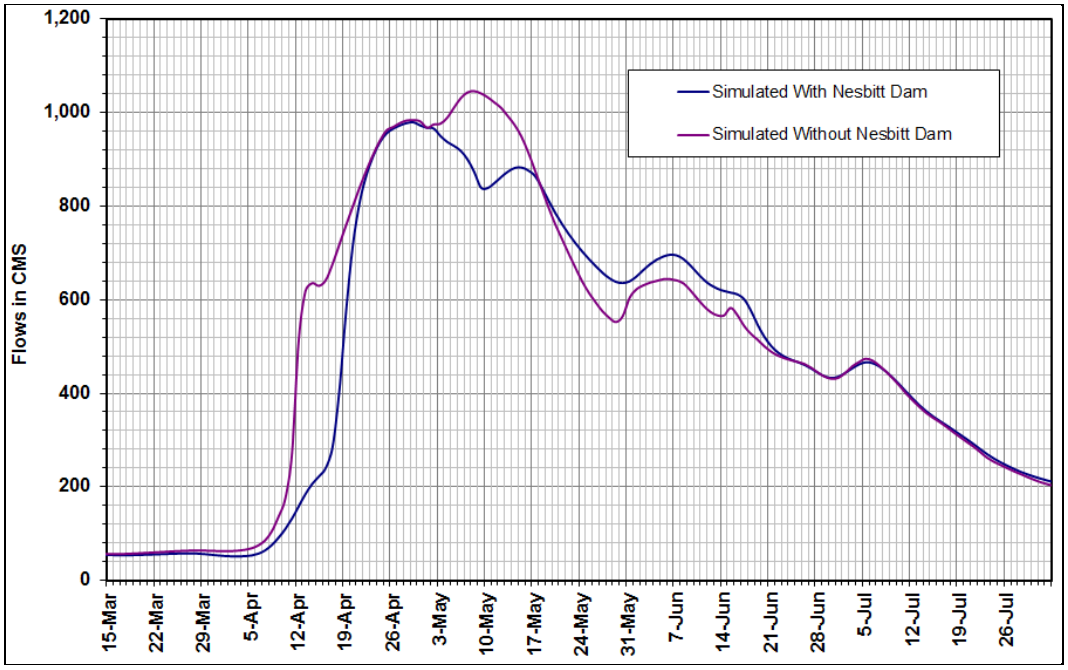


Figure 12.34
Muskingum Routing Results of a 50-Year Event
at Portage with and without Nesbitt Reservoir



As shown on Figure 12.31, the simulated peak discharge of the Assiniboine River at Portage la Prairie for the 1:500 year event, without the dam, is 1,931 m^3/s (68,193 cfs). The corresponding peak discharge with the Nesbitt Dam, with outflow holdback operation, is 1,821 m^3/s (64,299 cfs). Based on the results of the routings, the peak flow reduction capacity of the Nesbitt Dam was estimated for the 1:50 to 1:500 year events. As discussed previously, the reduction in the peak flows are conservative from a project viability perspective and are the most optimistic estimations of the maximum benefit, which the Nesbitt Dam can provide due to holdback operation considerations. The results at Portage, considering the uncertainties with being able to operate the gates to achieve the maximum benefits as discussed in Section 12.3.4.1, are summarized in Table 12.2.

Table 12.2
Peak Flow Reduction Capacity of Nesbit Dam at Portage

Return Period (year)	Reduction in peak at Portage due to Nesbitt Dam (m^3/s)	Percent reduction in peak at Portage due to Nesbitt Dam (%)
50	50	5
100	82	6
200	88	6
500	83	5

Note: Flows can be converted to cfs by multiplying metric values by 35.3.

12.3.5 Costs

A cost estimate was developed for the two large storage reservoir projects, which was taken from the PFRA (1989) report. At the time, costs were estimated in 1986$ and included the main embankment, the spillway outlet works, reservoir damages and clearing and care of river during construction. Reservoir damages included costs for raising and relocation of roads, transmission line relocation, and relocation of farms and demolition of abandoned buildings.

Table 12.3 lists the project costs for the High Souris and Nesbit dams. The 1986 costs were escalated to 1997$ using the Canadian Electric Utility Construction Price Index (CEUCP). The 1997 costs were further escalated to 2014$ using an assumed escalation rate of 5% per year based on Consumer Price Index data from Statistics Canada, as well as experience with other

similar, recent KGS Group projects. The 1997 cost index value was the last year that the CEUCPI was available.

The flood control benefits determined in this study assumed vertical lift gates to be operated to maximize storage benefits. The project costs that were determined by PFRA did not include gates on the High Souris Dam. The cost of gates for this reservoir was determined using a unit rate of $ 25,000/m^2, determined based on KGS Group's experience with the recent Pointe du Bois Project. Eight additional spillway piers would also be necessary on the High Souris Dam due to the addition of the nine gates. The cost for the additional spillway piers was based on an assumed 1.5 m width to each pier and a unit rate of $3,000/$m^3$ of concrete, also determined based on KGS Group's experience with the recent Pointe du Bois Project. The corresponding costs to include the additional gates and additional spillway piers on the High Souris Dam are included in Table 12.3. For the Nesbitt Dam, it was assumed that the difference in costs between the radial gates originally considered in the PFRA studies and the vertical lift gates considered in the current study were negligible. Therefore, the Nesbitt Dam did not require an additional cost for gates.

Table 12.3
Project Cost Estimates

1986 Components	High Souris Dam	Nesbitt Dam
Land	$ 4,600,000	$ 2,200,000
Reservoir Clearing	$ 1,394,000	$ 3,485,000
Reservoir Damages	$ 2,310,000	$ 4,380,000
Care of Water	$ 300,000	$ 300,000
Earth Dam	$ 1,097,000	$ 15,129,000
Spillway	$ 4,348,000	$ 9,968,000
Conduit	$ 833,000	$ 4,404,000
Sub Total Direct Cost (1986$)	$ 14,882,000	$ 39,866,000
Sub Total Direct Cost (2014$)	$ 45,980,000	$ 123,172,000
Additional Components		
Gates	$ 14,247,000	
Spillway Piers	$ 7,392,000	
Allowance for Uncertainties (10%)	$ 6,762,000	$ 12,317,000
Total Direct Cost (2014$)	$ 74,380,000	$ 135,489,000
Environmental Mitigation (15%)	$ 11,157,000	$ 20,323,000
Indirect Costs (Engineering, Approvals, Administration and Owner's Costs) (20%)	$ 14,876,000	$ 27,098,000
Contingencies (30%)	$ 30,124,000	$ 54,873,000
Total Cost in 2014 Dollars	**$ 130,537,000**	**$ 237,783,000**

An allowance for "reservoir damages" was considered in the previous studies and included costs for raising and relocation of roads, transmission line relocation, relocation of farms and demolition of abandoned buildings. Detailed information on the items considered was not always available for each project. Further investigations beyond the scope of the current study are necessary to determine the exact amount and items considered in the previous studies and whether or not these values are significantly different today. To account for this high level of uncertainty, an additional allowance of 10% of the total direct cost was included in the estimation.

In addition to the costs outlined above, consideration must be given to the costs for easements and compensation, environmental assessment and licensing, and mitigation of environmental impacts. Due to the potential significant environmental impacts (discussed in Section 12.3.6) associated with the proposed dam projects, major environmental mitigation costs are expected; and, therefore, a cost allowance equivalent to 15% of the total direct cost was added to the estimate. As per the economic guidelines set out for the study and discussed in Section 7.0 of the report, an allowance of 20% of the total direct cost was also added to account for indirect costs such as engineering, approvals, administration and owner's costs, and a contingency allowance of 30% of the direct and indirect cost was added for uncertainty.

It should be noted that it is possible that the assumed proportion of public (crown) and private lands and the resulting land acquisition costs presented in this study (which were based largely on the 1972 SNBB values) could potentially be less than actual farmland market costs due to changes in proportion of land ownership since 1972. Consequently, the actual project costs for land acquisition could be greater than that estimated. Higher land costs would make the proposed storage project(s) even more costly and more difficult to justify on the basis of flood control benefits alone.

12.3.6 Environmental and Socio-Economic Impacts

Construction of the proposed dams and reservoirs will have substantial physical, biological and socio-economic environmental effects on the surrounding area. The reservoirs considered were previously studied in the late 1960s and early 1970s; and as such, the environmental issues were not addressed very well. To identify key environmental issues associated with the construction of the storage reservoirs, the study used the results of the more recent Ste. Agathe Detention Structure evaluated as part of the Flood Protection for Winnipeg (KGS Group, 2000c) and the Treherne Dam Pre-Feasibility Study (KGS Group and PMA, 2002). The results of these studies are representative of other large reservoirs in the prairie region; and, therefore, most issues identified in these studies are considered applicable to the reservoirs that were assessed for the current study.

The impoundment of the reservoirs will inundate large areas of land. Regulated releases of flow will change the flow regime on the downstream river and potentially affect fish and fish habitat

and block fish passage and also affect the navigability of the rivers. Prior to construction, approvals would be required under the *Canadian Environmental Assessment Act* (CEAA, 2012), *Fisheries Act*, *Navigation Protection Act*, and the *Manitoba Environment Act*. The project would also need to conform to many other federal and provincial acts and regulations including but not limited to the *Environmental Protection Act*, *Wildlife Act*, *Endangered Species and Ecosystems Act*, *Heritage Resources Act*, and *Water Protection Act*.

While a comprehensive Environmental Impact Assessment would be required to obtain the necessary project approvals, for the purpose of this study a high level environmental scoping was conducted to identify the most probable issues and effects.

The potential environmental issues and effects associated with construction of the studied dams and reservoirs include:

- Loss of substantial terrestrial habitat and agriculture land:
  - Construction of the High Souris and Nesbitt reservoirs is expected to inundate approximately 1,400 and 2,900 ha (3,500 and 7,100 ac) of land, respectively.
  - The majority of the land is under agricultural production and would require substantial land acquisition and relocation of buildings and farmsteads. There would be strong opposition from land owners directly affected and in the surrounding area that would substantially delay or stop the approval process.
- Alteration of aquatic life and habitat along the rivers:
  - Construction of the dam will obstruct the movement of fish and reduce or eliminate access to spawning.
  - Formation of the reservoir will change the riverine habitat into lake habitat, which will change the habitats and natural productive capacity of different fish species, resulting in increased sediment and debris accumulation upstream of the dam and deprivation of sediment downstream of the dam.
  - The reservoir and dams will alter spring flows downstream of the structures, which can adversely affect channel maintenance and fish spawning habitat.
- Impede navigation along the rivers:
  - While only the Assiniboine River is on the list of scheduled waters under the *Navigation Protection Act*, the Souris River is used for recreational purposes.
- Quality of water both in and downstream of the new reservoir:

- Impoundment and flooding of terrestrial habitat can increase mercury levels in the water and bioaccumulate in fish.
- Impoundment and flooding of agricultural land will increase nutrient levels in the water as they leach from the agricultural land. Retention of the water in the reservoir combined with higher nutrient loads can cause algal blooms during summer.
- Oxygen depletion can occur due to degradation of organic material (dead algae) and water releases from the depth of a reservoir.

- Ground water levels in the immediate vicinity of the impoundment:
  - Reservoir water may seep into groundwater supplies and over time raise the groundwater level affecting yields, salinity and drainage on surrounding agricultural fields.

12.3.7 Conclusions

Based on the results of the analysis documented in this report, the following conclusions have been made:

- Two large dams and reservoirs on the Souris River have been investigated in this study having project costs (2014$) of $131 million for the High Souris Dam and $238 million for the Nesbitt Dam.
- Outflow control using gates at the dams is shown to be less effective than dams on the Assiniboine River due to the timing of the peak discharges on the Souris River and the Assiniboine River. If gate control is used for minimizing peak flows on the Souris River, the subsequent releases after the reservoir has filled can result in the Assiniboine River peak in Portage being higher than without control at the Souris River dams. Therefore, at this conceptual phase of analysis, it was assumed that the gates on the Souris River Reservoir would be operated after the peak on the Souris River has passed, to coincide with the Assiniboine River peak, in order to maximize benefits downstream and reduce the Assiniboine River peak in Portage.
- The Nesbitt Dam provides the largest peak flow reduction on the Assiniboine River at Portage (5% to 6%), followed by the High Souris Dam (3% to 5%). However the High Souris Dam is more viable than the Nesbitt Dam based on the cost of the options per one m^3/s of flow reduction.
- Peak water levels in the Nesbitt Dam Reservoir would rise by approximately 30 m (100 ft) and flood an area of approximately 2,900 ha (7,100 ac). Peak water levels in the High Souris Dam Reservoir would rise by approximately 10 m (33 ft) and flood an area of approximately 1,400 ha (3,500 ac).
- Construction of the proposed dams and reservoirs is not considered a feasible option for flood protection, as they will have substantial environmental effects on the surrounding area and will receive very strong public opposition. The most substantial environmental issues and effects associated with construction of the proposed dams and reservoirs

includes loss of agricultural land, blocking of fish passage, altering fish habitat associated with the change in flow regime and impeding navigation of the rivers. The loss of agricultural land will receive substantial opposition that would delay or stop the approval process.

- The social and environmental impacts together with the relatively high costs of the projects would likely render the storage reservoir alternatives difficult to complete. On this basis, the two projects have not been identified as preferred alternatives for the current study. These conclusions are consistent with comments received at the technical workshop undertaken at the beginning of this study when a majority of experts and stakeholders present at the workshop suggested that most of the large storage reservoir projects should not be considered for this study due to environmental concerns and relatively high costs.

12.4 DIKES UPGRADES FOR SOURIS, MELITA AND WAWANESA

High water levels during the flood of 2011 triggered the emergency construction of temporary dikes in the communities of Souris, Melita and Wawanesa. Following the flood of 2011, a geotechnical review of the diking system in the three communities was conducted by KGS Group for the RM of Arthur and the towns of Souris, Melita and Wawanesa (KGS Group, 2013c), (KGS Group, 2013d), (KGS Group, 2013e) (KGS Group, 2013f). Commitments were subsequently made to upgrade flood protection in Souris and Melita to the highest flood on record (2011 Flood). In Souris, the 2011 Flood was estimated to have a return period of approximately 1:150 years; whereas in Melita, it was estimated to have a return period of approximately 1:180 years. These projects are now well underway and have nearly been completed. The costs of these projects were estimated at $8 to $10 million, with an expected completion date in 2015. For the current study, it was assumed that these dikes reflected current conditions in the communities and, therefore, they are referred to below as the “existing” dikes. In Wawanesa, the dikes that were constructed in 2011 are higher; therefore the community is already protected to the flood of record (return period of approximately 1:140 years) and the 1:200 year flood.

12.4.1 Conceptual Designs and Cost Estimates

Conceptual designs and cost estimates were prepared to upgrade the existing diking system in the communities to increase the flood protection level to a 1:200 year event (Souris and Melita) and also to a 1:500 year event (Souris, Melita and Wawanesa). The conceptual design of the dike upgrades was based on the layout and design of the existing dikes. The design elevation

for the dikes in Souris and Melita was determined based on rating curves provided by MIT for the communities and included a 0.6 m (2 ft) freeboard allowance. In Wawanesa, a water surface profile taken from the Souris River Basin Board Study (SRBB, 1978) was used for the analysis.

In Melita, approximately 850 m (2,790 ft) of dike length would have to be raised by 0.1 m (0.3 ft) to achieve a 1:200 year flood protection level (flow of 790 m^3/s (27,900 cfs)). For the 1:500 year flood (flow of 1,116 m^3/s (39,400 cfs)), approximately 850 m (2,790 ft) of dike length would have to be raised by 0.7 m (2.2 ft) and approximately 1,450 m (4,760 ft) would have to be raised by 0.3 m (0.9 ft). A plan showing the Melita dike alignment and a sketch of locations that require increased height is provided in Appendix H.

In Souris, approximately 1,200 m (3,940 ft) of dike length would have to be raised by 0.5 m (1.6 ft) and approximately 100 m (330 ft) would have to be raised by 1.0 m (3.3 ft) to achieve a 1:200 year flood protection level (flow of 913 m^3/s (32,200 cfs)). For the 1:500 year flood (flow of 1,263 m^3/s (44,600 cfs)), approximately 500 m (1,640 ft) of dike length would have to be raised by 1.1 m (3.6 ft), approximately 1,200 m (3,940 ft) would have to be raised by 1.9 m (6.2 ft) and approximately 100 m (330 ft) would have to be raised by 2.4 m (7.9 ft). Due to the limited available space to construct a dike between the river and the Souris water treatment plant (located just northwest of the PTH 22 Bridge), a bin wall would have to be constructed along the dike at this location. A plan showing the Souris dike alignment and a sketch of the locations that require increased height is provided in Appendix H.

In Wawanesa, approximately 500 m (1,640 ft) of dike length would have to be raised by 0.2 m (0.7 ft) and approximately 800 m (2,620 ft) would have to be raised by 0.8 m (2.5 ft) to achieve a 1:500 year flood protection level (flow of 1,213 m^3/s (42,800 cfs)). A plan showing the Wawanesa dike alignment and a sketch of locations that require increased height is provided in Appendix H.

The total volume of fill material required to raise the dikes was calculated by subtracting the volume of the proposed dikes from the volume of the existing dikes. The proposed dikes were assumed to have a 4 m top width and 4:1 or 5:1 side slopes depending on the locations.

In Souris and Wawanesa, a significant portion of the dike alignment is located on private property. In Souris, for the existing dikes, easements were negotiated with most landowners prior to construction. However, a few land purchases and expropriations were necessary where the dike alignment was too close to existing structures, or where easements could not be negotiated. A similar process would be expected for the proposed dikes in Souris and Wawanesa. Therefore, at this conceptual level of analysis, an average allowance of $10,000 per property was incorporated into the estimate. The number of properties was determined based on MLI data and air photos. Detailed analyses beyond the scope of the current study are necessary to confirm whether the $10,000 allowance per property is sufficient. In Melita, the dikes are mostly located on crown, municipal or agricultural land. Therefore, an average rate of $10,000/ha was assumed for the cost estimate.

The quantities for dike construction and the estimated costs of the options are provided in Tables 12.4 and 12.5 for Souris, 12.6 and 12.7 for Melita, and 12.8 for Wawanesa.

Table 12.4
Estimated Costs of Dike Upgrades in Souris for 1:200 Year Flood Protection

Item	Description	Cost
Dike Fill	9,600 m^3 @ $20/$m^3$	$ 192,000
Dike Fill – allowance for longer haul	9,600 m^3 @ $5/$m^3$	$ 48,000
Revegetation	2 ha @ $5,500/ha	$ 10,000
Land Acquisition	59 properties @ $10,000/property (residential, commercial, industrial, etc.)	$ 590,000
Bin Wall (at Water Treatment Plant)	$150,000 – lump sum	$ 150,000
Indirect Costs (Owner's Costs, Engineering, Administration, Approvals)	20% of Total Direct Costs	$ 198,000
Contingency Allowance	30% of Total Direct and Indirect Costs	$ 357,000
Total Cost		**$ 1,645,000**

Table 12.5
Estimated Costs of Dike Upgrades in Souris for 1:500 Year Flood Protection

Item	Description	Cost
Dike Fill	60,200 m^3 @ \$20/m^3	\$ 1,204,000
Dike Fill - allowance for longer haul	60,200 m^3 @ \$5/m^3	\$ 301,000
Revegetation	5 ha @ \$5,500/ha	\$ 27,000
Land Acquisition	84 properties @ \$10,000/property (residential, commercial, industrial, etc.)	\$ 840,000
Bin Wall (at Water treatment plant)	\$150,000 - lump sum	\$ 150,000
Indirect Costs (Owner's Costs, Engineering, Administration, Approvals)	20% of Total Direct Costs	\$ 505,000
Contingency Allowance	30% of Total Direct and Indirect Costs	\$ 908,000
Total Cost		**\$ 3,935,000**

Table 12.6
Estimated Costs of Dike Upgrades in Melita for 1:200 Year Flood Protection

Item	Description	Cost
Dike Fill	4,000 m^3 @ \$20/m^3	\$ 80,000
Dike Fill - allowance for longer haul	4,000 m^3 @ \$5/m^3	\$ 20,000
Revegetation	2 ha @ \$5,500/ha	\$ 11,000
Land Acquisition	0.2 ha @ \$10,000/ha	\$ 2,000
Indirect Costs (Owner's Costs, Engineering, Administration, Approvals)	20% of Total Direct Costs	\$ 23,000
Contingency Allowance	30% of Total Direct and Indirect Costs	\$ 41,000
Total Cost		**\$ 177,000**

Table 12.7
Estimated Costs of Dike Upgrades in Melita for 1:500 Year Flood Protection

Item	Description	Cost
Dike Fill	31,400 m^3 @ \$20/m^3	\$ 628,000
Dike Fill - allowance for longer haul	31,400 m^3 @ \$5/m^3	\$ 157,000
Revegetation	7 ha @ \$5,500/ha	\$ 39,000
Land Acquisition	1 ha @ \$10,000/ha	\$ 10,000
Indirect Costs (Owner's Costs, Engineering, Administration, Approvals)	20% of Total Direct Costs	\$ 166,000
Contingency Allowance	30% of Total Direct and Indirect Costs	\$ 300,000
Total Cost		**\$ 1,300,000**

Table 12.8
Estimated Costs of Dike Upgrades in Wawanesa for 1:500 Year Flood Protection

Item	Description	Cost
Dike Fill	31,700 m^3 @ \$20/m^3	\$ 634,000
Dike Fill - allowance for longer haul	31,700 m^3 @ \$5/m^3	\$ 159,000
Revegetation	4.3 ha @ \$5,500/ha	\$ 24,000
Land Acquisition	14 properties @ \$10,000/property (residential, commercial, industrial, etc.)	\$ 140,000
Indirect Costs (Owner's Costs, Engineering, Administration, Approvals)	20% of Total Direct Costs	\$ 191,000
Contingency Allowance	30% of Total Direct and Indirect Costs	\$ 344,000
Total Cost		**\$ 1,492,000**

12.4.2 Conclusions

1. Flood protection projects are currently underway at Souris and Melita to provide flood protection to the highest flood on record (the 2011 Flood) at a cost of approximately \$8 to \$10 million.

2. In Melita, the dikes currently under construction would have to be raised by 0.1 m (0.3 ft) for 1:200 year protection and 0.3 m to 0.7 m (0.9 ft to 2.2 ft) for 1:500 year protection, at an estimated cost of \$177,000 and \$1,300,000, respectively.

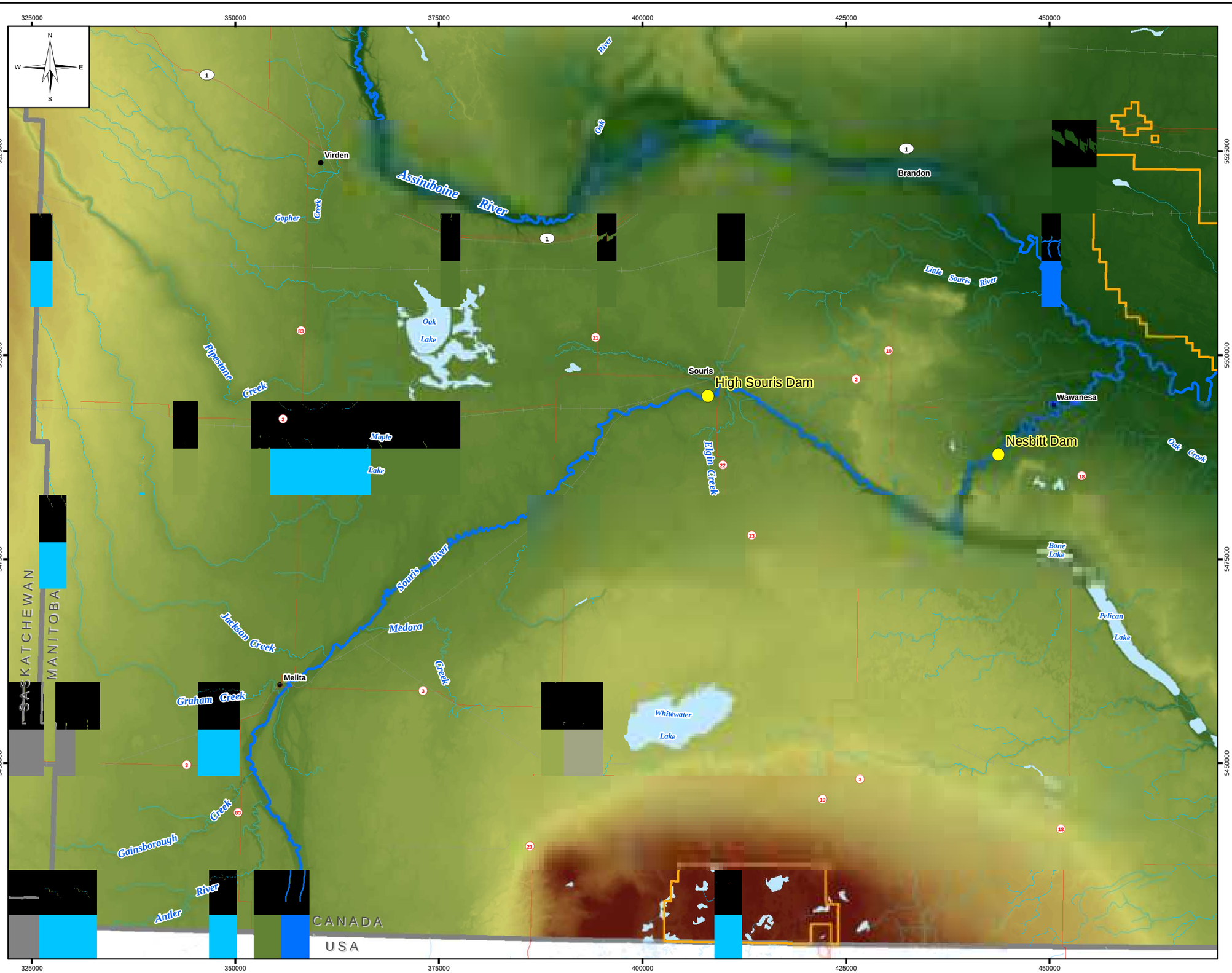
3. In Souris, the dikes currently under construction would have to be raised by 0.5 m to 1.0 m (1.6 ft to 3.3 ft) for 1:200 year protection and 1.1 m to 2.4 m (3.6 ft to 7.9 ft) for 1:500 year protection at an estimated cost of $1,645,000 and $3,935,000, respectively.

4. In Wawanesa, the dikes constructed in 2011 are higher and the community is protected to the flood of record and the 1:200 year flood. The dikes would have to be raised by 0.2 m to 0.8 m (0.7 ft to 2.5 ft) for 1:500 year protection at an estimated cost of $1,492,000.

5. Linear diking is the most feasible means of flood protection for the communities on the Souris River due to the relatively low cost compared to other flood mitigation options.

DRAWINGS

Portions of data presented are owned by the Province of Manitoba and are produced under the licence agreement with the Province of Manitoba 2016 Queen's Printer.



LEGEND:

- Large Storage Reservoir
- Rail Line
- Road
- Major River
- Other Streams
- Provincial Forest
- Waterbody
- Province or State

5 0 5 10 15 20

Kilometers

SCALE: 1:450,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
0	16/01/04	ISSUED WITH FINAL REPORT	PAL

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

JANUARY 2016	DWG 12.1	REV: 0

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\Rev0\Final Report\12-0300-011_D12.1_Rev0.mxd
11"x17" PLOT SCALE 1:1

13.0 DAUPHIN LAKE

13.1 INTRODUCTION

High water levels were experienced on Dauphin Lake in 2011, and there is a need for more effective control of water levels on the lake to reduce flooding during severe runoff events.

This section:

- Provides a brief description of the characteristics of Dauphin Lake, the catchments that drain into the lake, and the characteristics of the only outlet, Mossy River.
- Presents the preliminary results of analyses conducted on several options to improve conveyance of runoff through Dauphin Lake and increase the level of regulation.
- Provides some guidance on the feasibility of each of those options and assesses the impact on Lake Winnipegosis water levels.
- Re-assesses the effects of the influx of sediment to the lake on its storage capacity.

As part of the work described herein, numerous reports on previous studies were reviewed. A water-balance numerical model of the lake and its outlet was developed to simulate and predict lake levels for a range of outlet conditions. This model was also used to estimate the hydrograph of inflows-available-for-outflow (IAO) as they occurred in 2011. Finally, considerable effort was directed towards digitizing cross section data for the Mossy River that had been collected by PFRA, developing a numerical hydraulic model of the river, and calibrating that model against data published by Water Survey of Canada (WSC) for water levels and discharges.

The practicality and costs of three general strategies to lower lake levels during severe flood events were examined within the context of two overall objectives:

1. To maintain water levels during a 2011 Flood event between El 260.0 m (El 853.02 ft) and El 260.4 m (El 854.33 ft).
2. Since the cost of achieving the first objective is clearly very high, to understand what would be required to achieve more modest reductions in water levels that could still provide some economic benefits at affordable costs.

13.2 CHARACTERISTICS OF DAUPHIN LAKE AND SURROUNDING WATERSHED

Dauphin Lake is located about 10 km (6.2 mi) due east of Dauphin, Manitoba (Figure 13.1). The lake is about 42 km (26 mi) long and has a width that varies from 8 km (5 mi) at the north end of the lake to 18 km (11 mi) at the south end. The long axis of the lake is oriented in a NNW direction. The lake has a surface area of approximately 500 km^2 (193 mi^2). Lake levels have been measured in a systematic way since 1949 by Water Survey of Canada (WSC), and the lake has been regulated over this entire period of record. Since about the mid-1960s, lake levels have seldom been allowed to fall below 260.0 m (853.0 ft) with water levels below that threshold occurring only for a few days each year – the recorded minimum lake level is approximately El 259.7 m (El 852 ft) (Figure 13.2). Lake levels have exceeded El 261.5 m (857.94 ft) in five separate years since the mid-1960s. The maximum recorded water level of El 262.34 m (El 860.7 ft) was experienced in 2011. This was about 0.40 m (1.3 ft) higher than the previous high in 1974. It should be noted that the quoted water levels are based on daily averages and may reflect both wave heights and wind-related lake setup to some extent.

Figure 13.1
Dauphin Lake Drainage Basin

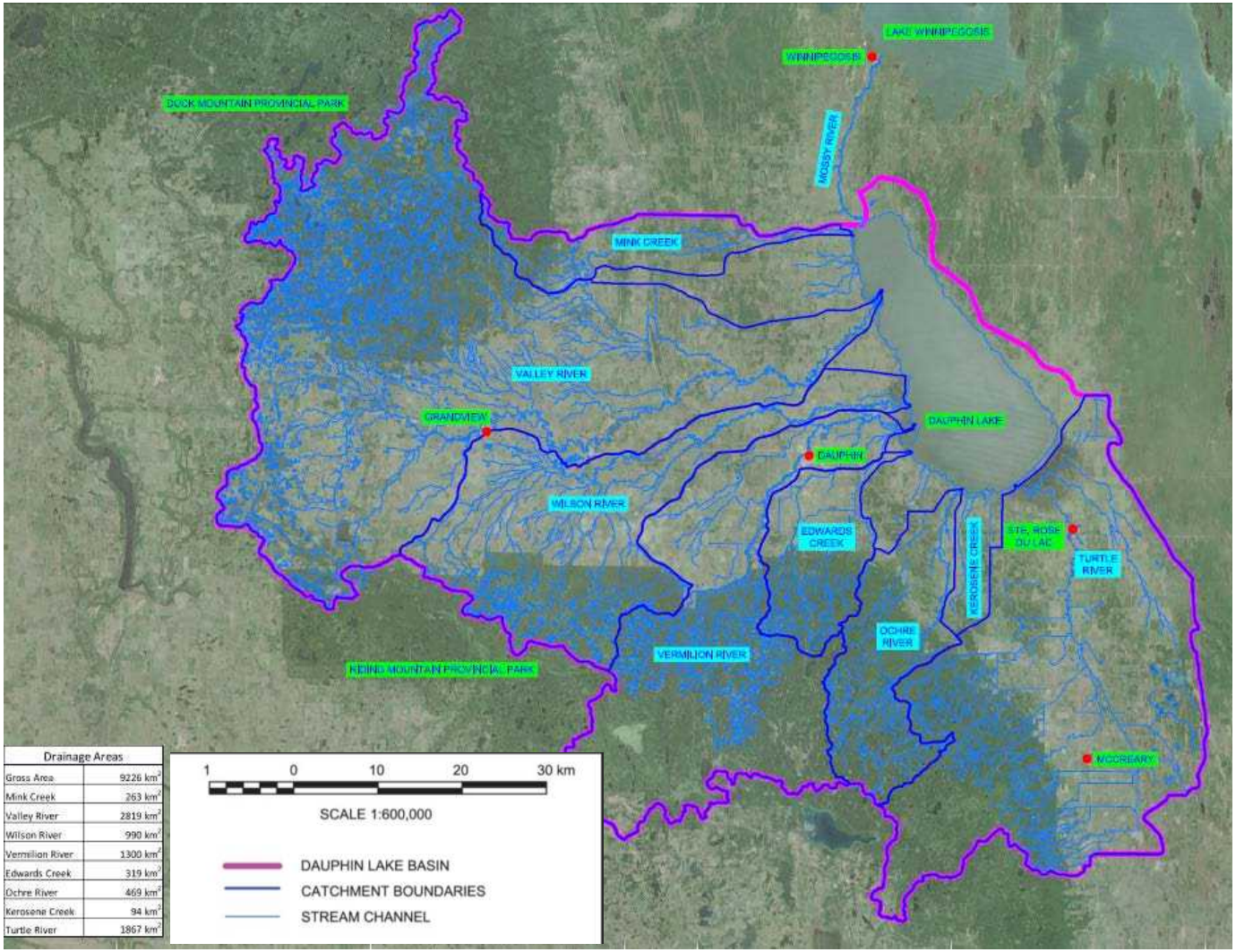
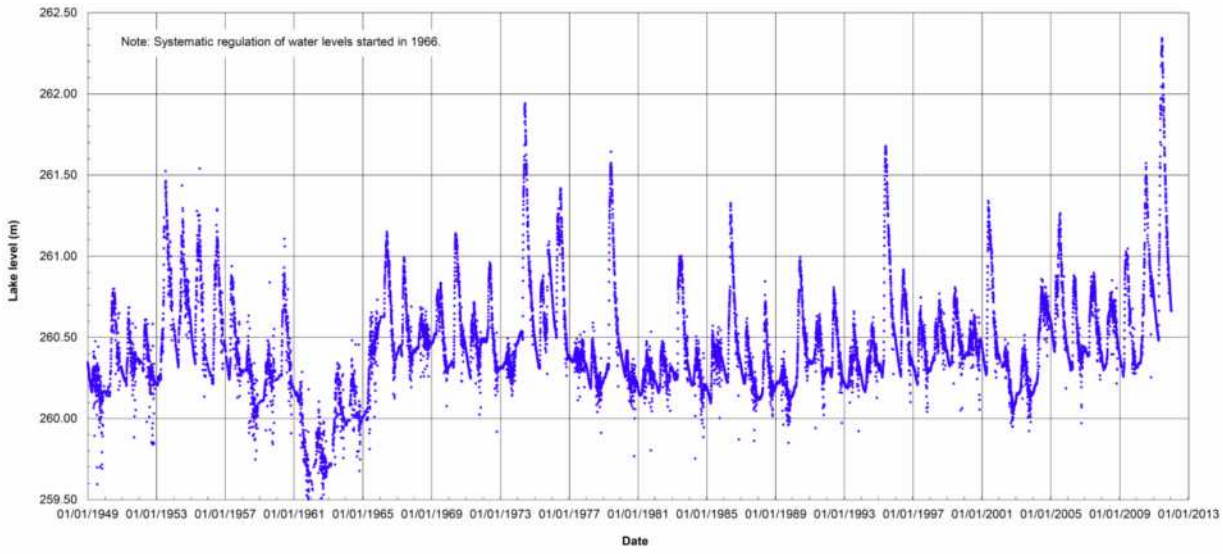


Figure 13.2
Dauphin Lake Water Level Measurements, WSC Gauge 05LJ009

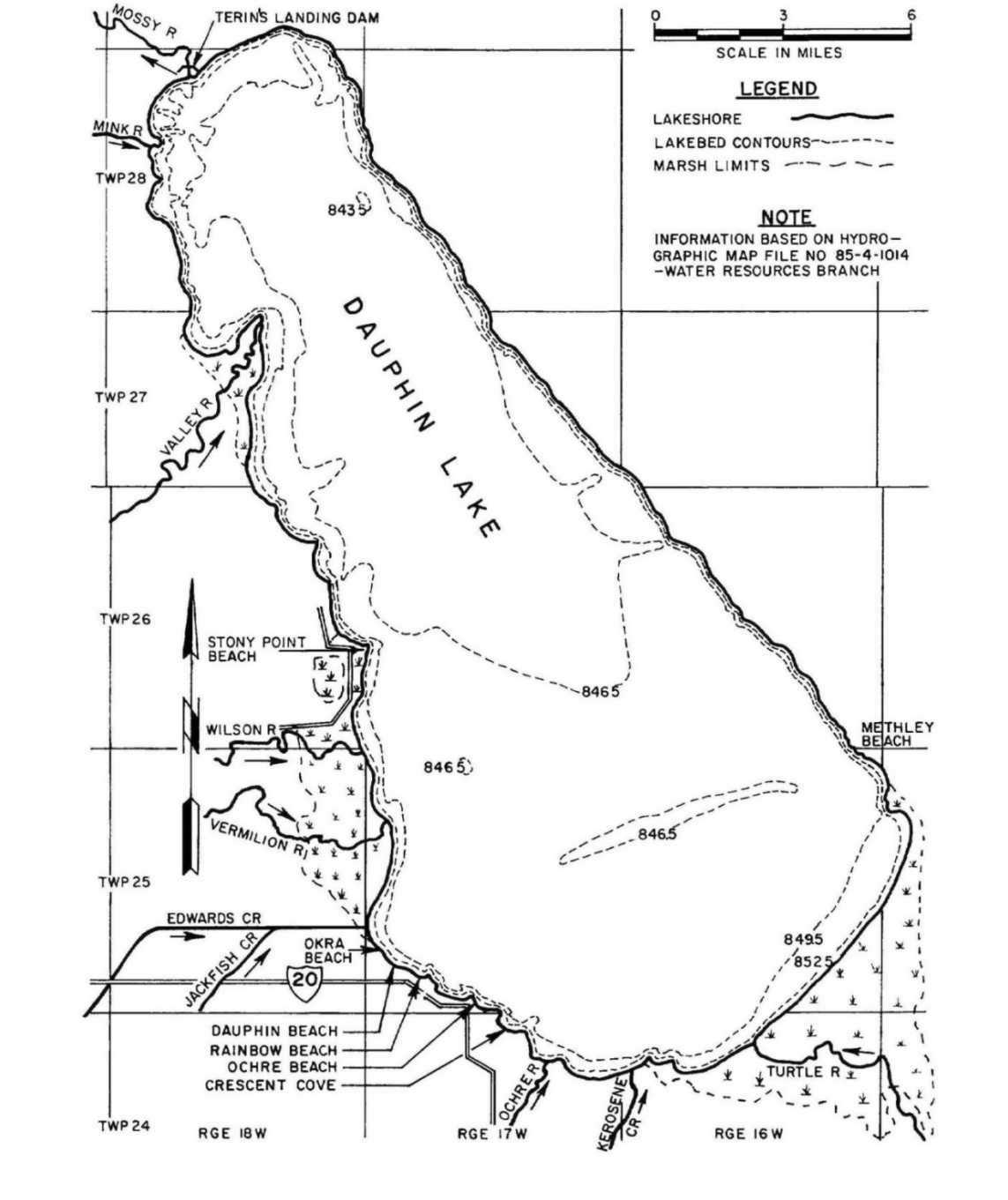


Note: Water levels can be converted to feet by dividing metric values by 0.3048.

The only outflow from the lake is via the Mossy River, which flows northward into Lake Winnipegosis. Lake Winnipegosis is located about 15 km (9.3 mi) north of Dauphin Lake. The average water level of Lake Winnipegosis is about El 253.0 m (830.05 ft) – some 7 m (23 ft) lower than Dauphin Lake. It has a surface area of approximately 5,400 km^2 (2,085 mi^2), which is about 11 times the surface area of Dauphin Lake. Clearly, Lake Winnipegosis can accommodate a significant amount of outflow from Dauphin Lake without adversely affecting its own water levels. For example, release of a volume of water associated with a reduction of 1 m (3.3 ft) in water level on Dauphin Lake would result in an increase of only about 0.10 m (0.33 ft) of water level on Lake Winnipegosis, and an increase of only a few centimetres on Lake Manitoba.

The mean lake level for the period of record 1949-2011 is El 260.46 m (El 254.53 ft). At the mean lake level, the average and maximum lake depths would be 2.4 m (7.9 ft) and 3.7 m (12.1 ft), respectively (Figure 13.3, MWC, 1973 and 1984). The lake is very shallow, with a gradually sloped lake shore above and below the water line. For example, at the north end of the lake, the shore slopes away at about 0.6 m (2.0 ft) per 100 m (328.1 ft) (slope of 0.056 m/m). Typically, a 1 m (3.3 ft) change in water level on Dauphin Lake corresponds to a volume of water of about 500,000 dam^3 (405,000 ac-ft).

Figure 13.3
Dauphin Lake Bathymetry



Note: Bathymetric data shown in feet. To convert to SI units, multiply elevation by 0.3048.
Image Source: Dauphin Lake Regulation – Volume 1, the Manitoba Water Commission, April 1973.

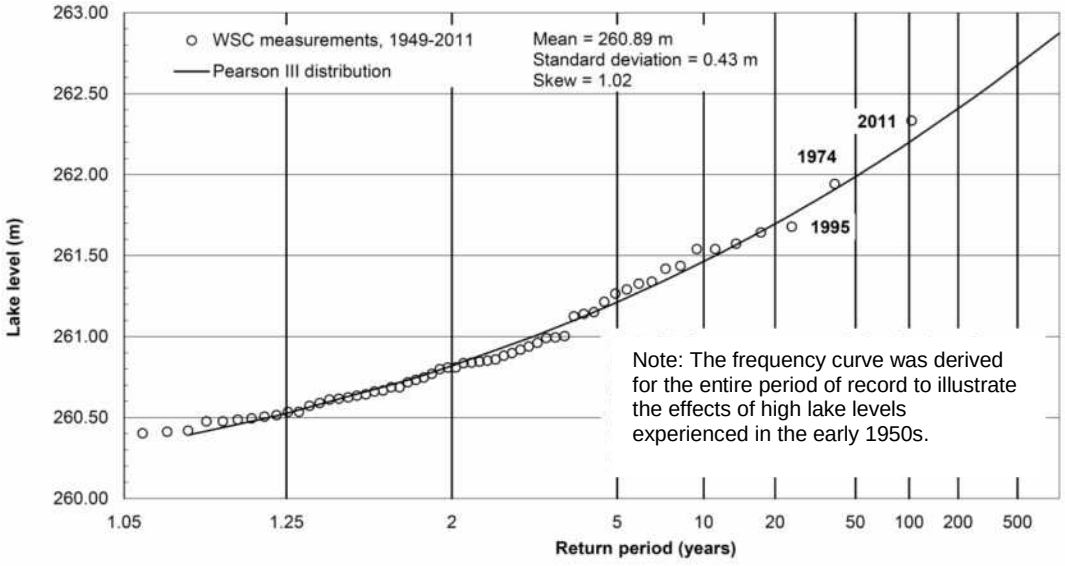
Water levels on Dauphin Lake are regulated by Mossy Dam, which is located on the Mossy River about 150 m (490 ft) downstream of the outlet from the lake. The dam consists primarily of

a stoplog structure with ten 3.66 m (12 ft) wide bays. The sill of the structure is at El 259.08 m (El 850.0 ft), but when all the stoplogs are installed the crest level would be at El 260.50 m (El 854.66 ft). A fish ladder with an invert El 259.385 m (El 851.0) is operated continuously throughout the year. Operating procedures at the Mossy Dam have been established to maintain water levels in the range of El 260.0 m (853.0 ft) to El 260.5 m (El 854.8 ft) – the former to accommodate spring runoff, and the latter to minimize adverse effects of high water on surrounding properties and recreation facilities.

During severe inflow events, such as in 2011, all the stoplogs would be removed to promote as much outflow as possible. During dry years and during the winter period, the stoplogs would be replaced to limit outflow and maintain water levels as high as would be appropriate. In fact, most of the outflow during winter has been typically via the fish ladders. However, the structure is operated in the winter based on information on snowpack measurements.

In spite of the inherent difficulties of carrying out statistical analyses on regulated systems, it is useful to examine the annual probabilities of experiencing a range of given lake levels. Figure 13.4 shows the frequency curve of lake levels for the period of record between 1949 and 2011.

Figure 13.4
Frequency Curve of Annual Peak Water Levels on Dauphin Lake



Note: Water levels can be converted to feet by dividing metric values by 0.3048.

Adopting a Log Pearson Type III probability distribution, the 100- and 200-year lake levels are estimated to be El 262.31 m (El 860.60 ft) and El 262.54 m (El 861.35 ft), respectively. The 2011 peak water level would appear from this data to have an average return period of about 150 years. Table 13.1 summarizes the lake levels for other salient return periods.

Table 13.1
Dauphin Lake Peak Annual Water Levels
for Selected Return Periods

Return Period (yr)	Dauphin Lake Peak Annual Level (m)
200	262.54
100	262.31
50	262.06
20	261.74
10	261.49
5	261.22
2	260.83

Note: Water levels can be converted to feet by dividing metric values by 0.3048.

13.3 HYDROLOGIC CHARACTERISTICS OF THE DAUPHIN LAKE WATERSHED

The drainage basin of Dauphin Lake has a total surface area of approximately 8,740 km^2 (3,375 mi^2). Inflows into the lake are predominantly from eight sub-catchments located on the south and west sides of the lake. These catchments (Figure 13.1) are located on the south slopes of the Duck Mountains, the northeast slopes of the Riding Mountains, the plains area between those two prominent landforms, and the plains area east of the Riding Mountains. Figure 13.5 shows the locations of the WSC hydrometric stations used to assess the hydrologic characteristics of the eight catchments; and Table 13.2 summarizes the salient characteristics of those gauges - location, drainage area, period of record, and April-October median runoff volume.

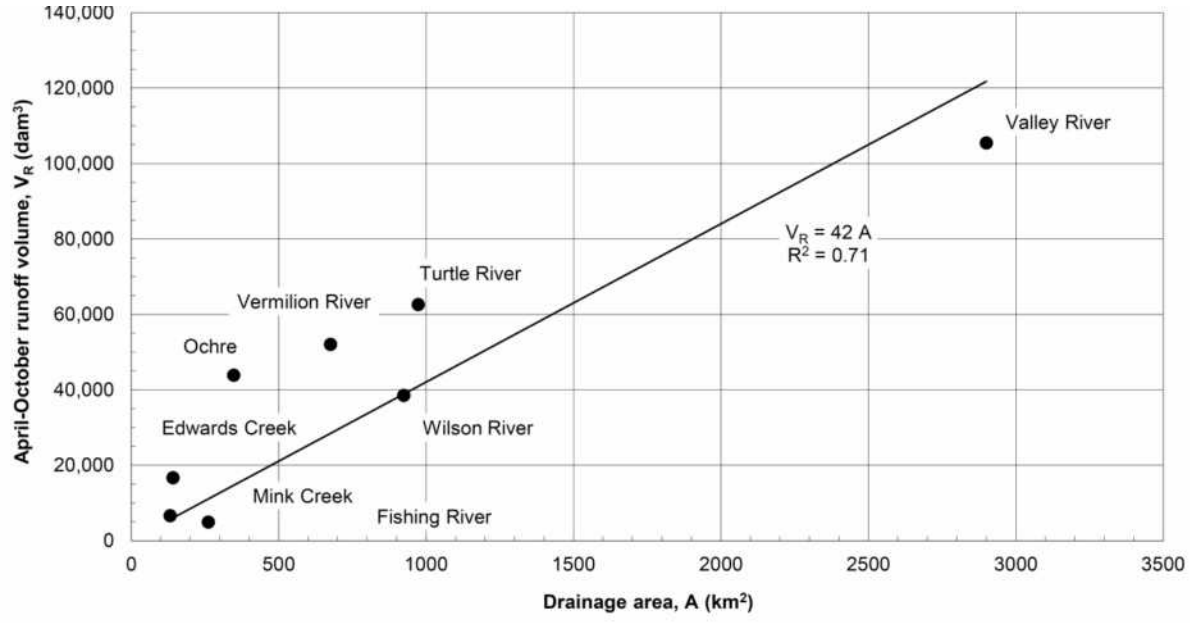
Figure 13.5
Dauphin Lake Location Plan

The April-October runoff volume is roughly proportional to the drainage area involved (Figure 13.6). However, there is considerable scatter of that data. A linear representation suggests that the median runoff volume would be about 42 dam^3 (34 ac-ft) per square kilometer of drainage area on average. This would indicate that in a median year, the inflow to the lake could be about 350,000 dam^3 (283,700 ac-ft), or equivalent to about 0.7 m (2.3 ft) of lake level rise, if there were no outflows.

Table 13.2
WSC Gauges that Represent Inflows into Dauphin Lake

Waterbody	Station Number	Period of Record Used in Analysis	Drainage Area (km^2)	Median Runoff Volume (dam^3)	Median Runoff per km^2 (dam^3/ km^2)
Ochre River at Ochre River	05LJ005	1950-2011	348 (134 mi^2)	43,900	126
Turtle River near Laurier	05LJ007	1950-2011	974 (376 mi^2)	62,600	64
Valley River near Dauphin	05LJ010	1950-2011	2,900 (1120 mi^2)	105,000	36
Wilson River near Dauphin	05LJ011	1965-1982	925 (357 mi^2)	38,500	42
Vermilion River near Dauphin	05LJ012	1948-2011	676 (261 mi^2)	52,000	77
Fishing River near Fork River	05LJ015	1948-1982	262 (101 mi^2)	4,890	19
Edwards Creek near Dauphin	05LJ022	1954-1980	142 (55 mi^2)	16,700	118
Mink Creek near Ethelbert	05LJ019	1954-2011	132 (51 mi^2)	6,630	50

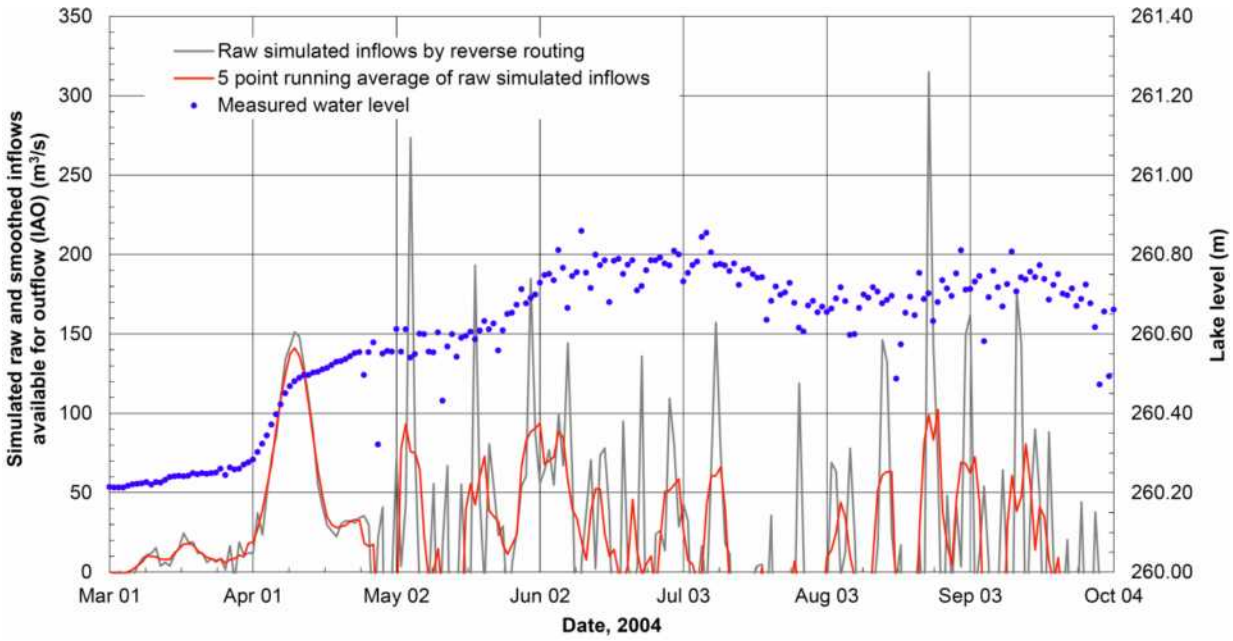
Figure 13.6
Median April-October Runoff Volumes from Streams Flowing into Dauphin Lake



The calculated hydrographs of inflow-available-for-outflow (IAO) (Figure13.7 shows a typical year) fluctuated widely over short-term periods due to wind effects. Large differences in lake

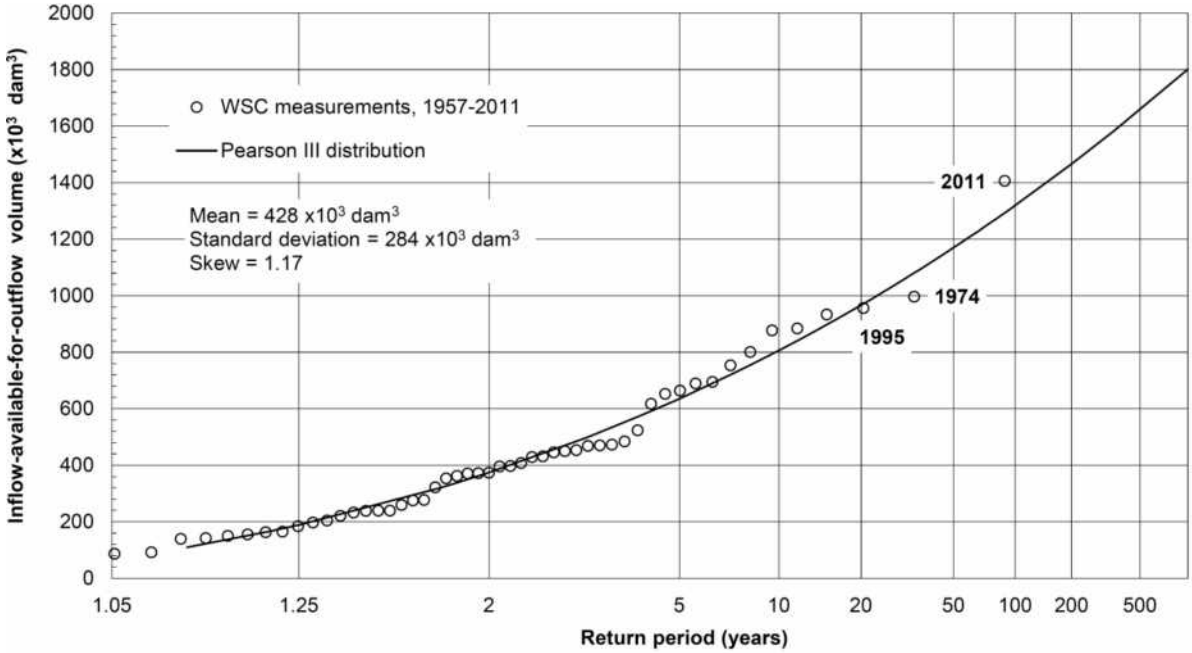
inflow due to day to day variations in lake levels would theoretically result. To compensate for this and provide a better approximation of the true steady inflows, the calculated inflows were smoothed using a 5-day running average. This produced a more realistic inflow hydrograph reflecting the small day to day changes in runoff that would be typical of the large contributing basin upstream of the lake.

Figure 13.7
Typical Record of Measured Dauphin Lake Levels and Simulated Inflows to the Lake by Reverse Routing



Note: Water levels can be converted to feet by dividing metric values by 0.3048.

Figure 13.8
Frequency Curve of IAO Volumes to Dauphin Lake



Note: Inflow volumes determined from back routing on the basis of measured daily lake levels and Mossy River outflows.

Figure 13.8 shows the frequency curve of annual IAO volumes for Dauphin Lake. Using again the Log Pearson Type-III distribution to fit the data, the mean and the maximum annual IAO volumes are 428,000 dam^3 (347,000 ac-ft) and 1,400,000 dam^3 (1,135,000 ac-ft), respectively. The median IAO volume is 400,000 dam^3 (324,000 ac-ft). This approximates the runoff that would be estimated from known regional runoff characteristics, as described above. The 100-year IAO value is estimated to be 1,300,000 dam^3 (1,054,000 ac-ft) (see Table 13.3 for this and other volumes for a range of return periods). From this perspective, the IAO volume for 2011 corresponds to about a 150-year event – the same as the peak lake level in 2011. As a comparison, the IAO volume for 1974 would have been about a 20-year event.

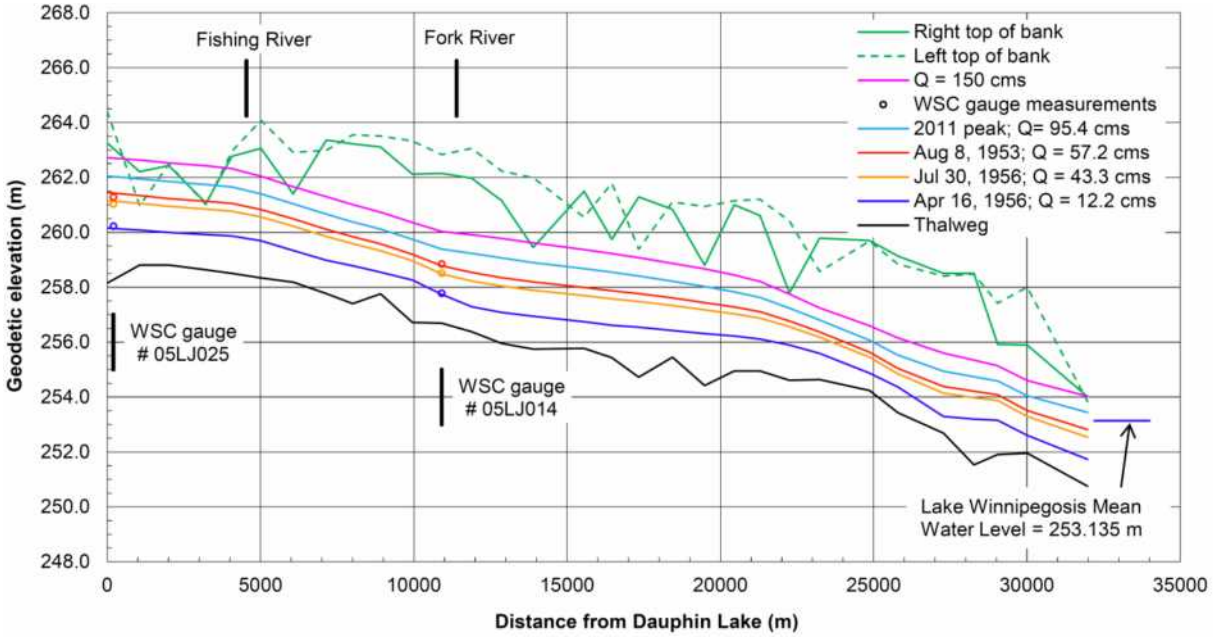
Table 13.3
Dauphin Lake IAO Volumes for Selected Return Periods

Return Period (years)	IAO Volume (dam^3)
200	1,470,000 (1,192,000 ac-ft)
100	1,300,000 (1,054,000 ac-ft)
50	1,170,000 (949,000 ac-ft)
20	970,000 (786,000 ac-ft)
10	810,000 (657,000 ac-ft)
5	635,000 (515,000 ac-ft)
2	375,000 (304,000 ac-ft)

13.4 CHARACTERISTICS OF MOSSY RIVER

The Mossy River is about 32 km (20 mi) long and conveys water from Dauphin Lake to Lake Winnipegosis. The river bed slope of the upper 25 km (16 mi) of channel is about 0.21 m/km (1.11 ft/mi) while the lower 7 km (4 mi) of the river has a bed slope of 0.40 m/km (2.11 ft/mi) (Figure 13.9). The hydraulic characteristics of the upper 10 to 15 km (6 to 9 mi) of channel immediately downstream of the lake outlet control the outflows from the lake during high lake levels; whereas at times of low lake levels, the outlet structure is used to control the outflow from the lake. The average width of the channel and mean depth at bank full conditions are 80 m and 2.7 m (262 ft and 9 ft), respectively, in the 10 km (6 mi) long reach below the outlet structure.

Figure 13.9
HEC-RAS Model Calibration Profiles of the Mossy River



Note: Water levels can be converted to feet by dividing metric values by 0.3048.

Flows on the Mossy River have been reported at two gauging locations over the period of record. From 1948 to 1956, the gauge (05LJ014) was located about 11 km (7 mi) downstream of the lake outlet – just upstream of the confluence with the Fork River. Conventional flow measurements were made and rating curves were constructed on a routine basis at this location, although the gauge datum was never referenced to a geodetic datum. From 1957 to the present, the gauge (renumbered to 05JL025) was located just below the outlet structure. The largest flow on record was 95.4 m^3/s (3,370 cfs) and occurred in 2011. By comparison, the mean peak annual flow over the period of record was 30.6 m^3/s (1,080 cfs).

The Fishing and Fork Rivers enter the Mossy River from the west, 4.3 km and 11.1 km (2.7 mi and 6.9 mi) downstream of Dauphin Lake, respectively. Peak inflows to the Mossy River from these two tributaries are not coincidental with those on the Mossy River due to the lag effects provided by storage in Dauphin Lake. Therefore, the flows from these two tributaries are not germane to the role that the Mossy River plays in causing high water levels on Dauphin Lake. It is interesting to note also that the flows in the Fork and Fishing Rivers can be occasionally so large as to cause a temporary backflow through the Mossy River Control Structure into Dauphin

Lake. Although an interesting phenomenon, this has no influence on the high flood levels on Dauphin Lake.

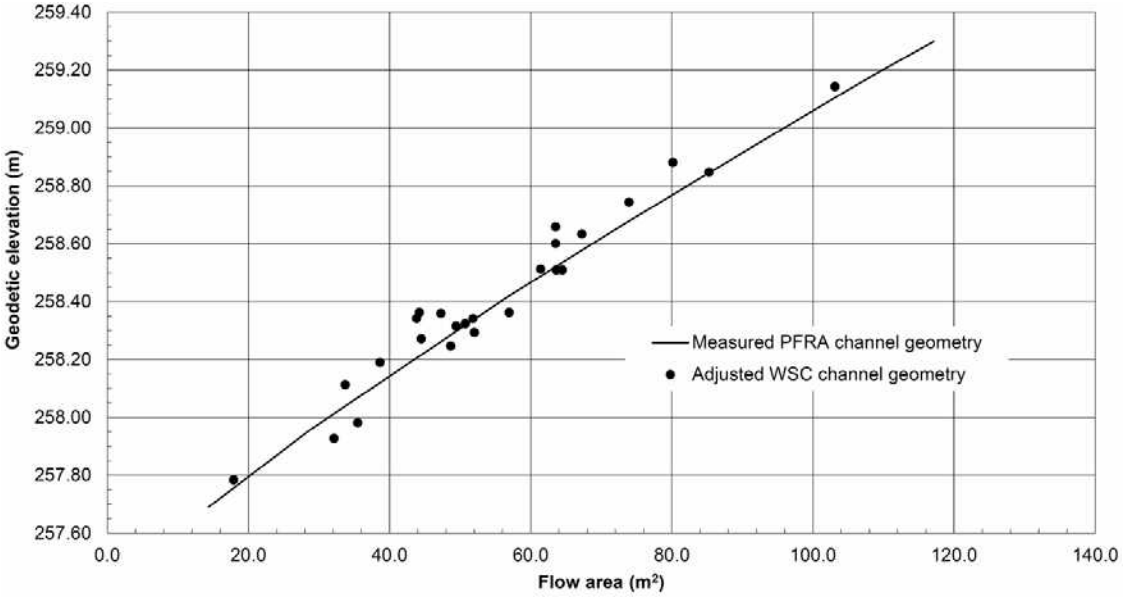
There appear to be no recent bathymetric surveys of the Mossy River. The last systematic survey is believed to have been conducted in 1949 by the Prairie Farm Rehabilitation Association (PFRA) when 31 cross sections were established at more or less equal intervals along the river between the two lakes. It is assumed that those river cross sections are still reasonably representative of the current channel geometry, and a HEC-RAS model was developed on that basis. The cross sections were digitized from the hard copy charts provided by the PFRA, and the cross section locations were transferred to GoogleEarth from the 1949 location maps. Model boundary conditions were based on water levels on Lake Winnipegosis measured at WSC Gauge 05LH001, and flows provided at WSC Gauge 05LJ025 at the outlet of Dauphin Lake.

Since the water levels at Gauge 05LJ014 had not previously been corrected to geodetic elevations, it was necessary to establish a relationship between:

1. The channel cross sectional area (referenced to a geodetic datum) of the PFRA cross sections close to the WSC metering site.
2. The local cross sectional area geometry as measured by WSC during numerous discharge measurements and for which the water level was referenced to the gauge datum.

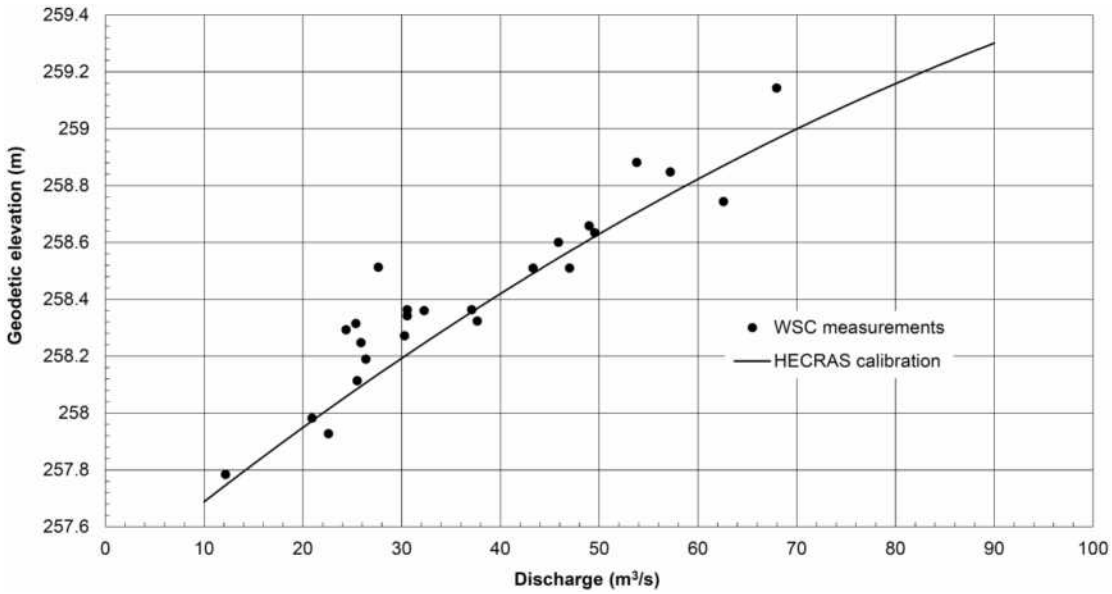
Figure 13.10 shows the comparison between the measured channel geometry of the PFRA cross sections and the WSC cross sectional area adjusted to the appropriate datum. Gauge zero at Gauge 05LJ014 was estimated to be El 229.8 m (El 753.9 ft).

Figure 13.10
Relationship between Flow Area
and Water Level at WSC Gauge 05LJ014



The corresponding rating curve at the gauge adjusted to geodetic datum is shown in Figure 13.11.

Figure 13.11
Rating Curve for WSC Gauge 05LJ014

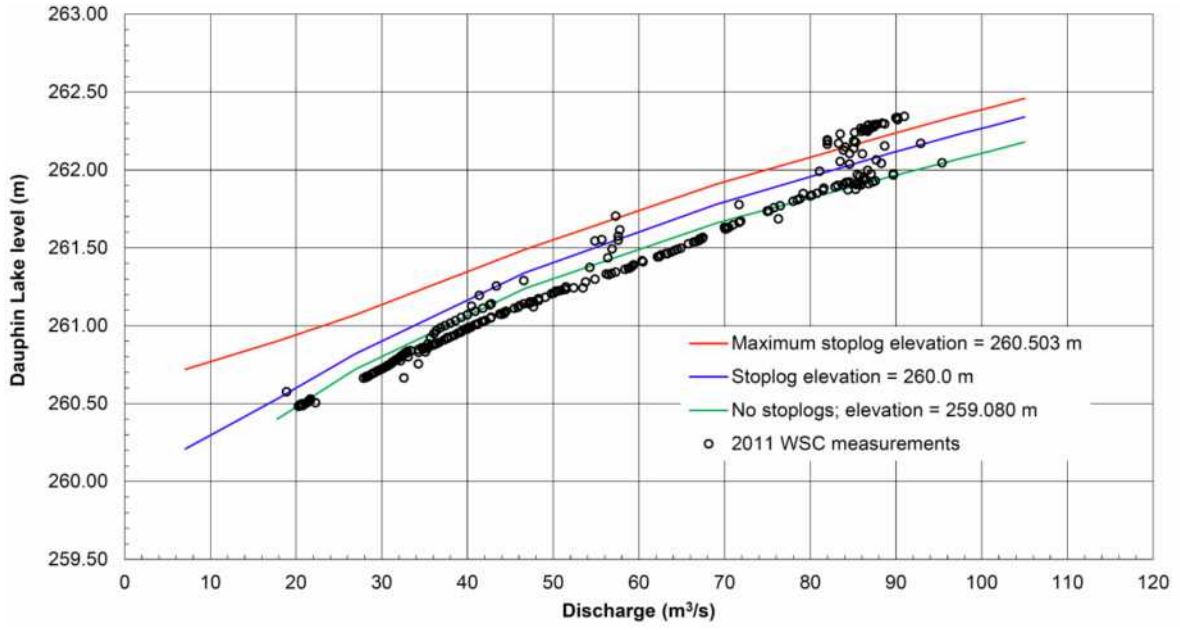


Notes: Flows can be converted to cfs by multiplying metric values by 35.3.
Water levels can be converted to feet by dividing metric values by 0.3048.

The Manning's roughness coefficient of the modeled riverbed was estimated from calibration against measured water levels at the two WSC gauges. The selected roughness coefficient of 0.034 would be applicable over a wide range of high flows. Several simulated profiles are shown in Figure 13.9. The discharge capacity of the Mossy River channel is estimated from the HEC-RAS analyses to be approximately 150 m^3/s (5,300 cfs) with what appears to be only a minor amount of overbank flooding upstream of the Fishing River confluence. Minor diking might increase the discharge capacity further, but this would need to be confirmed with more detailed surveys.

The HEC-RAS model results were also used to generate a stage-discharge relationship downstream of the Mossy Dam that would be independent of the stoplog configuration and reflect only the channel conveyance characteristics (Figure 13.12). That figure also shows the stage-discharge relationships for two scenarios of stoplog removal. Superimposed on the three curves are individual discharge-elevation points that were reported by WSC, but without corresponding information on the stoplog settings. When the stoplogs are removed from the dam, the channel rating curve downstream of the structure reconciles reasonably well with the WSC data for discharges above 20 m^3/s (700 cfs).

Figure 13.12
Rating Curve for the Dauphin Lake Outlet
for a Range of Stoplog Sill Elevations



Notes: Flows can be converted to cfs by multiplying metric values by 35.3.
Water levels can be converted to feet by dividing metric values by 0.3048.

13.5 IMPROVEMENTS TO LAKE LEVEL REGULATION

Currently, the regulation scheme for Dauphin Lake and the existing structure and channel are unable to achieve the passage of large runoff events quickly enough to prevent large increases in lake level. This was evident in 1974, 1995, and 2011.

Methods to improve the regulation of Dauphin Lake fall into two broad categories:

1. Reduce and delay the inflow volume into the lake by temporarily storing runoff in the headwaters. This method was analyzed by assessing existing upstream storage and comparing the typical flood volumes of Dauphin Lake's tributaries to what could realistically be stored in the headwater areas.
2. Increase the capability to release water from the lake. Two options were assessed:
   a) Construction of a separate diversion channel with a control structure to increase the potential for outflow from Dauphin Lake.
   b) Increase the conveyance of the Mossy River by excavation in conjunction with an enlarged control structure at the lake outlet.

13.5.1 Upstream Storage

The development of storage reservoirs on various streams flowing into Dauphin Lake could reduce peak lake levels by limiting peak inflows to the lake, and releasing the volumes over longer periods of time. In fact, a number of storage sites have already been developed in the Dauphin Lake watershed. Although they are multipurpose reservoirs, they still have the potential to contribute to some small reduction in lake levels. There are currently six documented reservoirs (Table 13.4) operated by the Province of Manitoba. These regulate outflows from a wide range of catchments whose surface areas range from about 27 km^2 to over 1,200 km^2 (10.4 mi^2 to over 463 mi^2).

Table 13.4
Existing Storage in the Dauphin Lake Basin

Name	Waterbody	Drainage Area (km^2)	Volume (dam^3)
Gilbert Plains Dam	Valley River	715 (276 mi^2)	67.8 (55 ac-ft)
Jackfish Dam	Jackfish Lake	27.2 (10.5 mi^2)	2,060 (1,670 ac-ft)
Pleasant Valley Dam	Pleasant Valley Creek	85.0 (32.8 mi^2)	15,400 (12,480 ac-ft)
Ste Rose Dam	Turtle River	1212 (468 mi^2)	185 (150 ac-ft)
Upper Dam	Valley River	492 (190 mi^2)	8,200 (6,650ac-ft)
Vermilion Dam	Vermilion River	357 (138 mi^2)	3,210 (2,600 ac-ft)

The total storage volume of the existing upstream reservoirs amounts to about 29,100 dam^3 (23,600 ac-ft). This is about 8% of the median annual IAO volume to Dauphin Lake, or about 2% of the 100-year inflow volume. Based on the outcome of the 2011 Flood, it is clear that the current level of upstream storage would be totally inadequate to attenuate the peak water level in Dauphin Lake, even if that was the sole purpose of those reservoirs.

Inflows to the lake are derived from more than ten catchments distributed around the lake, with the four largest ones (Table 13.5) being the source of about 70% of the total inflow volume in a median year. Any serious effort to provide sufficient upstream storage would need to be distributed throughout at least the major catchments.

Table 13.5
Runoff Volumes of Large Events in the Dauphin Lake Basin

Catchment	Annual Runoff Volume (dam^3)	
	Median	Maximum (Year)
Ochre River	43,900 (35,600 ac-ft)	107,000 [2011] (86,700 ac-ft)
Turtle River	62,600 (50,800 ac-ft)	203,000 [2011] (165,000 ac-ft)
Valley River	105,000 (85,000 ac-ft)	414,000 [2011] (336,000 ac-ft)
Vermilion River	52,000 (42,000 ac-ft)	168,000 [2011] (136,000 acre-ft)

The six sites summarized in Table 13.4 represent the physical characteristics of other storage opportunities in the Dauphin Lake Basin. As an average for those sites, about 62 ha (0.24 mi^2) of land has been inundated for every 1,000 dam^3 (800 ac-ft) of storage volume developed

(0.062 ha/dam^3) (0.2 ac/ac ft). On this basis, providing sufficient upstream storage to reduce the peak level of Dauphin Lake in 2011 by 0.5 m (1.6 ft) would require about 11,000 ha (42 mi^2) of new reservoir area to be constructed within the drainage basin. Based on the characteristics of the existing reservoirs, this would require the construction of about 33 large reservoirs / dams on the scale of the Pleasant Valley Dam, or alternatively 156 small reservoirs / dams on the scale of the Vermilion Dam – all to affect only a 0.5 m reduction in water levels. To maintain the 2011 lake level within the adopted operating range of El 260.0 to 260.4 m (El 853 ft to 854.3 ft) would require 10 times more reservoirs (the equivalent of 330 Pleasant Valley Dams for example). This is clearly not a practical solution, and no further assessment of this strategy is proposed.

13.5.2 Improvements to the Capacity of Lake Outflow

Two optional concepts to increase outflows from Dauphin Lake were analyzed:

1. The existing Mossy River control structure, combined with an auxiliary outlet channel with a control structure at its inlet.
2. An improved Mossy River channel, combined with an expanded control structure at the outlet from Dauphin Lake.

A numerical model of daily lake levels and outflows over the period of record was developed to assist the analysis. This model accounts for the effects of whatever management options were in force at the time. It also represents the various stoplog configurations selected over the period of record, identifies the water level statistics for any given return period (or lake level durations) for any time period, and ultimately assists in determining the optimal management scheme. A comparison of computed water levels for the various options against the design criteria would indicate the benefits associated with each scheme. The advantage of this numerical model is that it offers the ability to use the entire water level record for the lake to assess current flood risks.

This numerical model assumes that the past climate and runoff characteristics represent future conditions. On the basis of that assumption, the model is able to examine changes to the lake level regime that would result from any particular intervention. This can be assessed by a comparison of the computed lake level duration curves and/or frequency curves. This approach:

- Reduces ambiguities about antecedent lake levels arising out of regulation of outflows.
- Provides a continuous computation of water levels over all seasons.
- Provides a means to optimize the gate operations / settings to meet water level management objectives for different seasons.
- Provides a clear indication to the public of what the managed lake levels would have been, had the new management options been in place over the historical period.

13.5.2.1 Numerical Model Configuration and Calibration

Dauphin Lake levels were simulated using a numerical modeling scheme utilizing daily recorded lake levels and Mossy River outflows, and that implicitly accounted for inflows without requiring an explicit definition of daily precipitation and evaporation over the surface of the lake. The main challenge was to establish representative outflow rating curves for the Mossy River during any time step – particularly when lake levels are low and stoplog settings are being adjusted frequently. This problem was addressed by establishing a recursive relationship between actual lake levels and outflows to define a state-based discharge rating curve. The rating curve would reflect the stoplog settings that would be in effect during each computational time step.

The outflows determined by the model are based on the simulated water levels and the prevailing stoplog configuration that is defined by the relationship between the measured lake level and the measured outflow. The only requirement is that the daily water levels in the simulation are within a range that would not invoke a change in stoplog settings. Whenever there is a change in the stoplog settings, the transition from one rating curve to another in the simulations is averaged over a period of about five days in the numerical model. As a result, there may be typically one or two days when the simulated outflows are not accurately estimated. However, this is considered minor and quite acceptable within the context of this screening analysis.

13.5.3 Auxiliary Lake Outlets

Auxiliary Lake Outlet Structures – The discharge capacity of the Mossy River is inadequate to maintain lake levels within the desired range when large inflow volumes occur. One way to achieve the lake level objectives during these events is to increase the outflow capacity by adding an additional outlet with a channel devoted to the release of that extra outflow. The basic

concept for the control could be a vertical drop structure. Figure 13.13 is a generic illustration of such a concept. The advantages of this concept are the high efficiency of the discharge over the crest, and the energy dissipation within a hydraulic jump contained in the structure that avoids erosion downstream. The structure would comprise a number of approximately 5 m (16.4 ft) wide bays with facilities to insert and remove stoplogs to regulate the outflow and the water level as required. The two degrees of freedom in the design of the structure are its crest length and its sill elevation – the latter being the lowest setting with all stoplogs removed that could be attained to maximize outflows.

Figure 13.13
Concept Sketch of Auxiliary Lake Outlet Structure

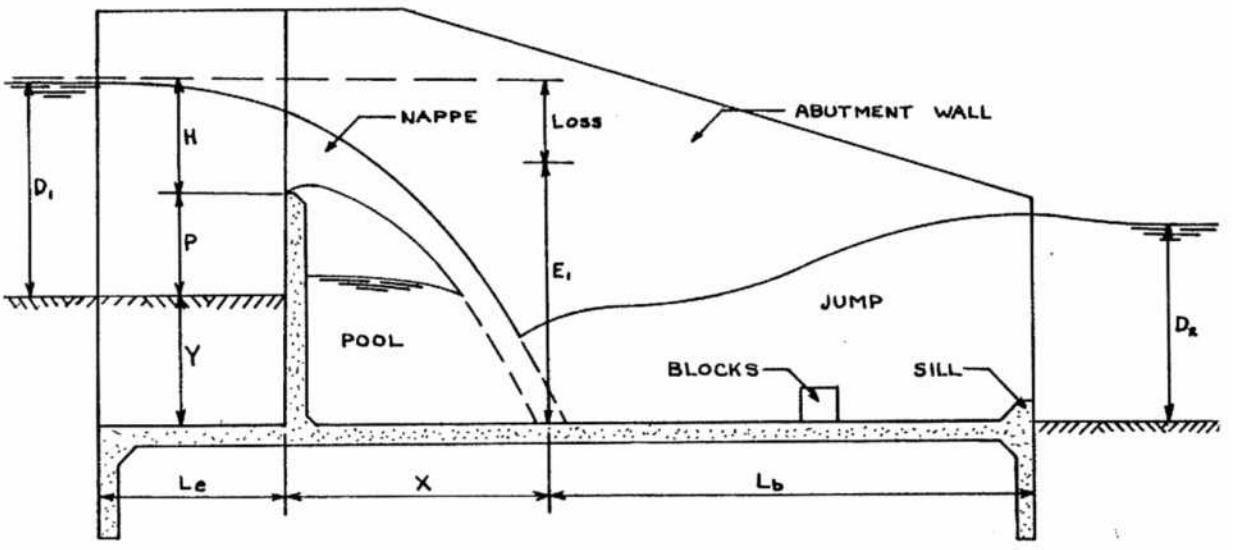


Image Source: Smith, C. D. "Hydraulic Structures". University of Saskatchewan Printing Services.1978. Chapter 6, Figure 6.

For the conceptual analysis undertaken herein, the structure dimensions have been determined on the basis of the standard dimensionless approach in which the width is prescribed on the basis of the required discharge capacity and the lengths of the various components of the structure are proportional to the head differential over the structure. The total length of the structure in the upstream / downstream direction reflects three main components (Figure 13.13): the length of an approach channel upstream of the weir, the nappe length, and the stilling basin length.

An alternative design could be a simple sluice structure consisting of piers on a horizontal slab, and stoplogs that can control flow under non-flood conditions. This structure would be somewhat more costly than the hydraulically more efficient vertical drop structure. In addition, its feasibility would be contingent on locating and installing adequate riverbed armouring downstream of the structure so that a concrete chute and stilling basin would not be required. If this concept were shown in more advanced design to be feasible, it would have the advantage of operating effectively with tailwater levels typically about 0.6 to 0.7 m (1.97 to 2.30 ft) higher than for the vertical drop structure. This permits considerable economy in the channel downstream and more than offsets the potentially larger cost of the control structure itself as discussed in Section 13.5.3.1.

The effects on lake levels of a range of structure alternatives with different widths and sill elevations were assessed using the simulation model and assuming that the control structure on the Mossy River was operated according to past practice. It also assumed that the structure would be paired with a suitable channel downstream so that the desired outflow could be achieved. The target for the modifications was first considered to be reduction of the peak water level that occurred in 2011 to El 260.4 m (El 854.3 ft) if the new structure(s) had been in place at that time. The results are summarized in Table 13.6.

Table 13.6
Structure Options to Maintain
2011 Lake Levels within Target Range

Effective Width of Structure (m)	Sill Level (m)	Simulated Peak Diversion Outflow	Simulated Mossy River Peak Outflow (1)	Simulated Total Peak Outflow (1)
20	257.60	178 m^3/s (6,285 cfs)	13 m^3/s (460 cfs)	191 m^3/s (6,745 cfs)
30	258.20	187 m^3/s (6,600 cfs)	13 m^3/s (460 cfs)	200 m^3/s (7,060 cfs)
50	258.73	205 m^3/s (7,240 cfs)	13 m^3/s (460 cfs)	218 m^3/s (7,700 cfs)
60	258.89	213 m^3/s (7,520 cfs)	13 m^3/s (460 cfs)	226 m^3/s (7,980 cfs)
100	259.25	236 m^3/s (8,335 cfs)	13 m^3/s (460 cfs)	249 m^3/s (8,790 cfs)

(1) The Mossy River flow depends on the lake level. With the peak lake level limited to 260.4 m (854.3 ft), the peak flow on the Mossy River is also limited.

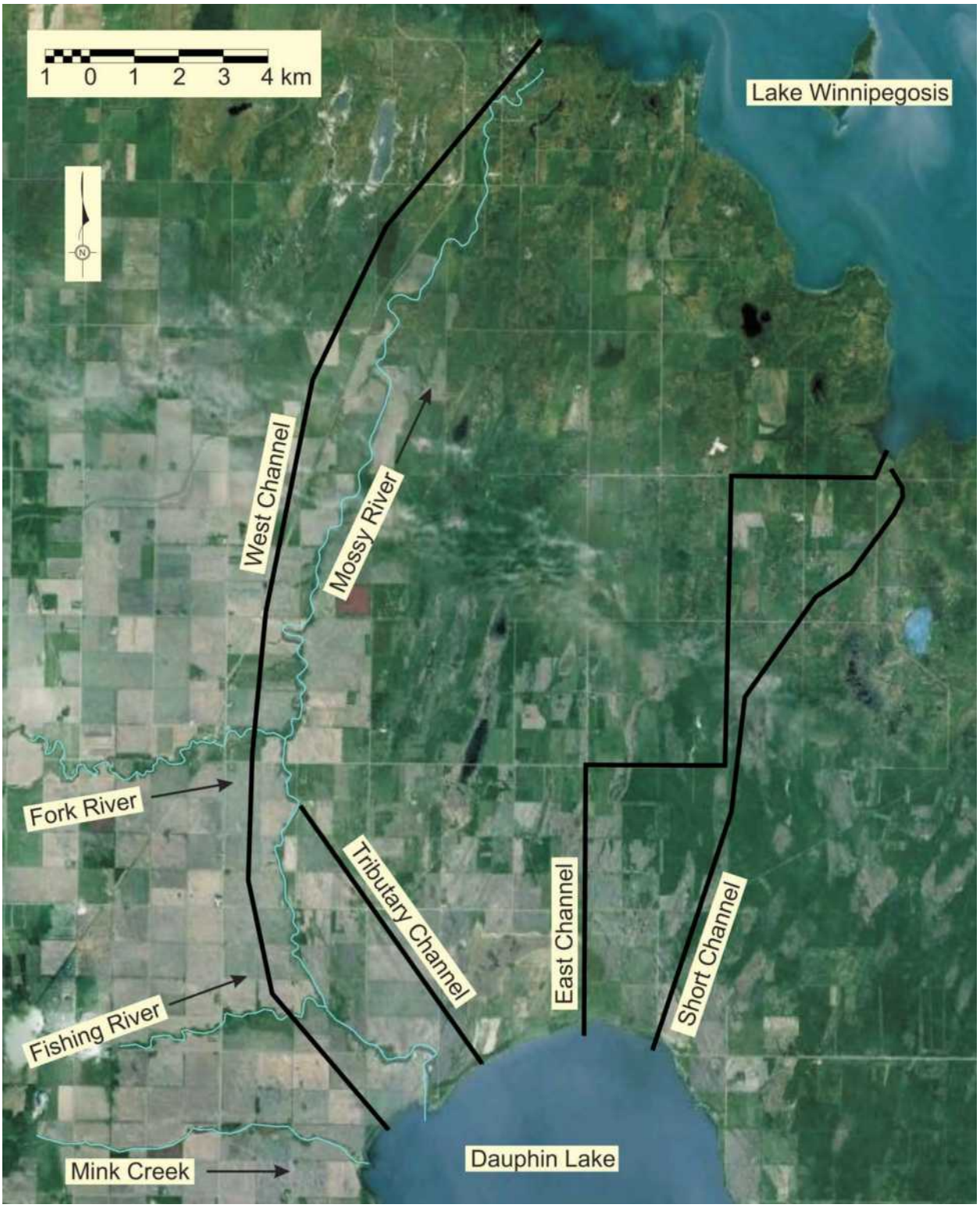
The sill level of the existing Mossy River Control Structure is El 259.1 m (El 850.06 ft), and the ground level around the lake shoreline is estimated from available bathymetric maps to be approximately El 260.0 m (El 854.3 ft). The contour at El 258.9 m (El 849.41 ft) is estimated to be located about 400 m (1312 ft) offshore. It is expected that it would prove difficult to install the sill of the structure at an elevation much below El 259.0 m (El 854.3 ft) without having to excavate an impractical amount of the lake bed. Therefore, the minimum width of the structure would appear to have to be about 60 m (196.85 ft) to keep the peak water level in a 2011 Flood at El 260.4 m (El 254.33 ft).

13.5.3.1 Auxiliary Outlet Channels

A wide variety of auxiliary channel alignments could convey flows to Lake Winnipegosis for any of the options of control structures described above. Four alignments are presented for discussion purposes. Channel locations are defined in Figure 13.14 and are described below:

- **Short Channel** is the short and most direct route based on an interpretation of the topography between the two lakes. This alignment will result in the lowest excavation volumes, but may require easements through private lands, and the lower few kilometres would be constructed within an existing stream.
- **East Channel** is similar to Short Channel but follows the approximate property lines as is evident on aerial photographs. It is longer than the Short Channel and would require more excavation. Easement costs may be less however.
- **Tributary Channel** exits the lake just east of the Mossy River and connects to the Mossy about 1.5 km upstream of the confluence of the Mossy and Fork rivers, at a point where the conveyance capacity of the Mossy River increases due to a slope increase (see Figure 13.9).
- **West Channel** follows the Mossy River along its west bank. This clearly is not a good option because of the large excavation volumes and because it would intercept the stream courses of the Fishing and Fork rivers. Nevertheless, it is shown for completeness.

Figure 13.14
Location Plan of Potential Auxiliary Outlet Channel Alignments



A standard cross section with an appropriate bed width and slope that would accommodate the required peak flow was selected. It was based in each case on channel side slopes of 5H:1V, a Manning roughness coefficient for the channel bed of 0.030, maximum allowable average velocity of 0.7 m/s (2.3 ft/s), with an expected peak local velocity at bends of 1.0 m/s (3.3 ft/s), and maximum shear stress on the channel bed of 5.0 N/m^2 (0.0007 psi). For flows in the range of 100 to 120 m^3/s (3,531 to 4,237 cfs), a bed width of 30 m (98 ft) and a channel bed slope of 0.15 m/km (0.79 ft/mi) comply with the above criteria and provide a framework to assess the excavation volumes of each of the channel options. It is possible that in more detailed design stages information from field investigations may reveal that steeper or flatter side-slopes may be required, or that a higher threshold of shear stress could be tolerated. However, the adopted parameters are considered representative and adequate for this screening level of study.

For the purposes of the initial analysis, a vertical drop structure at the lake outlet was assumed, as described above. The structure could be equipped with stoplogs to permit additional control of flow in the future. The Short and East Channels would also require a drop structure to dissipate energy at an appropriate location, as well as a channel outlet structure at the release point into Lake Winnipegosis. The same concept of a concrete vertical drop structure has been adopted for these locations. The West Channel is long enough that a stable slope can be constructed without the need for a drop structure.

The size and number of drop structures proposed for these concepts are considered representative and suitable for the purposes of this conceptual planning study. However, these design features should be optimized in later design phases.

Conceptual analyses clearly showed that the Tributary Channel could not achieve the desired objectives in discharge release from Dauphin Lake to achieve a peak water level of El 260.4 m (El 854.3 ft) during a repeat of the 2011 Flood. It was eliminated from further comparisons. It should, nevertheless, be retained as a concept that may be applicable if economic studies show that a low cost scheme (with less aggressive objectives for lake level reduction) would be the best option. The Short Channel is by far the least costly alternative of the remaining options, and was carried forward as a basis to represent a scheme to achieve a peak water level of El 260.4 m (El 854.3 ft) in a repeat of the 2011 Flood.

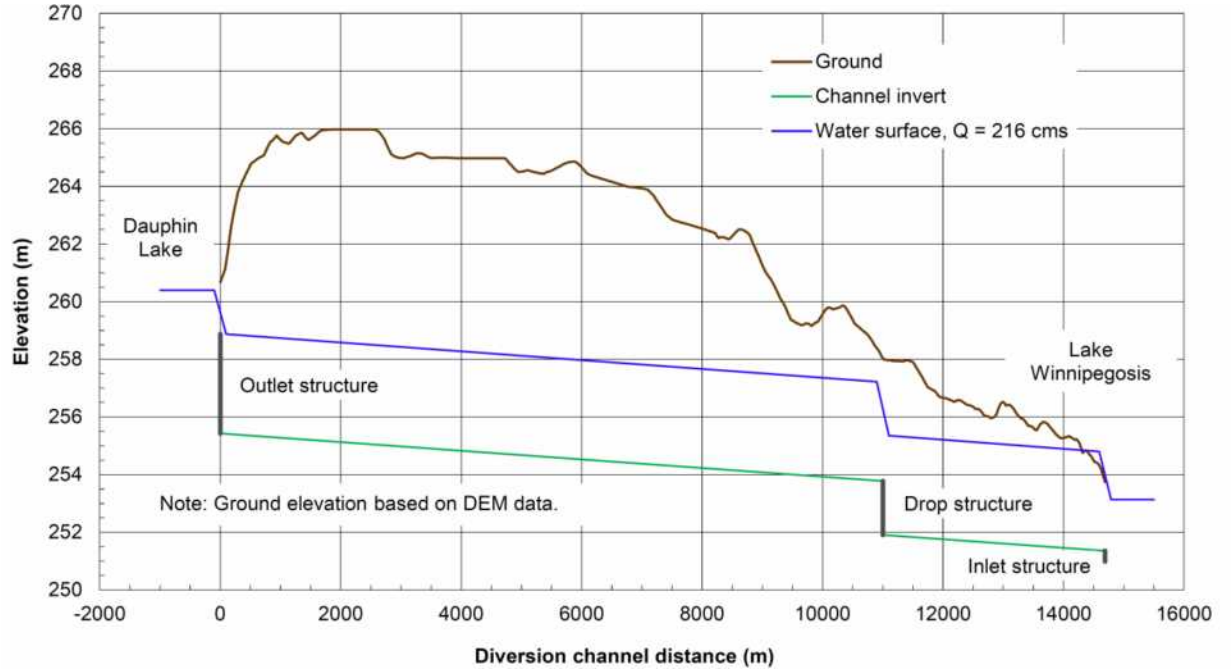
Table 13.7 summarizes the estimated costs for such a concept. This requires a design discharge capacity of approximately 110 m^3/s (4,000 cfs) in the new channel. The total cost is estimated to be about $253,500,000. The channel profile is shown in Figure 13.15.

Table 13.7
Structure Characteristics and Costs to Keep 2011 Lake Levels below 260.4 m for the Short Channel Alignment

Component	Parameter	Inlet Control Structure on Dauphin Lake	Drop Structure	Outlet Structure at Lake Winnipegosis	Cost
Structure Characteristics	Effective width	60.0m (197 ft)	60.0m (197 ft)	60.0m (197 ft)	$29,294,000
	Gross width [1]	65.5m (215 ft)	60.0m (197 ft)	60.0m (197 ft)	
	Approx. Head	1.5m (4.9 ft)	1.9m (6.2 ft)	1.7m (5.6 ft)	
	Length	15.0m (49 ft)	15.0m (49 ft)	14.0m (46 ft)	
	Footprint	990 m^2 (10,656 ft^2) @ \$10,000/m^2	900 m^2 (9,687 ft^2) @ \$10,000/m^2	840 m^2 (9,042 ft^2) @ \$10,000/m^2	
Channel	Length	14.7 km (9.1 miles)			$119,000,000
	Maximum cut	10.9 m (36 ft)			
	Volume of excavation	11,900,000 m^3 (15,600,000 yd^3) @ \$10/m^3			
Bridges (PR 364, PR 269 and 2 municipal roads)		2160 m^2 @ \$2,500/m^2			\$5,400,000
Land Acquisition		221 ha @ \$5,000/ha			\$1,105,000
Revegetation		221 ha @ \$5,500/ha			\$1,215,000
Environmental Mitigation Costs		5% of Total Direct Cost			\$7,801,000
Indirect Cost (Owners Cost, Engineering, Administration, Approvals)		20% of Total Direct Cost			\$31,203,000
Contingency Allowance		30% of All Costs			\$58,505,000
Total Cost					**\$253,523,000**

[1] Assumes five 0.5 m (1.6 ft) wide piers to hold stoplogs.

Figure 13.15
Short Channel Profile to Carry the 2011 Peak
(Weir Width = 60 m, Sill Elevation 258.87 m)
(Weir Width = 197 ft, Sill Elevation 849.3 ft)



Note: Water levels can be converted to feet by dividing metric values by 0.3048.

The cost shown in Table 13.7 is based on the conceptual arrangement using a vertical drop structure at the outlet from the lake. As described above, it is possible that an outlet control structure employing a simple sluice arrangement could result in a net cost reduction. Such a structure would allow the tailwater level to be approximately 0.6 m (2 ft) higher than assumed for the channel design and for the cost estimation shown in Table 13.7. But that would require a larger structure with a footprint area of about 1,200 m^2 (12,916 ft^2). The cost increase at $10,000 per square metre would be approximately $2.1 million in direct costs. However, the increase in the tailwater level at the structure would permit a comparable increase in the bed elevation of the channel (0.6 m or 2 ft) over the entire length of the channel from Dauphin Lake to Lake Winnipegosis. That has been estimated to result in a savings in excavation volume of 500,000 m^3 (650,000 yd^3) and a reduction in direct cost of the channel of approximately $5 million. The overall effect would be a reduction in the direct cost of the scheme by $2.9 million. However, this represents less than a 3% reduction in the estimated direct cost of the structure and channel, and less than that for a reduction in the overall total cost of the

scheme. This is well within the precision in estimating the cost of such works. The type of structure has, therefore, been considered an item that would be addressed in more detailed levels of design, if the scheme were to move forward. But the costs represented in Table 13.7 are considered adequate for this level of conceptual planning.

13.5.3.2 Other Options of Auxiliary Channels and Control Structures

The relatively high cost of the scheme described in Table 13.7 suggests the need to consider other less aggressive options (smaller structures, lower discharge capacity of channels) that would be more affordable.

The effects on lake levels of a range of lower cost alternatives with different widths and sill elevations were assessed using the simulation model. Again, it was assumed that the control structure on the Mossy River would be operated according to past practice. The results are summarized in Table 13.8.

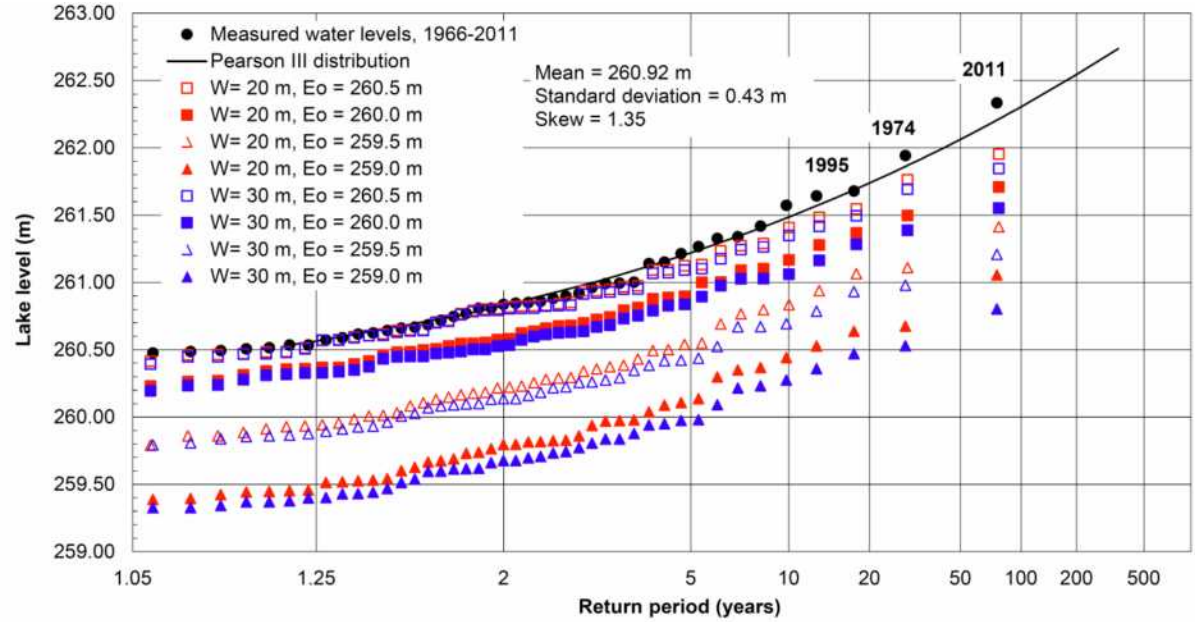
Table 13.8
The Effect of Alternative Weir Configurations on Lake Levels

Effective Width (m)	Sill Level (m)	Simulated 100-year Peak Level and Level Reduction (m)	Simulated 2011 Peak Level and Level Reduction (m)	Simulated 1974 Peak Level and Level Reduction (m)	Simulated 2011 Peak Diversion Outflow	Simulated 2011 Mossy River Peak Outflow
20	259.0	261.07 1.24	261.06 1.29	260.67 1.27	112 m^3/s (3,955 cfs)	43 m^3/s (1,520 cfs)
	259.5	261.43 0.88	261.41 0.93	261.11 0.83	101 m^3/s (3,570 cfs)	59 m^3/s (2,080 cfs)
	260.0	261.75 0.56	261.72 0.62	261.49 0.45	85 m^3/s (3,000 cfs)	72 m^3/s (2,540 cfs)
	260.5	261.98 0.33	261.96 0.38	261.76 0.18	67 m^3/s (2,365 cfs)	83 m^3/s (2,930 cfs)
30	259.0	260.85 1.46	260.80 1.54	260.53 1.41	138 m^3/s (4,875 cfs)	31 m^3/s (1,095 cfs)
	259.5	261.26 1.05	261.21 1.14	260.98 0.96	127 m^3/s (4,485 cfs)	50 m^3/s (1,765 cfs)
	260.0	261.61 0.70	261.56 0.78	261.39 0.55	110 m^3/s (3,885 cfs)	65 m^3/s (2,295 cfs)
	260.5	261.89 0.42	261.85 0.49	261.69 0.25	89 m^3/s (3,140 cfs)	78 m^3/s (2,755 cfs)

Notes: Water levels can be converted to feet by dividing metric values by 0.3048.

The options shown in Table 13.8 would not provide as much benefit as any of those described in Table 13.7. However, they still provide substantial water level reductions while not adversely affecting flows on the Mossy River in a significant way. For example, the 100-year water level would be reduced by between 0.33 m (1.1 ft) and 1.46 m (4.8 ft) – depending on which option is adopted. The effects of the various weir options on the lake level frequency curve are shown in Figure 13.16.

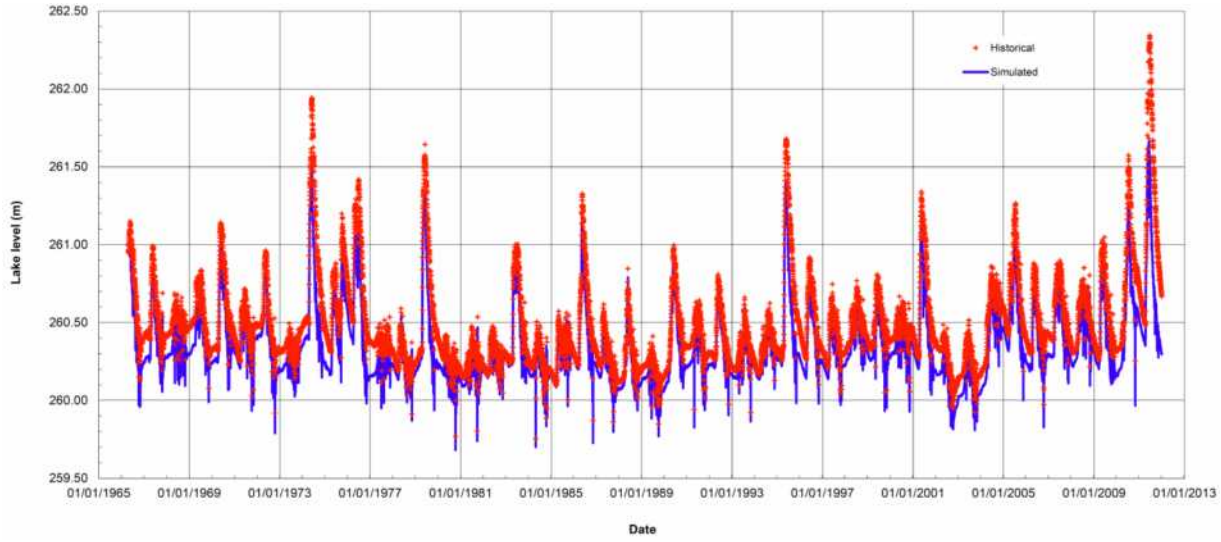
Figure 13.16
Comparison of Frequency Curves of Dauphin Lake Levels for a Range of Outlet Structure Alternatives



Note: Water levels can be converted to feet by dividing metric values by 0.3048.

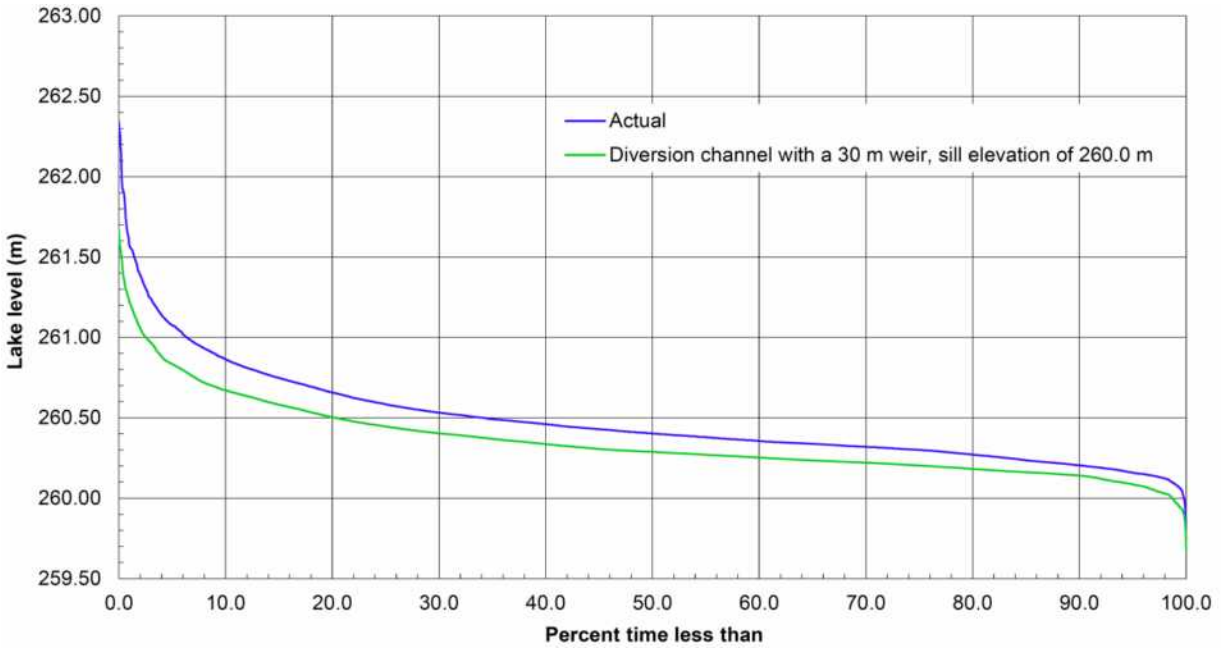
There are too many potential options to evaluate in detail within the scope of this study. As a compromise, in anticipation of potential difficulties in excavating the lake bed in the approach area to the control structure***, the 30 m (98 ft) wide weir option with a sill at El 260.0 m (El 853 ft) has been used herein for demonstration purposes.*** This option would have reduced the 2011 peak lake levels by 0.78 m (2.6 ft) to El 261.56 m (El 858.14 ft), and the 1974 peak would have been reduced to 261.39 m (857.58 ft) (Figure 13.17). Figure 13.18 compares the actual, or historical, water level duration curve for Dauphin Lake to that which would have occurred had the diversion channel been operational for the 1966 to 2011 period. The additional diversion capability for this scheme would have lowered the lake levels over their entire range by an average of approximately 0.30 m (0.98 ft). Figure 13.19 shows the flow duration curves for the Mossy River for the current condition and with the sample diversion channel. Due to the overall lower lake levels, Mossy River flows would be reduced; and the diversion channel would carry a portion of the flow previously conveyed by the Mossy River.

Figure 13.17
Comparison of Historical and Regulated Water Levels
on Dauphin Lake, 1966 – 2011
(Weir Width = 30 m, Sill Elevation = 260.0 m)



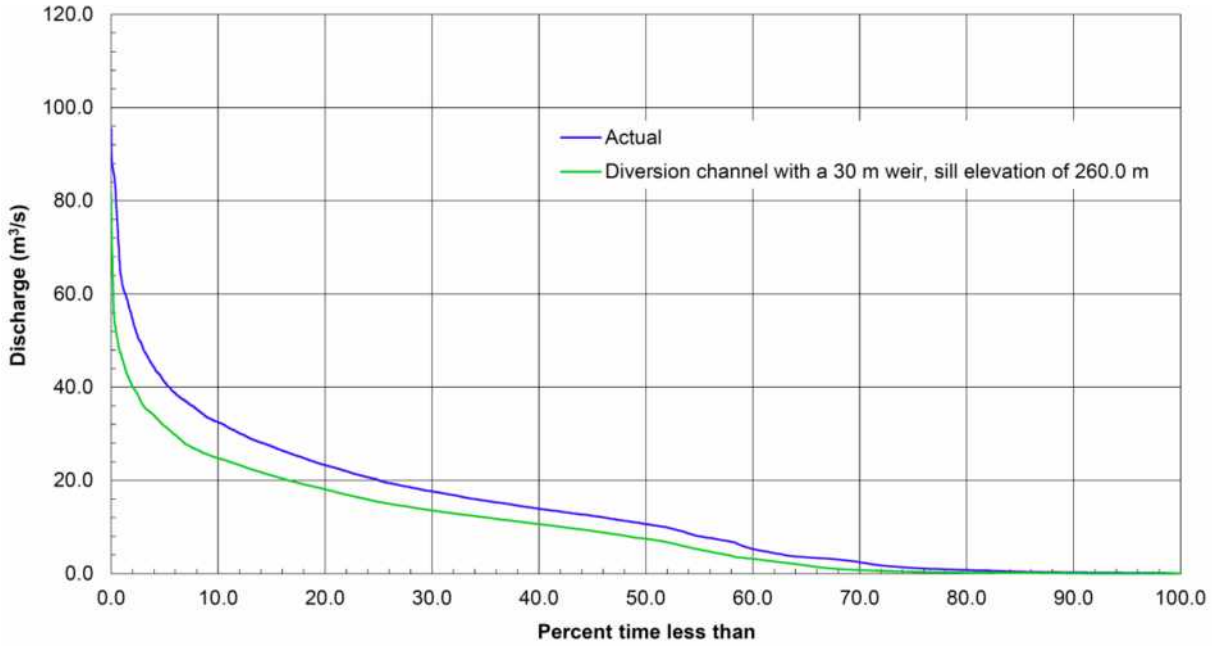
Note: Water levels can be converted to feet by dividing metric values by 0.3048.

Figure 13.18
Comparison of Duration Curves for Dauphin Lake Levels



Note: Water levels can be converted to feet by dividing metric values by 0.3048.

Figure 13.19
Comparison of Duration Curves for Mossy River Discharges



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

Key features of the channels associated with the 30 m (98 ft) wide control structure are compared conceptually in Table 13.9. The same criteria on side slope, roughness, maximum velocity, etc., as described above were used for the conceptual design of the channels. The Short and East Channels would also require drop structures at an appropriate location and a channel outlet structure at Lake Winnipegosis.

The West Channel is long enough that a stable slope could be constructed without the need for a drop structure. Clearly, the size and number of drop structures are design aspects that can be optimized; but for this preliminary comparative analysis, the configurations summarized in Table 13.10 form the basis for the cost comparisons. The locations of the drop structures were chosen to minimize the channel excavation volume and the volume of diking material that would need to be placed above the natural ground to convey the design flow.

Table 13.9
Excavation Quantities and Costs
for the Auxiliary Outlet Channel Alignments

Channel Option	Length (km)	Maximum Depth of Cut (m)	Volume of Excavation (m^3)	Excavation Cost [1] (million $)
Short Channel	14.7 (9.3 mi)	9.6 (31.50 ft)	7,400,000 (9,700,000 yd^3)	67.0
East Channel	19.6 (12 mi)	13.1 (42.98 ft)	12,600,000 (16,500,000 yd^3)	108.0
West Channel	27.2 (17 mi)	10.9 (35.76 ft)	15,800,000 (20,700,000 yd^3)	157.8

1 Based on an assumed unit excavation cost of $10.00/$m^3$.

Table 13.10 summarizes the estimated quantities and costs of the control and drop structures for each of the options.

Table 13.11 compares the total expected costs for the three optional channel alignments.

Table 13.10
Structure Characteristics and Costs
for Other Options of Auxiliary Outlets

Channel Option	Parameter	Outlet Control Structure on Dauphin Lake	Drop Structure	Inlet Structure on Lake Winnipegosis	Structure Cost[2] (million $)
Short Channel	Effective width (m)	30.0	30.0	30.0	13.6
	Gross width[1] (m)	32.5	30.0	30.0	
	Approx. Head (m)	1.6	3.0	1.7	
	Length (m)	15.0	15.0	14.0	
	Footprint (m^2)	490	450	420	
East Channel	Effective width (m)	30.0	30.0	30.0	13.6
	Gross width (m)	32.5	30.0	30.0	
	Approx. Head (m)	1.6	2.3	1.7	
	Length (m)	15.0	15.0	14.0	
	Footprint (m^2)	490	450	420	
West Channel	Effective width (m)	30.0	Not Required	30.0	9.1
	Gross width (m)	32.5		30.0	
	Approx. Head (m)	1.6		1.7	
	Length (m)	15.0		14.0	
	Footprint (m^2)	490		420	

1 Assumes five, 0.5 m wide piers to hold stoplogs.
2 Based on a unit cost of $10,000/$m^2$.

Table 13.11
Excavation Quantities and Costs for Other Auxiliary Outlets

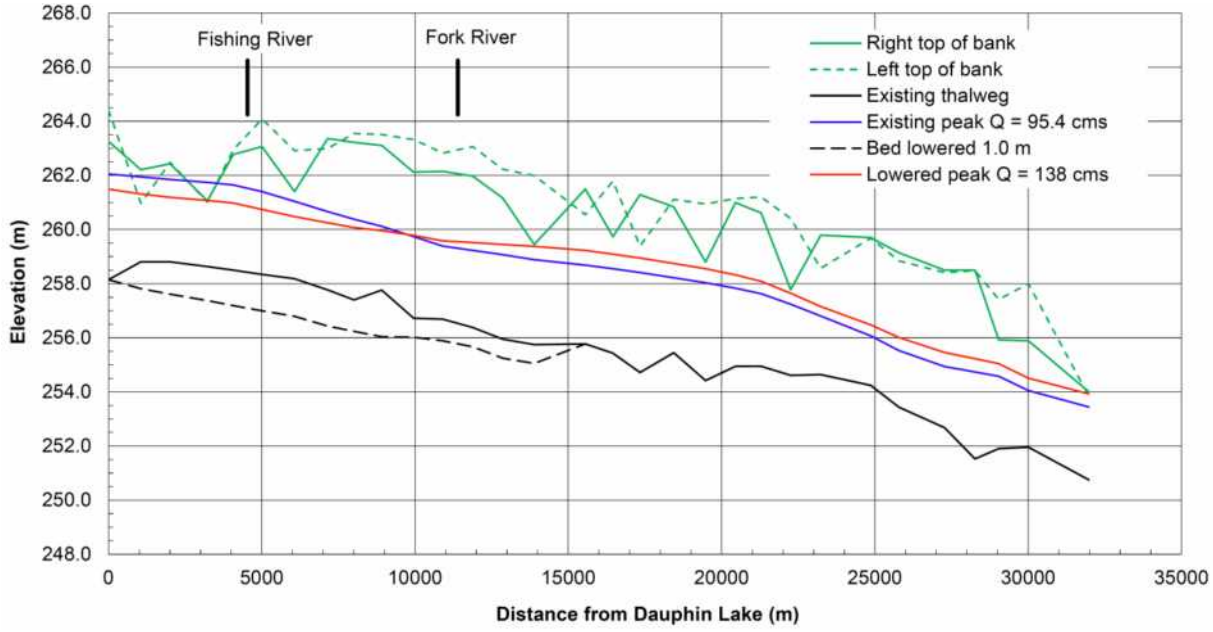
Cost Component	Short Channel	East Channel	West Channel
Channel Excavation (Table 13.9)	$67,000,000	$108,000,000	$157,800,000
Inlet, Outlet and Drop Structures (Table 13.10)	$13,600,000	$13,600,000	$9,100,000
Land Acquisition	221 ha @ $5,000/ha $1,105,000	294 ha @ $5,000/ha $1,470,000	408 ha @ $5,000/ha $2,040,000
Revegetation	221 ha @ $5,500/ha $1,215,500	294 ha @ $5,500/ha $1,617,000	408 ha @ $5500/ha $2,244,000
Bridge Allowance	4 bridges – Total 2,160 m^2 @ $2,500/$m^2$ $5,400,000	4 bridges – Total 2,160 m^2 @ $2,500/$m^2$ $5,400,000	4 bridges – Total 2160 m^2 @ $2,500/$m^2$ 2 PTH bridges – Total 1,200 m^2 @ $7,000/$m^2$ + $3M $19,800,000
Environmental Mitigation Costs	Minor (5% of Direct Costs) $4,416.000	Minor (5% of Direct Costs) $6,504,000	Moderate (10% of Direct Costs) $19,098,000
Indirect Costs (Owner's Costs, Engineering, Approvals, Administration)	$17,664,000	$26,017,000	$38,197,000
Contingency Allowance	$33,120,000	$48,783,000	$74,484,000
Total Cost	$143,521,000	$211,391,000	$322,763,000

13.5.4 Channel Improvements in the Mossy River

Currently, flow through the Mossy Dam when all stoplogs are removed is limited by the discharge capacity of the Mossy River (see Figure 13.12). The potential for increasing outflows from Dauphin Lake down the Mossy River was first studied in 1986 when it was proposed to lower the Mossy River bed by approximately 0.3 m (1 ft) over a 12.0 km (7.5 mi) reach downstream of Dauphin Lake (obtained from miscellaneous figures acquired from MIT over the course of this investigation).

This option has been re-examined herein using the HEC-RAS model based on the PFRA cross sections, and calibrated to WSC water level data. Three scenarios of channelization were assessed. The Mossy River channel was assumed to be lowered by 0.3, 0.5, and 1.0 m (0.98, 1.64 and 3.28 ft) in the upper 12 to 15 km (7.5 to 9.3 mi) reach of the river (Figure 13.20). The maximum depth of excavation was limited to 1.0 m (3.28 ft) in an attempt to avoid destabilization of the channel banks. This depth would require confirmation by geotechnical studies in future detailed design phases. Widening of the river would also be a possible option, but would result in larger excavation volumes to achieve the same effect as the hydraulically more efficient deepening of the channel. Analysis of this option was not addressed in the study but could be considered in future planning.

Figure 13.20
Comparison of 2011 Peak Water Surface Profiles Down the Mossy River



Note: Water levels can be converted to feet by dividing metric values by 0.3048.

Dauphin Lake levels were simulated for the 2011 inflow event for the three channelization scenarios (Table 13.12). The 2011 peak would have been reduced by between 0.29 m and 0.60 m (0.95 ft and 1.97 ft) and the corresponding peak outflow would have been 120 to 138 m^3/s (4,237 to 4,873 cfs) (see the water surface profiles in Figure 13.20) as the depth of excavation increased. One metre of lowering the channel bed would reduce the peak water levels by the same amount as a 20 m (66 ft) wide auxiliary outlet structure set at a sill elevation of 260.0 m (853 ft). However, the peak water level would still have been 1.33 m (4.4 ft) above the target operating level of 260.4 m (854.3 ft).

Table 13.12
Simulated 2011 Lake Level Results for Various Bed Lowering Scenarios

Depth of Excavation in Mossy River (m)	Options		
Depth of Excavation in Mossy River (m)	0.3	0.5	1.0
Reduction in 2011 Peak Lake Level (m)	0.29	0.38	0.6
Peak Outflow in Mossy River	120 m^3/s (4,240 cfs)	126 m^3/s (4,450 cfs)	138 m^3/s (4,870 cfs)
Volume of Channel Excavation (m^3)	360,000	600,000	1,200,000
Excavation Cost [(1)]	$18,000,000	$30,000,000	$60,000,000
Environmental Mitigation Costs (Major – 15% of Direct Costs)	$2,700,000	$4,500,000	$9,000,000
Indirect Cost (Owners Cost, Engineering, Approvals, Administration) (20% of Direct Costs)	$3,600,000	$6,000,000	$12,000,000
Contingency Allowance (30% of All Costs)	$7,290,000	$12,150,000	$24,300,000
Total	$31,590,000	$52,650,000	$105,300,000

1 Based on an assumed unit price of excavation of $50.00/$m^3$. A further study of soil conditions in the channel and method of construction would be required to better define the appropriate unit cost.

Alternatively, to have maintained the lake levels below El 260.4 m (El 854.3 ft) during the 2011 Flood would require that the bed of the Mossy River be lowered by approximately 3.0 m (9.9 ft) and the effective width of the existing outlet structure increased to 74 m (243 ft). This would increase the Mossy River flows to about 206 m^3/s (7,275 cfs) at the peak of the 2011 Flood. This is well above the estimated bank full discharge of 150 m^3/s (5,297 cfs) – resulting in flooding downstream of the excavated reach and likely producing massive instabilities in the channel. Furthermore, a 3 m (9.9 ft) reduction in river bed levels would adversely affect the regime of the river, and as well, require that drop structures be constructed on the two major tributaries entering the Mossy River to mitigate the effects of the lower bed levels. Due to the complications associated with this proposed option, it was deemed not feasible and was beyond the scope of this report to go into the detail required to estimate its total cost.

Major environmental mitigation costs would be expected for the channel improvement schemes regardless of the proposed depth of excavation in the Mossy River. It is likely that a significant amount of fish habitat would be destroyed or altered as part of this scheme. This could result in opposition from local stakeholders including the recreational and commercial fishing industry in the area. Other environmental impacts, such as changes in the flow patterns in the river could also be considered harmful under the *Canadian Fisheries Act*. For these reasons, the river channel excavation options have not been considered feasible flood mitigation solutions for Dauphin Lake.

13.6 EFFECTS ON LAKE WINNIPEGOSIS

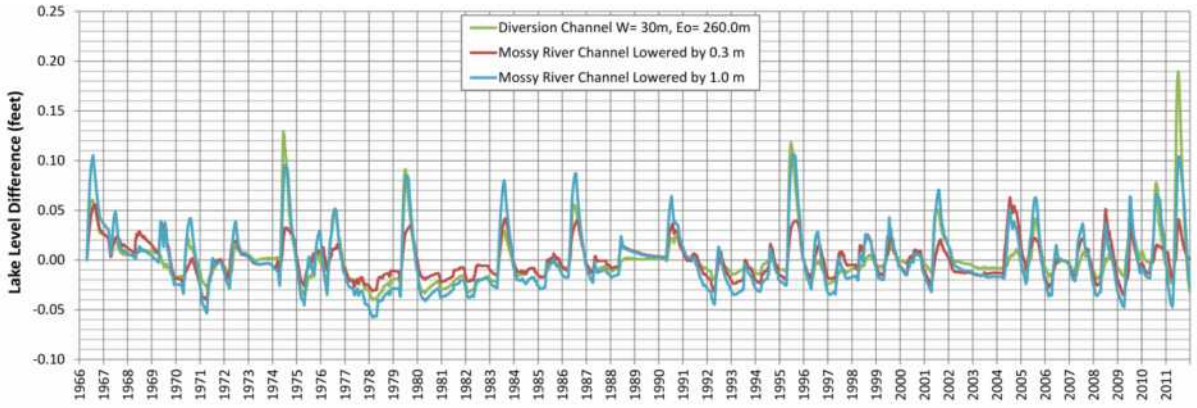
As mentioned earlier, reduction of the available storage volume in Dauphin Lake would result in higher inflows to Lake Winnipegosis during flood events than have occurred under current conditions. As a first approximation, given the relative surface areas of the two lakes, a 1 m (0.3 ft) decrease in storage in Dauphin Lake will produce about a 0.1 m (0.3 ft) increase in the levels of Lake Winnipegosis, if the concurrent increases in outflows from Lake Winnipegosis (due to rising lake levels) were not accounted for. Therefore, a 0.6 to 0.8 m (2.0 to 2.6 ft) decrease in water levels that would be associated with a 30 m (98 ft) wide auxiliary outlet set a sill elevation of 260.0 m (853 ft) would produce, at most, an increase in lake levels of 0.055 and 0.072 m (0.18 and 0.24 ft), respectively. If additional outflow capacity had been provided to maintain 2011 lake levels below 260.4 m (854.3 ft), the level of Lake Winnipegosis would have been, at worst, about 0.2 m (0.7 ft) higher than what was actually recorded.

Simulations using a rigorous water balance model that accounts for both the increase in outflow from Dauphin Lake as well the outflows from Lake Winnipegosis were undertaken. The objective was to determine the effects of the improvements at Dauphin Lake discussed above on Lake Winnipegosis water levels. Four scenarios were simulated:

1. 30 m (98 ft) wide weir at a sill elevation of 260.0 m (853.0 ft), along with the Short Diversion Channel.
2. Mossy River channel lowered by 1.0 m (3.3 ft).
3. Mossy River channel lowered by 0.3 m (1.0 ft).
4. 60 m (196.9 ft) wide weir at a sill elevation of 258.89 m (849.4 ft), along with the Short Diversion Channel.

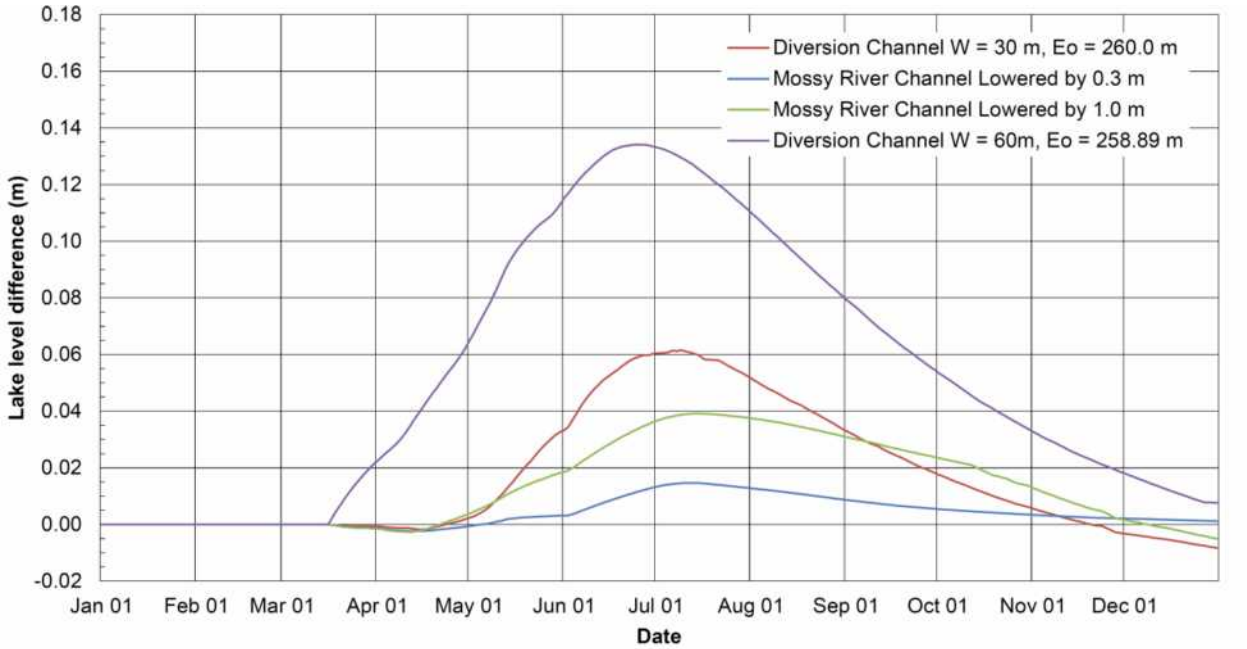
Figure 13.21 presents the results of the simulations for the 1966 to 2011 period for the first three scenarios. Water level increases would have been between 0.015 and 0.06 m (0.05 and 0.2 ft) on Lake Winnipegosis, with the highest increase occurring in 2011, followed by 1974 and 1995. Note that Figure 13.21 is presented in imperial units (feet). Figure 13.22 compares the increases in Lake Winnipegosis water levels that would have occurred for each of the four scenarios had they been operational in 2011. The much higher inflows required to maintain Dauphin Lake water levels below 260.4 m (854.3 ft), as affected by the 60 m (196.9 ft) wide weir at crest El 260.0 m (853.0 ft), would have resulted in a water level increase of 0.13 m (0.43 ft) on Lake Winnipegosis.

Figure 13.21
Impacts of Improvement from Outflow from Dauphin Lake on Lake Winnipegosis Lake Levels



Note: Levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 13.22
2011 Simulation of Lake Winnipegosis Water Levels
for Selected Dauphin Lake Outflow Options



Note: Water levels can be converted to feet by dividing metric values by 0.3048.

13.7 SUMMARY OF ESTIMATED COSTS

Table 13.13 provides a summary of a potential reduction in the peak water levels of the 100-year flood and the associated cost each of the alternatives that were evaluated.

Table 13.13
Summary of Regulation Alternatives and Estimated Costs

Alternative	Reduction of 100-year Lake Level (m)	Total Estimated Cost (million $)	Comments
Excavate Mossy River by 0.3 m	0.29 (0.95 ft)	31.6	-
Excavate Mossy River by 0.5 m	0.38 (1.25 ft)	52.7	-
Excavate Mossy River by 1.0 m	0.60 (1.97 ft)	105.3	Maybe the maximum excavation possible without encountering channel stability issues – this speculation needs to be confirmed
Short Channel (30 m wide weir with sill level at 260.0 m)	0.70 (2.30 ft)	143.5	Most optimal route but could have easement difficulties
East Channel (30 m wide weir with sill level at 260.0 m)	0.70 (2.30 ft)	211.4	Follows section lines to potential easement problems
West Channel (30 m wide weir with sill level at 260.0 m)	0.70 (2.30 ft)	322.8	Major impact on Fork and Fishing rivers
Short Channel (60 m wide weir with sill level at 258.87 m)	1.94 (6.36 ft)	253.5	Most optimal route but could have easement difficulties

13.8 SEDIMENTATION IN DAUPHIN LAKE

The deposition of sediment in Dauphin Lake is an important issue to users of the lake. This is mostly from the perspective of the water quality in the lake, but also because sediment inflow and its subsequent deposition are perceived to potentially reduce the storage capacity of the lake and adversely affect the ability of the lake to attenuate flood outflows. The effect of incoming sediment on lake storage is examined in this section. Consideration of water quality impact is beyond the terms of reference of this study.

Suspended sediment data has been collected by WSC on six streams entering the lake (Table 13.14). The data represent different time periods, different basin characteristics (broadly characterized as upland and lowland), and a range of contributing watershed areas. However, the data provide a reasonable basis upon which to estimate the annual volume of sediment delivered into the lake from surrounding catchments.

Table 13.14
Suspended Sediment Measurements by WSC in the Dauphin Lake Watershed

Watershed	Period of Record	Number of Measurements	A	b	r^2
Ochre River	1980-82 1990-93	27	193.8	0.551	0.42
Turtle River	1966-82	140	309.7	0.346	0.15
Valley River	1981-87	366	5.0	0.884	0.77
Wilson River	1980-82	26	53.9	0.746	0.31
Vermilion River	1970-82	248	273.6	0.457	0.38
Edwards Creek	1969-79	147	294.9	0.789	0.39

Suspended sediment concentrations typically increase in proportion to the discharge. When plotted on a logarithmic chart, the relationship between the discharge and the suspended sediment rating curve can be represented by a power relationship of the form:

$$C = A\,Q^{b}$$

Where:

Q is the discharge (m^3/s);

C is the suspended sediment concentration (mg/L); and

A and b are best fit coefficients.

The individual power relationships for each of the above watersheds are shown in Appendix I and the coefficients that best fit the relationships are summarized in Table 13.14.

The geomorphic characteristics of the gauged catchments are expected to approximate the same characteristics of the ungauged areas of the drainage basin. On this basis, the annual measured sediment input to Dauphin Lake can be scaled up by a factor of approximately 1.5 to account for contributions from the ungauged parts the watershed. With the inclusion of estimates of sediment from the ungauged areas, the total influx of sediment to the lake is estimated to be 300,000 tonnes (330,000 tons) per year, on average.

The density of the deposited sediment is a function of the sediment gradation and the age of the deposit. The Turtle and Vermilion rivers contribute the largest percentage of annual sediment yield and data from WSC indicates that the suspended sediment has an average gradation of 40% clay, 50% silt, and 10% sand. An empirical relationship developed by Lara and Pemberton (1963) can be used to calculate the initial specific weight of the initial deposits. The empirical equation developed by Miller (1953) can be used to calculate the compaction over time. These analyses indicate that the average specific weight of the deposits after 100 years would be approximately 1,080 kg/m^3 (67 lb/ft^3). The annual volume lost to the storage of water would, therefore, be 280 dam^3 (230 ac-ft) – or about 0.018% of the total volume of the lake when the water level is at El 260.54 m (854.79 ft). Over a 100-year period, the total loss in storage would amount to about 1.8% of the current lake volume. From another perspective, the sediment influx could account for an average increase in the elevation of the lake bed by some 0.6 mm (0.25 in) on an annual basis and 6 cm (2.4 in) over a 100-year period. Adopting the worst-case upper bound of the suspended sediment rating curves results in an annual storage loss of some 1,120 dam^3 (910 ac-ft) and a lake bed increase of 24 cm (0.79 ft) over the next 100 years.

The sedimentation rate from the 1990 study (MWRB, 1990) was estimated to be about 6 cm/100 yrs (2.4 in/100 yrs) over the full area of the lake – identical to that estimated herein when the best fit relationship between the discharge and the suspended sediment concentration is adopted. This would not significantly impact on flood management in Dauphin Lake.

It should be acknowledged that the majority of sediment deposition is likely to occur as deltaic formations near the mouths of the tributaries that release into the lake. The development of such deposits may result in the perception that the volume of the deposition on a lake-wide basis is more extensive than it really is.

13.9 CONCLUSIONS FROM STUDIES OF DAUPHIN LAKE

- **Upstream Storage Development –** This is not practical due to:
  - The limited number of storage sites available on streams feeding into the lake.
  - The small size of the potential storage sites.
  - The need to construct numerous impoundments on a large number of streams of comparable size that feed into the lake.

It is estimated that about 11,000 ha (27,182 ac) of reservoir area would need to be constructed within the basin to reduce the 2011 lake level by 0.5 m (1.6 ft). This would be equivalent to constructing about 33 large reservoirs / dams on the scale of the Pleasant Valley Dam, or 156 small reservoirs / dams on the scale of the Vermilion Dam throughout the various catchments upstream of the lake. To maintain the 2011 lake level within the adopted operating range of 260.0 to 260.4 m (853.0 to 854.3 ft) would require an order of magnitude more reservoirs. It has been concluded that increasing upstream storage is not a reasonable strategy.

- **Auxiliary Diversion Channel to Increase Outflows to Lake Winnipegosis –** Analysis indicates that a new diversion channel would need to carry about as much flow as the Mossy River to lower the large historical peaks to tolerable levels. Four potential diversion options in four different settings were selected to assess the full range of possible routes:

  - 14.7 km (9.1 mi) long Short Channel located east of the Mossy River.

  - 19.6 km (12.2 mi) long East Channel, also located east of the Mossy River.

  - 7.1 km (4.4 mi) long Tributary Channel adjacent to Mossy River (Outlet from the lake would be adjacent to the Mossy River near the existing structure, and the diversion channel would intersect the Mossy River at a point about 10 km downstream of Dauphin Lake).

  - 27.2 km (16.9 mi) long West Channel located west of the Mossy River and aligned more or less parallel to it.

- Regardless of which route is selected, there would be a need to excavate through the ridge of high ground north of Dauphin Lake with cuts of about 10 to 14 m (33 to 46 ft) over a distance of several kilometres to provide a channel invert level low enough to achieve sufficient outflow from the lake. Depending on the route selected, the appropriate channel slope could be achieved with between one and three drop structures to accommodate about 7 m (23 ft) of excess grade between Dauphin Lake and Lake Winnipegosis. The Short Channel appears to be the most economical diversion channel alternative. Complete with an inlet structure on Dauphin Lake, one drop structure, and an outlet structure on Lake Winnipegosis, it could provide a similar 0.7 m (2.3 ft) water level reduction at a cost of $144 million. Expanding the capacity of the Short Channel to maintain the 2011 lake level below El 260.4 m (El 854.3 ft) would cost $254 million. While costs may theoretically be reduced somewhat by lowering the sill elevations of the inlet structures, consideration of the shallow depths of the near-shore area may limit how low the sill levels can be constructed. Selection of a sluice-type of control structure as opposed to the vertical drop structure on which all the alternatives herein were based, could reduce the costs modestly and should be examined at the next level of engineering.

- **Improve Conveyance of Mossy River –** Lowering the channel bed of the Mossy River is believed to be the most effective way to increase its discharge capacity. A 1 m (3.3 ft) reduction in bed levels in the upper 10 to 15 km (6.2 to 9.3 mi) of the river could reduce the 100-year water level by about 0.5 m (1.6 ft), even without expanding the existing control structure. Additional excavation may tend to destabilize the river banks and would only be recommended pending a detailed geotechnical investigation. The

estimated cost of this option with 1 m (3.3 ft) of deepening (excluding the cost of any slope stabilization that may ultimately be proven to be necessary) is estimated to be $105 million. To maintain the lake level within the normal target range with a repeat of the 2011 Flood, would require a 3 m (9.8 ft) deep channel excavation – essentially doubling its average depth – and an expansion of the control structure from its existing effective width of 36.6 m to 74 m (120.1 to 242.8 ft). The costs of this option were not investigated due to the difficulties in evaluating a large number of adverse effects that would be caused. Additionally, due to the significant potential environmental impacts associated with river channel excavation works, the river channel improvement schemes were not considered feasible flood mitigation solutions for Dauphin Lake.

There are clearly a number of options available to increase the regulation of Dauphin Lake, but at a significant cost. Additional analysis could identify cost effective synergies between increasing the discharge capacity of the Mossy River and constructing an optimally sized auxiliary diversion channel. Regardless of which strategy may be followed, additional control of flood levels in Dauphin Lake could increase water levels on Lake Winnipegosis by at most about 0.2 m (0.7 ft).

Finally, with respect to the effects of sediment deposition in the lake, analysis indicates that the sedimentation rate is essentially the same as was determined in earlier analyses done in the 1990s (MWRB, 1990). Sediment inflows appear to be decreasing the storage volume in the lake by about 280 dam^3 (230 ac-ft) or 0.018% of the lake volume per year. This will not have a significant effect on the long-term management of floods. The majority of this deposition is believed to be occurring as deltaic formations near the mouths of the tributaries to the lake. The reduction in depth in the vicinity of these deposits may be perceived to be serious, but it does not accurately represent the actual deposition on a lake-wide basis.

14.0 LAKE WINNIPEGOSIS

14.1 OVERVIEW

Lake Winnipegosis is the major source of inflows to Lake Manitoba. When levels are high on Lake Winnipegosis, the outflows through its outlet at the Waterhen River remain high, often resulting in flooding on Lake Manitoba. Furthermore, Lake Winnipegosis has a small outlet relative to its drainage area. As a result, the lake responds slowly to changing inflows. When the water levels on Lake Winnipegosis are high, as a result of prolonged heavy runoff, it can take months and even years for the lake to recede to its normal level. During this period, the outflows through the Waterhen River remain high, resulting in prolonged periods of above normal inflows to Lake Manitoba. If some of the water stored in Lake Winnipegosis during these high inflow events could be controlled either by a diversion out of the lake or by operation of a control structure on the Waterhen River, then flooding on Lake Manitoba and Lake St. Martin could be reduced. This section summarizes studies that have examined these potential mitigation works. It addresses the hydrology of the area and analyses the potential hydraulic impact of the mitigation works on Lake Winnipegosis, Lake Manitoba, and Lake St. Martin.

14.2 FLOOD DIVERSION TO CEDAR LAKE

An option for flood protection at Lake Manitoba is a diversion channel from Lake Winnipegosis to Cedar Lake (Saskatchewan River), which would convey excess flood waters during severe events. Cedar Lake is the reservoir for Manitoba Hydro's Grand Rapids Hydroelectric Generating Station. A diversion channel at the narrow land that separates Lake Winnipegosis and Cedar Lake has been studied previously by Manitoba Water Resources Branch as early as 1950. Information from that study (MWRB, 1955), although not extensive, was useful in confirming the general aspects of the schemes devised in the current study.

This section summarizes the key aspects of the study findings, including assumptions, analyses of operation and cost.

The concept takes advantage of the existence of a relatively narrow separation of land between Lake Winnipegosis and Cedar Lake (approximately 5.8 km (3.6 mi) in width), and the potential

advantage of occasional transfers of excess water north to the Saskatchewan River system. If excess flood water could be diverted from Lake Winnipegosis to Cedar Lake, it could reduce flood conditions on Lakes Winnipegosis, Manitoba and St. Martin.

The land separating Lake Winnipegosis from Cedar Lake / Saskatchewan River varies from 5.5 to 8.0 km (3.4 mi to 5.0 mi) in width, and rises more than 15 m (49 ft) above the elevations of the two lakes. This creates a substantial blockage that would require significant excavation quantities to form a viable diversion channel. Furthermore, the water level on Cedar Lake is consistently higher than the water level on Lake Winnipegosis, by an average of about 1 to 2 m (3.3 to 6.6 ft) during the period that diversion of floods would be required. Substantial pumping facilities would be required to make the scheme workable. The additional water that would be pumped to the Saskatchewan River would theoretically be available for additional energy generation at the downstream Grand Rapids Hydroelectric Generating Station. Each of these factors has been addressed in the following sections of the report.

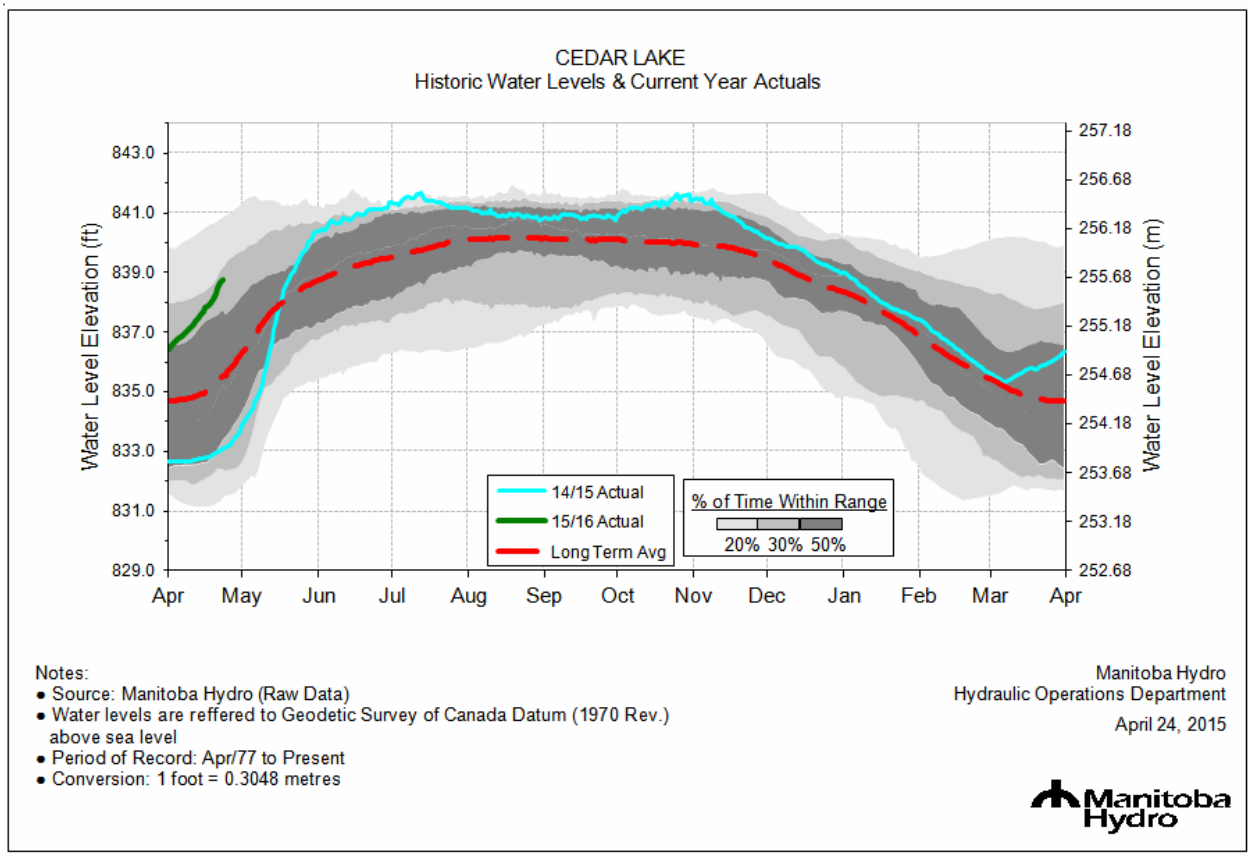
14.2.1 Location and Key Factors

Drawing 14.1 shows on a map the narrow land that separates Lake Winnipegosis from Cedar Lake.

From 1961 to 2012 the range of static water levels on Lake Winnipegosis (wind effects eliminated) was from El 252.4 m (El 828 ft) up to El 254.4 m (El 834.65 ft). In 2011, the static lake level peaked at El 254.4 m (El 834.65 ft).

As explained above, Cedar Lake is a reservoir that serves as the head pond for Manitoba Hydro's Grand Rapids Hydroelectric Generating Station. The water level on Cedar Lake typically ranges from approximately El 253.7 (El 832.3 ft) to El 256 m (El 840 ft). Figure 14.1 shows the range of Cedar Lake levels as depicted by Manitoba Hydro (MBH, 2015). In the spring flood season, the water level on Cedar Lake is typically at about El 255 m (El 836.6 ft), and is about 1.8 m (5.9 ft) above the water level on Lake Winnipegosis.

Figure 14.1
Cedar Lake Historic Water levels



The narrowest and lowest height of land between the lakes is located at “East Mossey Portage” as indicated on Drawing 14.1. Figure 14.2 in Section 14.2.2 shows a profile of the land between the lakes along this route. Fortunately for this study, the Manitoba Water Resources Branch investigated this route in 1950 as part of the “Dauphin River Power Survey”, and undertook subsurface investigations. The soil along the route was consistently proven to be clay with some boulders. No bedrock was found above El 243 m (797.2 ft) at any location along this route (MWRB, 1955).

14.2.2 General Arrangement of Diversion Concept

Diversion of excess water to Cedar Lake to relieve flood conditions on Lake Winnipegosis would require a channel approximately 5.9 km (3.7 mi) in length, and a flood pump station to lift water from Lake Winnipegosis and allow it to flow in the diversion channel to Cedar Lake. The most

cost-effective configuration would be installation of a flood pump facility at the south end of the channel near Lake Winnipegosis. The water would be lifted, to a channel that would have enough gradient northward to develop velocities that would minimize the volume (and cost) of excavation. Detailed optimization of larger channels with lesser gradients and reduced operating head at the pump station was not pursued in depth. Similarly, a channel with a higher bed elevation (correspondingly greater pump lifts), combined with strategically located drop structures in the channel was addressed briefly. This concept was shown to significantly reduce the excavation costs, but the increase in cost due to the larger pump station / power supply and the additional costs of drop structures resulted in a substantial overall cost increase. Further optimization was not considered necessary at this screening level of analysis.

Since the channel would be cut entirely in clay (some boulders), a channel cross section that is trapezoidal with side slopes of 1V: 5H was selected for planning and budgetary purposes. It is possible that further analysis with soil information from the area would reveal that this is too steep for long-term stability. However, at this point, an optimistic design was selected. Similarly, for the purposes of this screening level of studies, it was assumed that the diversion channel would be designed for open water conditions only.

Three options were addressed, one for each of the following discharge capacities: 70 m^3/s (~2,500 cfs), 140 m^3/s (~5,000 cfs), and 283 m^3/s (10,000 cfs).

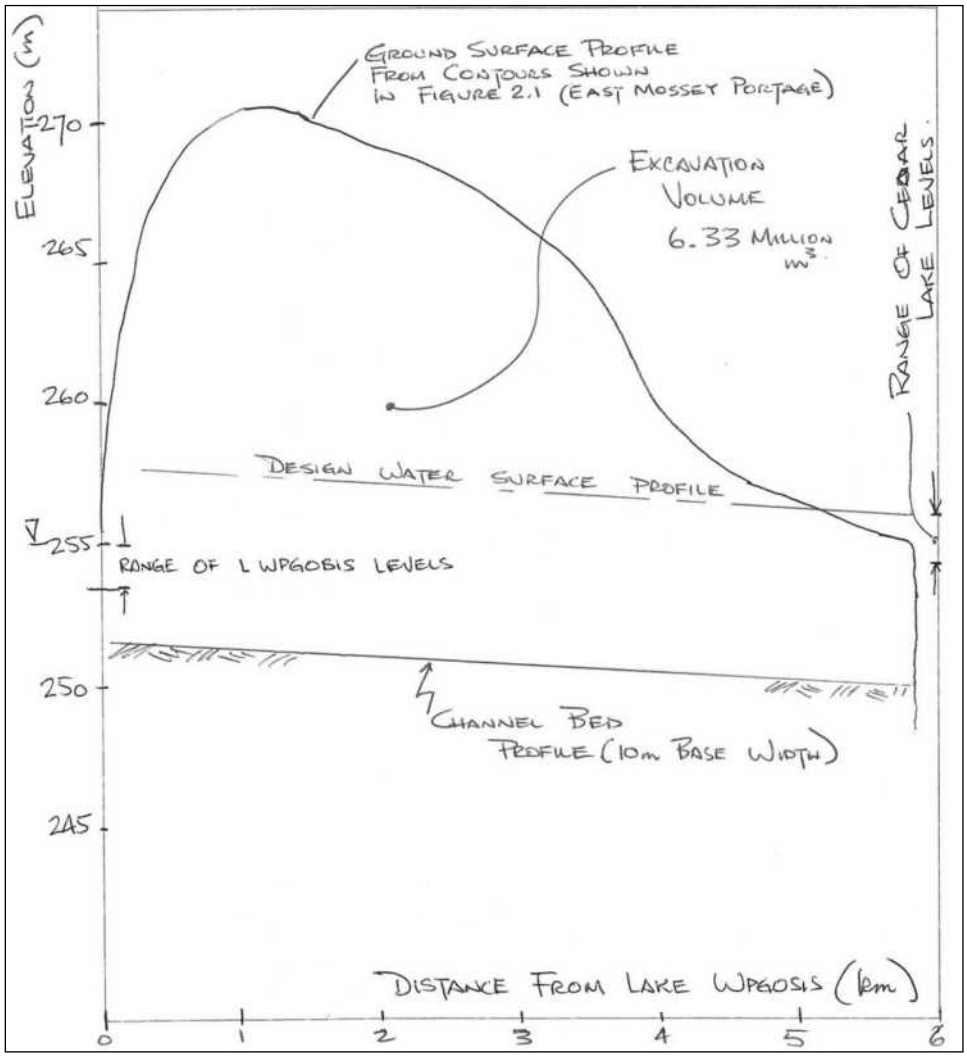
The selected upper limit of 283 m^3/s (10,000 cfs) is:

- Approaching 50% of the average annual flow of the Saskatchewan River.
- About 10% of the historical maximum recorded flow in the Saskatchewan River at this location.
- About 5% of the design inflow that was used for the spillway design criteria at the Grand Rapids Hydroelectric Dam.

The upper limit of 283 m^3/s (10,000 cfs) appears to be a reasonable choice for the current screening level of analysis.

Figure 14.2 shows a profile of the concept devised for the case with the largest discharge capacity of 283 m^3/s (10,000 cfs). Details and estimated construction costs of the three schemes are summarized in Tables 14.1, 14.2, and 14.3 of Section 14.2.4.

Figure 14.2
Diversion Route along East Mossey Portage



The intent of the diversion would be to reduce high water levels on Lake Winnipegosis without substantially changing the lower end of the normal fluctuation of levels of that lake. The primary focus of the concept was to consider operation of the diversion only when the level of Lake Winnipegosis would reach a clearly abnormally high water level. The initial analyses were based on an assumed activation of the diversion when the water level of Lake Winnipegosis would reach the upper 90 percentile thresholds approximately El 253.4 m in winter and El 253.8 m in

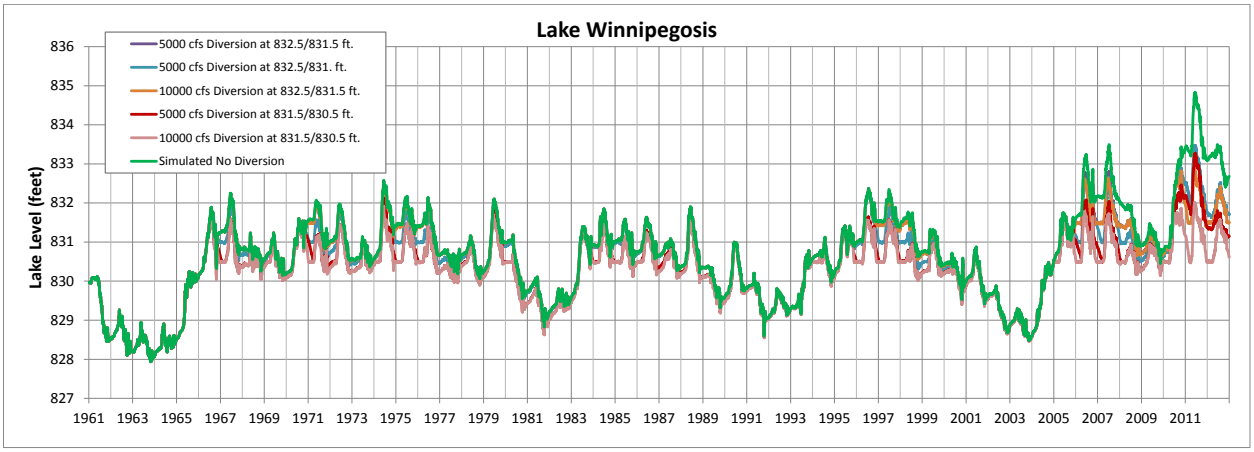
summer (approximately El 831.5 ft in winter and El 832.5 ft in summer), and return the lake to, or below, those thresholds as quickly as possible. More aggressive schemes were also investigated, assuming an activation level on Lake Winnipegosis as low as El 253.1 m (El 830.5 ft) in winter (currently exceeded about 50% of the time), and El 253.4 m (El 831.5 ft) in summer (currently exceeded about 30% of the time).

14.2.3 Hydrological Studies

As discussed previously, a range of options was investigated from the hydrologic perspective to understand what the frequency of use of this concept would be and what potential reduction of lake levels would be possible in the three affected lakes (Winnipegosis, Manitoba, St. Martin). Details of the hydrologic model are documented in Section 8.2.1.

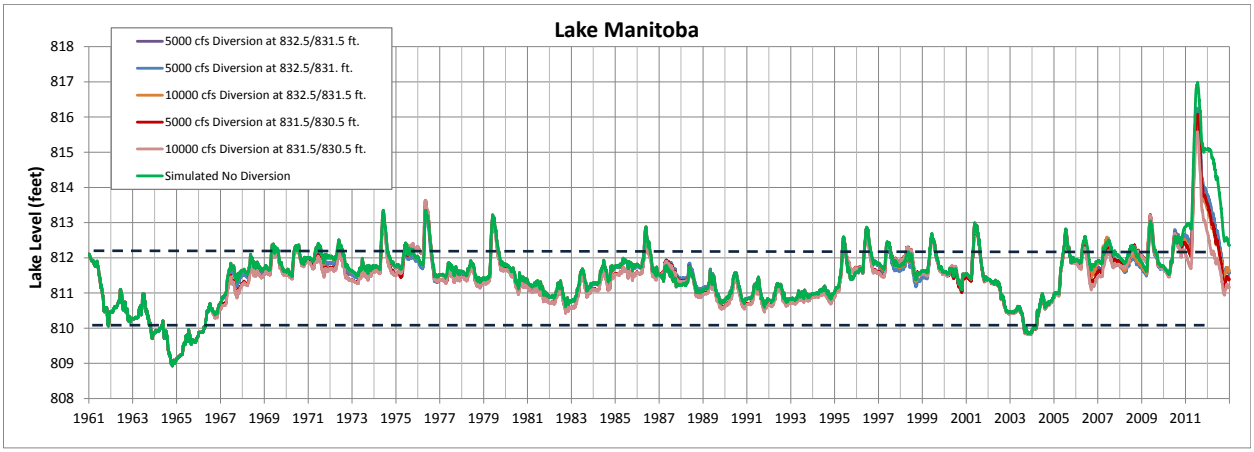
Figures 14.3 to 14.10 demonstrate the results of the hydrological studies in terms of the effects on water levels on Lake Winnipegosis, Lake Manitoba and Lake St. Martin. They demonstrate what the water levels on the three lakes were historically, and what they would have been if the diversion concepts had been installed for that period of record. Figures 14.3, 14.4 and 14.5 show the computed water levels for each of the lakes from 1961 to 2012. Figure 14.6 shows the same data on Lake Manitoba as Figure 14.4 but for the last decade only. Figures 14.7, 14.8 and 14.9 are duration curves based on hydrological records from 1961 to 2012, inclusive. Figure 14.10 shows the same data on Lake Manitoba as Figure 14.8, but for the 80% to 100% range. The legends in each of Figures 14.3 to 14.10 indicate the discharge capacity of each option and the water levels in Lake Winnipegosis above which the channel would be used in summer and in winter (respectively). Though the concept of a 70 m^3/s (~2,500 cfs) channel option was investigated, the resultant water levels for this option have not been shown on the figures as the impacts to water levels would be less.

Figure 14.3
Lake Winnipegosis Water Levels with Diversion



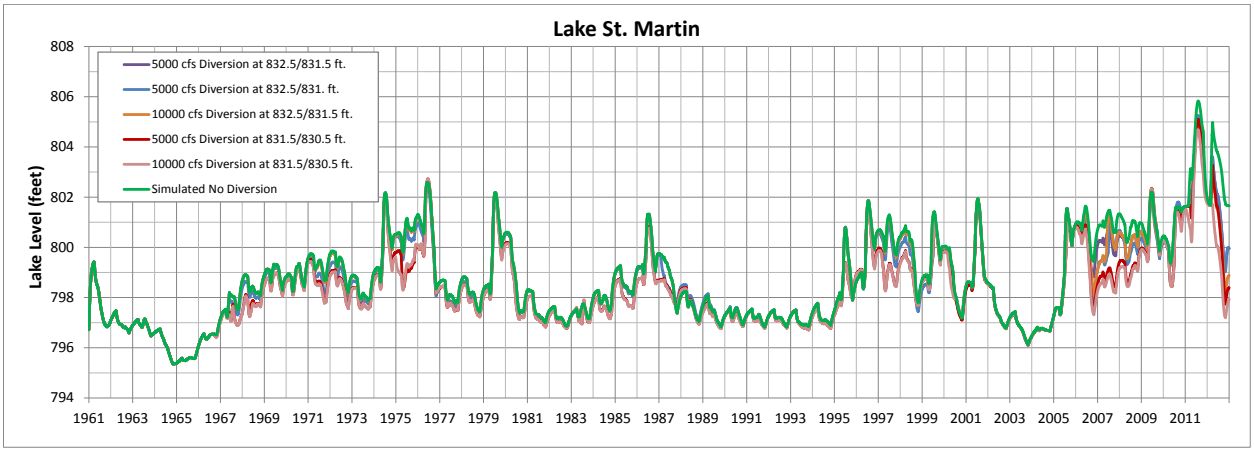
Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.4
Lake Manitoba Water Levels with Diversion



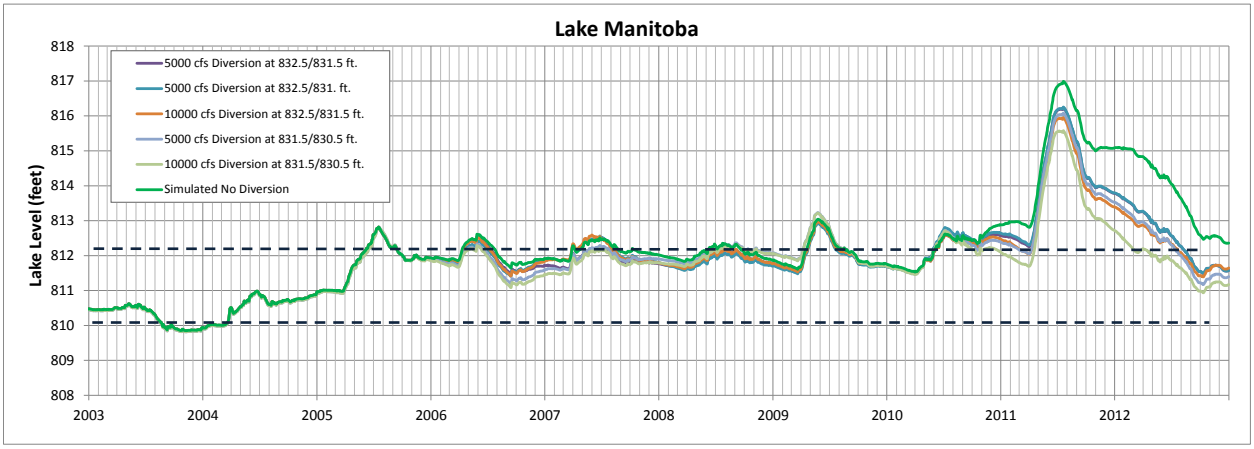
Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.5
Lake St. Martin Water Levels with Diversion



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.6
Lake Manitoba Water Levels with Diversion 2003 to 2012



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.7
Lake Winnipegosis Peak Levels with Diversion

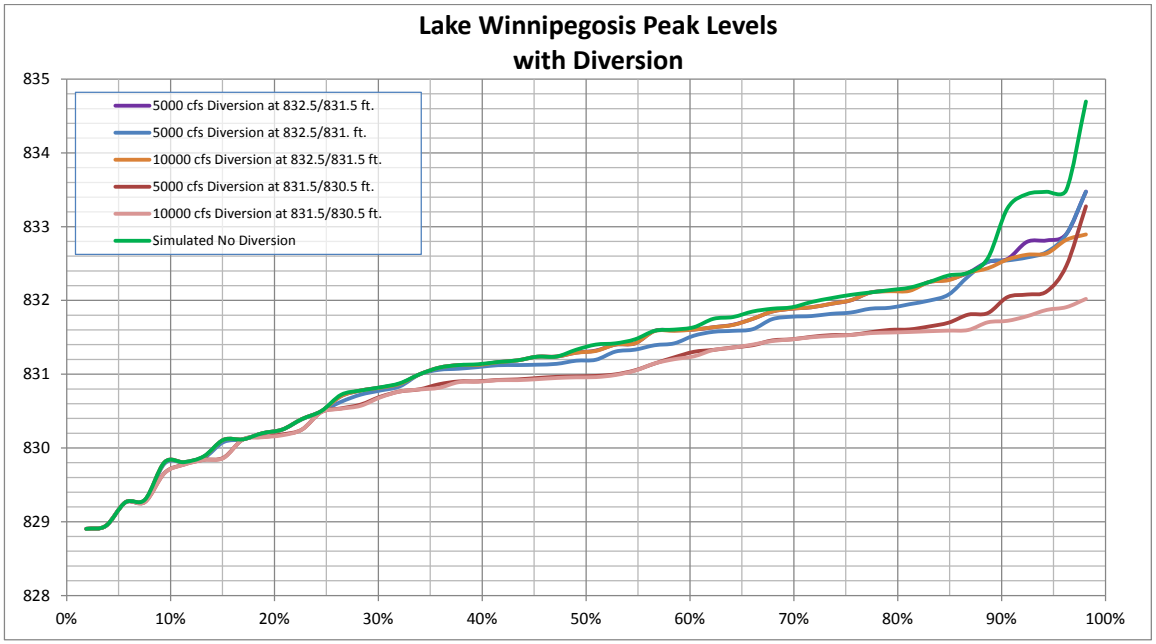
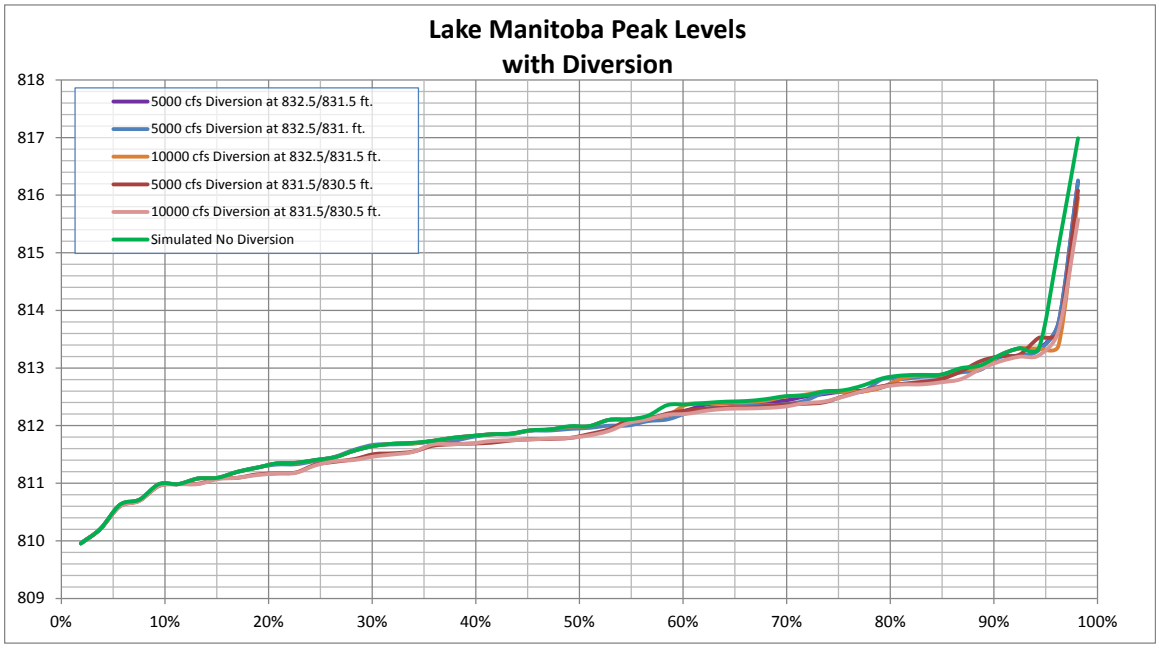


Figure 14.8
Lake Manitoba Peak Levels with Diversion



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.9
Lake St. Martin Peak Levels with Diversion

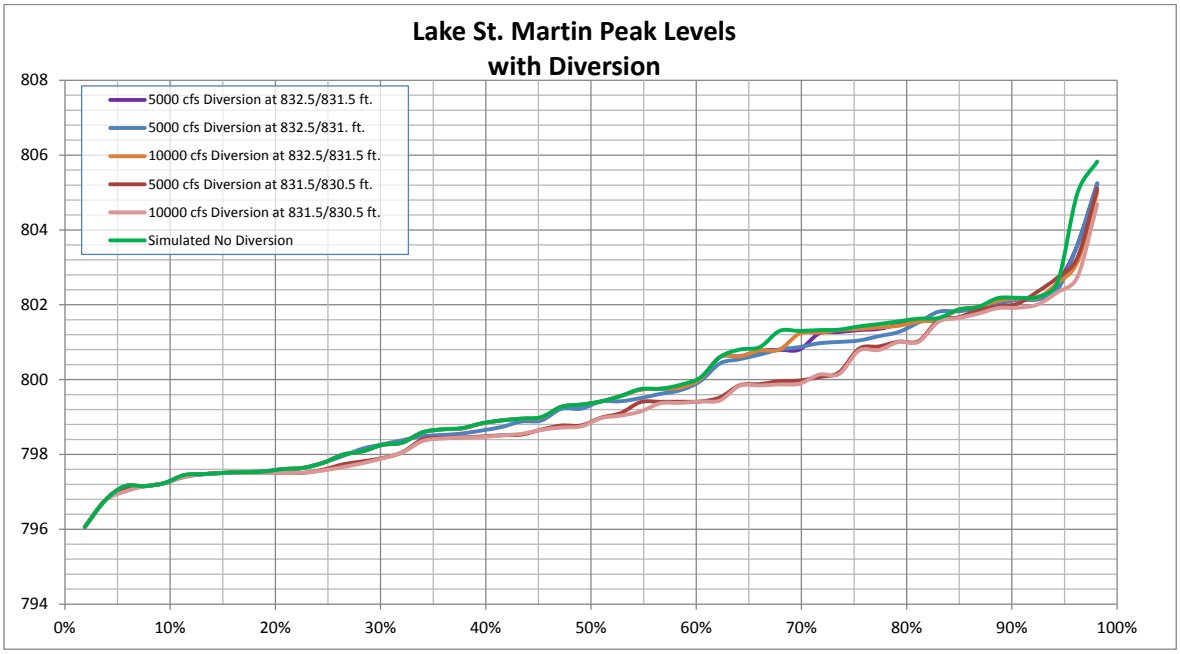
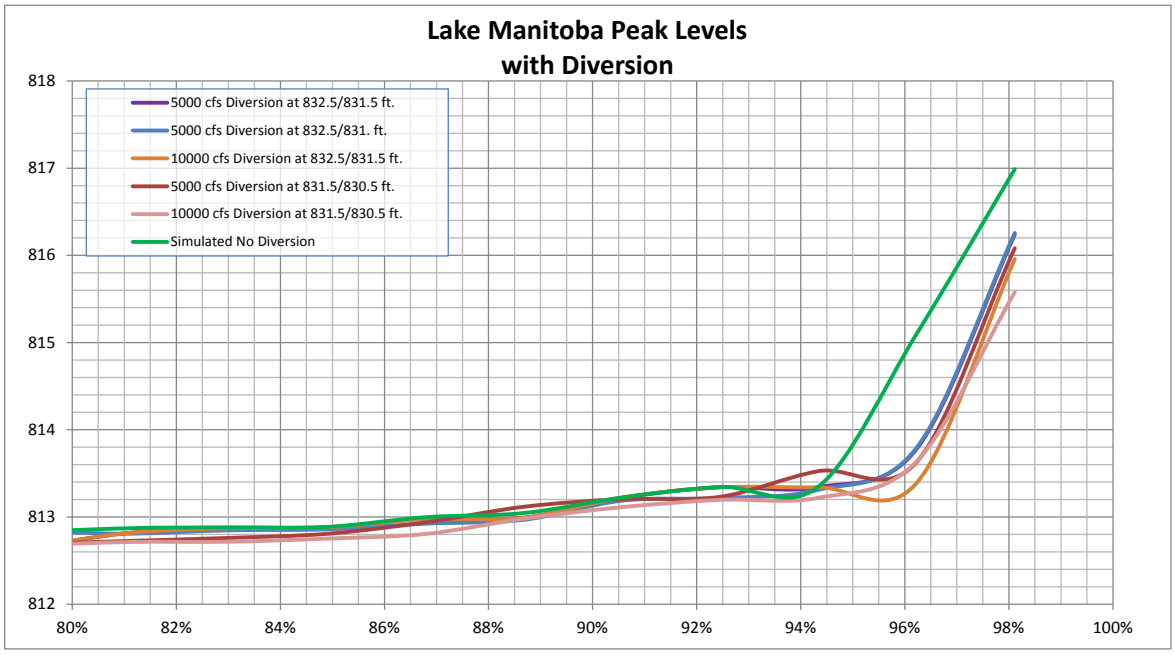


Figure 14.10
Lake Manitoba Peak Levels with Diversion 80% to 100%



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

For example, the greatest reduction in water level of Lake Manitoba was shown to be in 2011. The diversion scheme with a maximum capacity of 283 m^3/s (10,000 cfs), with relatively aggressive activation levels on Lake Winnipegosis of El 253.4 m (El 831.5 ft) (summer) and El 253.1 m (El 830.5 ft) (winter), was estimated to cause a reduction by approximately 0.4 m (1.34 ft) on Lake Manitoba.

As discussed in Section 5.2.2, heavy rains in the summer of 2014 resulted in yet another significant flood event within the Assiniboine River & Lake Manitoba Basins. On Lake Winnipegosis, peak water levels (wind eliminated) reached about El 254.3 m (El 834.3 ft), which was about 0.2 m (0.7ft) lower than in 2011. On Lake Manitoba, peak water levels (wind eliminated) reached about El 248.3 m (El 814.6 ft), which was about 0.8 m (2.5 ft) lower than in 2011. Due to the lower water levels during the 2014 Flood compared to 2011, the diversion scheme on Lake Winnipegosis would have likely had a lower reduction capability on Lake Manitoba in 2014 compared to 2011. On this basis, including the 2014 Flood into the analysis would not have changed the conclusions for this option.

14.2.4 Cost Estimation

Cost estimation was conceptual at this screening stage of analysis. Estimated capital costs and other pertinent information are summarized in Tables 14.1, 14.2 and 14.3. A key feature of this diversion concept is that pumping is required. As explained above, this is due to the fact that the water level on Lake Winnipegosis is typically 0.5 to 1.5 m (1.5 ft to 5 ft) lower than that of Cedar Lake where the diverted water would be released into the Saskatchewan River system. A detailed accounting of peripheral facilities for the pump station, such as an approach channel and release pool, has not been attempted. Rather, an estimate equivalent to 10% of the total direct costs of the option was considered sufficiently representative to cover these items at this conceptual level of assessment. It should be noted that the capital costs of this mitigation scheme are not affected by the operational assumptions of the trigger level on Lake Winnipegosis for initiation of diversion.

The cost estimates have also included an allowance for the environmental mitigation works that would be required to construct this scheme and address potential adverse environmental impacts. Due to the potential impacts of inter-basin water transfers from Lake Winnipegosis to

Cedar Lake, the changes to the natural Lake Winnipegosis and Cedar Lake water level regime and the presence of numerous stakeholders including First Nations communities around both lakes, environmental mitigation costs were optimistically assumed to be moderate. Although not reflected in the cost estimates, Manitoba Hydro would have to seek and obtain permission to expand its relatively narrow range of water levels on Cedar Lake.

It must be acknowledged that there would likely be significant opposition to the scheme, and the environmental assessment would almost certainly be long and protracted. It is conceivable that in the end, approval might not be granted.

Table 14.1
Summary of Design Parameters and Estimated Capital Costs for Diversion Capacity of 70 m^3/s (2,500 cfs)

Item	Description	Cost (where pertinent)
Channel Invert El at Lake Winnipegosis	El 254.9 m (El 836.3 ft)	
Channel Invert El at Cedar Lake	El 251 m (El 820.2 ft)	
Channel Base Width	5.0 m (16.4 ft)	
Channel Side Slopes	1V:5H	
Design Depth of Flow (open water)	3.0 m (9.8 ft)	
Bed Slope	1 m per 1500 m (1 ft per 4,920 ft)	
Maximum Velocity	1.2 m/s (3.9 ft/s)	
Volume of Excavation	4,200,000 m^3 @ \$10/m^3	\$42,000,000
Manning's n-value	0.03	
Revegetation	278.4 ha @ \$5,500/ha	\$1,531,000
Bridge (PR 60)	\$3 M (approaches) + 1,980 m^2 @ \$7,000/m^2	\$16,800,000
Maximum Pump Lift Required	5 m (16.4 ft)	
Pump Power	4.9 MW	
Pump Sta. Cost	\$4,000 per Kw	\$19,600,000
Diesel Power Station	\$1,500 per Kw	\$ 7,400,000
Peripheral Facilities	10% of Total Direct Costs	\$8,739,000
Environmental Mitigation Costs	10% of Total Direct Costs	\$9,613,000
Indirect Costs (Owner's Costs, Engineering, Administration, Approvals)	20% of Total Direct Costs	\$17,478,000
Contingency Allowance	30% of Total Direct and Indirect Costs	\$36,966,000
	Total Cost	**\$160,188,000**

Table 14.2
Summary of Design Parameters and Estimated Capital Costs for Diversion Capacity of 140 m^3/s (5,000 cfs)

Item	Description	Cost (where pertinent)
Channel Invert El at Lake Winnipegosis	El 253.4 m (El 831.4 ft)	
Channel Invert El at Cedar Lake	El 251 m (El 823.5 ft)	
Channel Base Width	5 m (16.4 ft)	
Channel Side Slopes	1V:5H	
Design Depth of Flow (open water)	4.35 m (14.3 ft)	
Bed Slope	1 m per 2415 m (1 ft per 7,920 ft)	
Maximum Velocity	1.2 m/s (3.9 ft/s)	
Manning's n-value	0.03	
Volume of Excavation	4,800,000 m^3 @ \$10/m^3	\$48,000,000
Revegetation	278.4 ha @ \$5,500/ha	\$1,531,000
Bridge (PR 60)	\$3 M (approaches) + 2,400 m^2 @ \$5,200/m^2	\$16,800,000
Maximum Pump Lift Required	4.8 m (15.7 ft)	
Pump Power	9.3 MW	
Pump Sta. Cost	\$4,000 per Kw	\$37,000,000
Diesel Power Station	\$1,500 per Kw	\$14,000,000
Peripheral Facilities	10% of Total Direct Costs	\$11,739,000
Environmental Mitigation Costs	10% of Total Direct Costs	\$12,913,000
Indirect Costs (Owner's Costs, Engineering, Administration, Approvals)	20% of Total Direct Costs	\$23,478,000
Contingency Allowance	30% of Total Direct and Indirect Cost	\$49,656,000
	Total Cost	**\$215,178,000**

Table 14.3
Summary of Design Parameters and Estimated Capital Costs
for Diversion Capacity of 283 m^3/s (10,000 cfs)

Item	Description	Cost (where pertinent)
Channel Invert El at Lake Winnipegosis	El 251.4 m (El 824.8 m)	
Channel Invert El at Cedar Lake	El 250 m (El 820.2 m)	
Channel Base Width	10 m (32.8 ft)	
Channel Side Slopes	1V:5H	
Design Depth of Flow (open water)	6.0 m (19.7 ft)	
Bed Slope	1 m per 2415 m (1 ft per 7,920 ft)	
Maximum Velocity	1.2 m/s (3.9 ft/s)	
Manning's n-value	0.03	
Volume of Excavation	6,330,000 m^3 @ \$10/m^3	\$63,000,000
Revegetation	278.4 ha @ \$5,500/ha	\$ 1,531,000
Bridge (PR 60)	\$3 M (approaches) + 2,400 m^2 @ \$5,200/m^2	\$16,860,000
Maximum Pump Lift Required	4.4 m (14.4 ft)	
Pump Power	17.4 MW	
Pump Sta. Cost	\$4,000 per Kw	\$70,000,000
Diesel Power Station	\$1,500 per Kw	\$26,000,000
Peripheral Facilities	10% of Total Direct Costs	\$17,739,000
Environmental Mitigation Costs	10% of Total Direct Costs	\$19,513,000
Indirect Costs (Owner's Costs, Engineering, Administration, Approvals)	20% of Total Direct Costs	\$35,478,000
Contingency Allowance	30% of Total Direct and Indirect Costs	\$75,036,000
	Total Cost	**\$325,158,000**

A substantial component of the costs in the tables above is for the pumping facilities necessary to make these schemes effective. As noted above, the water level of Lake Winnipegosis, even during floods, is almost always considerably lower than Cedar Lake. It is not possible to rely on gravity alone to drive water from Lake Winnipegosis to Cedar Lake. The schemes summarized above all incorporate a pump facility to lift the water to a height at the upstream end of the channel from which it would flow some 5.8 km (3.6 mi) by gravity to the downstream end and release into Cedar Lake. The costs of the pump facilities are approximate only. They are based on a power requirement that is proportional to the lift (i.e., vertical rise of the water to allow it to flow through the channel to Cedar lake) and to the design discharge capacity (for example

283 m^3/s or 10,000 cfs in the largest option considered). That is converted to a power in kilowatts (9.8 x lift (m) x discharge * efficiency), as listed in Tables 14.1 to 14.3. It was noted in Tables 14.1 to 14.3 that the lift varies modestly from 4.4 m (14.4 ft) for the largest channel to 5 m (16.4 ft) for the smallest channel. This variance is associated with the lift required to drive the water by gravity through the channel selected for that particular scheme. For all the channels, a maximum allowable design velocity of flow in the channel was set at 1.2 m/s (4 ft/s). This was considered a reasonable estimate to prevent erosion in the clay soils through which the channel would be excavated. Lower maximum velocities could be selected, and that would permit a lower lift to be selected for the pump station, thereby reducing its costs. However, the much larger excavation required to provide the lower velocity would cost much more and result in a significantly larger overall cost of the scheme. That same principle was found to govern all three of the options. It was concluded in this preliminary optimization that the best strategy, for the lowest cost, would be to select the combination of pump capacity and channel size that would maximize the velocity in the channel. Similarly, the channel cross section was selected to be as deep and narrow as practical, so as to minimize excavation costs.

The costs of large pump stations of this type are difficult to predict; but engineering judgement, based on experience with pump stations and comparably sized hydroelectric stations, would indicate that a unit cost of at least $4,000 per kW would be representative. That unit price has been adopted for the purposes of cost estimation in this screening study. It is expected that closer examination in more detailed analyses, possibly with the engagement of manufacturers of such equipment, would almost certainly lead to substantially larger estimates of cost.

In addition to the cost of the pump station itself, power is needed to operate it. There is an existing 230 kV transmission line in the area, but the feasibility of use of that line to supply the significant load of the pump station when it needs to be operated would have to be verified by complex system analyses by Manitoba Hydro. If such studies showed that the line could not be relied upon to supply adequate power to the pump station at the same time as other committed loads, then either an upgrade to the transmission line, or alternatively, a stand-alone diesel-driven power station would be needed. The latter has been assumed for the purposes of this study, and a representative unit cost of $1,500 per kW was adopted. It should be noted, however, that even if the total cost of the diesel station were to be discounted (assuming

optimistically that the existing transmission line could supply adequate power to the pump station), the economic feasibility of the option would still not be favourable.

The large costs for these options would be offset to some extent by the value of energy generated from the additional flow transferred from Lake Winnipegosis to the Saskatchewan River. That flow would be available for passage through the turbines at Manitoba Hydro's Grand Rapids Hydroelectric Generating Station. Nevertheless, that value would be small and has been excluded from the cost assessment at this stage of study. The rationale for this exclusion was based on a conceptual analysis of the potential for energy generation over the 47-year period of flow record from 1965 to 2011. A number of assumptions were adopted for this analysis. Those that required approximation were intentionally selected to provide a conservatively high estimate of the value of the increment in energy at the Grand Rapids plant. They included:

- Mean monthly flows were used for the calculations. This overstates the potential energy benefits because there may, in reality, be some days during the month where the diverted flow cannot be used due to short term spillage at Grand Rapids. That day to day refinement is not captured in the analysis.
- Flows are as reported by Water Survey of Canada for the Saskatchewan River at The Pas.
- Maximum turbine flow at the Grand Rapids Hydroelectric Station was taken from a Manitoba Hydro website and is 1,500 m^3/s (53,000 cfs). (This was revealed in the analysis to be a restriction for use of extra water from the diversion in only one month of the historical record of 47 years (it occurred in 1974).
- Net head at Grand Rapids is 36.6 m (120 ft).
- Overall efficiency of the Grand Rapids turbine / generating units is 87%.
- Average head for pumping from Lake Winnipegosis is 3 m (9.8 ft). This factor is required because the extra water delivered to Grand Rapids must first be provided through input of energy at the pump station. This energy has been deducted from the gain in energy that would be possible at the Grand Rapids plant. This average head of pumping is an approximation that is estimated to be +/-30% in accuracy. But such a large range of accuracy has only a very small influence on the net results.
- The overall efficiency of the pumps at the point of drawing water from Lake Winnipegosis is 70%.
- The value of energy, both for extra generation at Grand Rapids, as well as for the supply of power to the pump station on Lake Winnipegosis, is assumed to be $75/Mwh. More precision can be provided by Manitoba Hydro, but for the purposes of this study, this price is considered adequate.

The calculations of the value of diverted flow to the Grand Rapids plant are demonstrated in Table 14.4. It shows the monthly calculations for the year 1970. That year was selected because it was the first one in the record after 1965 that diversion of flow from Lake Winnipegosis would have been required. The columns in Table 14.4 are explained below:

- Column 1 – Month of the year.
- Column 2 – Recorded mean monthly outflow at Grand Rapids Generating Station in cfs.
- Column 3 – Computed mean monthly diversion of flow from Lake Winnipegosis to Cedar Lake (Grand Rapids Reservoir) in cfs.
- Column 4 – Computed mean monthly diversion of flow from Lake Winnipegosis to Cedar Lake (Grand Rapids Reservoir) in m^3/s.
- Column 5 – Maximum amount of flow that could be passed through the turbines without causing spillage (m^3/s).
- Column 6 – Actual amount of additional flow (from Lake Winnipegosis) that is assumed to pass through the turbines (in m^3/s).
- Column 7 – Additional energy in megawatt-hours generated by the flow diverted from Lake Winnipegosis (9.8 x col 6 x 36.6 m x 87% x 30.4 days x 24 hours /1000).
- Column 8 – Value of the energy generated (Col 7) at a unit price of $75/Mwh.
- Column 9 – Estimate of the energy (Mwh) required to lift the water at the pump station when diversion from Lake Winnipegosis is required.
- Column 10 – Cost of energy to lift the water at the pump station, at a price of $75/Mwh.

Table 14.4
Example Calculations of Net Value of Energy Gained from Water Diverted from Lake Winnipegosis
(Example Year – 1970 for the largest diversion scheme)

1	2	3	4	5	6	7	8	9	10
01--1970	1030	0.0	0.0	470	0.0	0.0	$0	0.0	$0
02--1970	1010	0.0	0.0	490	0.0	0.0	$0	0.0	$0
03--1970	858	0.0	0.0	642	0.0	0.0	$0	0.0	$0
04--1970	369	0.0	0.0	1131	0.0	0.0	$0	0.0	$0
05--1970	425	0.0	0.0	1075	0.0	0.0	$0	0.0	$0
06--1970	481	0.0	0.0	1019	0.0	0.0	$0	0.0	$0
07--1970	548	0.0	0.0	952	0.0	0.0	$0	0.0	$0
08--1970	583	0.0	0.0	917	0.0	0.0	$0	0.0	$0
09--1970	537	0.0	0.0	963	0.0	0.0	$0	0.0	$0
10--1970	302	0.0	0.0	1198	0.0	0.0	$0	0.0	$0
11--1970	558	333.3	9.4	942	9.4	2156.1	$161,708	290.2	$21,765
12--1970	856	645.2	18.3	644	18.3	4173.1	$312,983	561.7	$42,125
Totals							**$410,800**		**$63,890**

For 1970, the calculations show that the total value of the energy for that year would be approximately $410,800. This optimistically assumes that all of the water can be used to generate energy that can be used in the Manitoba Hydro system. The table also shows that the cost of the energy required to lift the water at the pump station would be approximately $63,900. The net gain in energy value is $346,900 for the 12-month period of 1970.

Energy values for other years from 1965 through to 2011 have also been estimated. The historical records beyond 2011 were not available from Water Survey of Canada at the time this analysis was carried out. Due to the high water levels on Lake Winnipegosis in 2014, diversion of water would have also occurred in 2014, likely resulting in some additional energy value for that year. However, the added value would only have minor impacts to the overall average annual value calculated from the results and therefore does not change the conclusions for this option. A summary of the findings for each of the 47 years is tabulated in Table 14.5.

Table 14.5
Summary of Computed Energy Values for Water Diverted to the Saskatchewan River at Cedar Lake (for Largest Diversion Scheme)

Year	Energy Value	Year	Energy Value	Year	Energy Value	Year	Energy Value	Year	Energy Value
1965	0	1975	0	1985	0	1995	0	2005	$1,259,500
1966	0	1976	0	1986	0	1996	$1,119,500	2006	$10,068,800
1967	0	1977	0	1987	0	1997	$699,700	2007	$7,600,500
1968	0	1978	0	1988	0	1998	$420,800	2008	0
1969	0	1979	0	1989	0	1999	0	2009	0
1970	$410,800	1980	0	1990	0	2000	0	2010	$11,850,000
1971	$541,700	1981	0	1991	0	2001	0	2011	$18,494,200
1972	0	1982	0	1992	0	2002	0		
1973	0	1983	0	1993	0	2003	0		
1974	$332,870	1984	0	1994	0	2004	0		

The results in Table 14.5 show that for the largest diversion scheme (283 m^3/s or 10,000 cfs), the average annual value of the energy generation would be approximately $1,120,000. More precision in a more detailed study would almost certainly reduce that estimated value. Furthermore, more than 57% of that average is comprised of the generation provided in the 2010-2011 flood seasons. The exceptional floods in those two years are considered to distort the real average of the potential increment in energy that would be available in the long-term.

The total construction cost of the scheme (see Table 14.3) is estimated to be approximately $325 million. Amortization of that cost with an interest rate of 7% per year over a 50-year life, in addition to estimated annual operation and maintenance costs of $3.25 million/year would result in an average annual cost of approximately $26.8 million for this option. The optimistically high average annual value of the energy available from diverted water is only $1.1 million and represents only 4% of the average annual costs. Nonetheless, this value was considered in the economic analysis of the option discussed in Section 22.0 of this report. This same principle would also apply to the other options with lesser discharge capacities. As shown in Section 22.0, the value of diverted water to the Grand Rapids Hydroelectric Station does not form a significant factor in the economic viability of the schemes.

Table 14.5 shows that there could be as many as 20 years between diversions of floodwaters from Lake Winnipegosis. Factors associated with the remoteness of this site and the infrequent deployment of the pump facilities are important:

- This is an isolated site and the combination of this factor with infrequent operation would create practical problems with supplies of replacement parts when needed, staffing by qualified operators when needed, and housing facilities for operation staff. Operation costs would be expected to be relatively high.
- Years of inactivity will have the potential to divert valuable maintenance funds to other important needs within government budgets, with a net result being the potential for unreliable operation when needed.
- Breakdown of the facilities, even if used frequently, would be possible, and could jeopardize the effectiveness of the scheme when it is needed the most. Infrequent use would add to the annual probability of such outages.

14.3 FLOOD DIVERSION TO LAKE WINNIPEG

Another option to mitigate floods is a diversion channel from Lake Winnipegosis to Lake Winnipeg. This channel would convey excess flood waters during severe events.

The concept is driven primarily by the obvious advantage of “short-circuiting” floodwaters from the Lake Winnipegosis drainage basin directly to Lake Winnipeg, without the need to pass through Lake Manitoba or Lake St. Martin and the Dauphin River.

While simple and attractive in concept, the land separating Lake Winnipegosis from Lake Winnipeg varies from 60 to 80 km (37 to 50 mi) in width, and rises more than 10 m (33 ft) above the normal water levels of Lake Winnipegosis. In addition, the water level on Lake Winnipeg is some 37 m (121.4 ft) lower than that of Lake Winnipegosis. From the point of high land between the lakes, the gradient down to Lake Winnipeg is extremely steep (more than 2 m/km (10.6 ft/mi)). This demands the inclusion of energy dissipation facilities along the lower reaches of the diversion route to prevent severe erosion in the diversion channel.

14.3.1 Location and Key Factors

Drawing 14.2 shows on a map the northern portion of the land that separates Lake Winnipegosis from Lake Winnipeg.

From 1961 to 2012, the range of static water levels on Lake Winnipegosis (wind effects eliminated) was from El 252.4 m (El 828 ft) up to El 254.4 m (El 834.65 ft). In 2011, the static lake level peaked at El 254.4 m (El 834.65 ft).

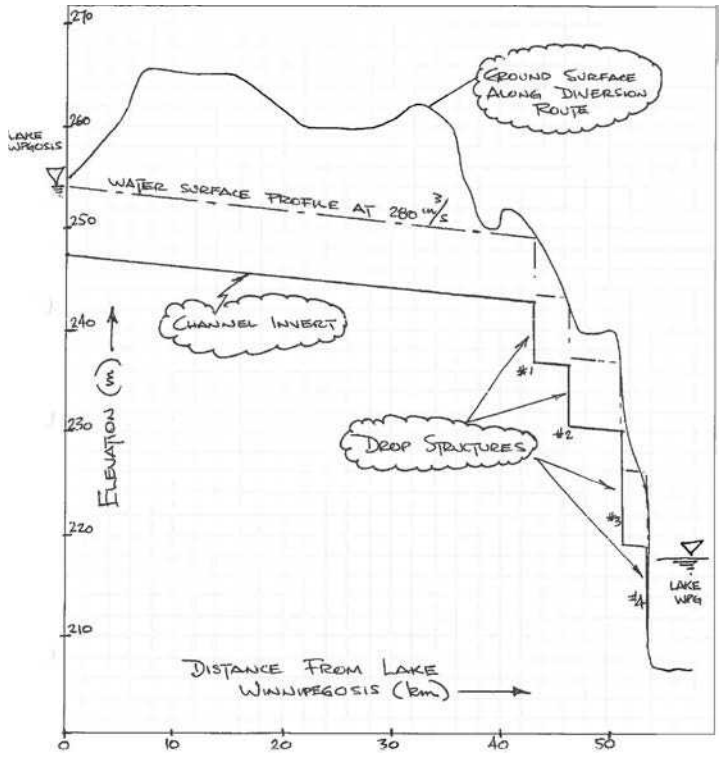
On the other hand, Lake Winnipeg levels typically range between El 216.7 (711 ft) and 217.9 m (715 ft). It is clear that a channel through the land between the lakes could permit the flow of water by gravity.

The narrowest and lowest portion of the land between the lakes is at the northeast end of Lake Winnipegosis. The soil type in this area is unknown, but it has been assumed for the purposes of this study to be similar to the ground along the northern shore of Lake Winnipegosis. That zone has been proven by explorations by the Manitoba Government to consist primarily of clay with some isolated boulders, with bedrock at significant depth (well below the invert of any potential diversion channel). The route for a channel has been selected to achieve the following criteria:

- The length of the channel should be minimized, to reduce environmental impacts and to keep the construction costs as low as possible.
- Existing lakes should be avoided wherever practical and possible, to avoid draining existing water bodies and causing significant environmental impacts. For example, diversion through Kanusk, Kaweenakumik and Kutiwik lakes would be feasible, and would reduce the volume of excavation compared to a fully land-based diversion route. However, diversion through the lakes could cause those lakes to drain substantially, and potentially to disappear. These lakes now comprise an existing water surface area of over 200 km^2 (77 mi^2). Draining such large lakes was considered unacceptable.

Drawing 14.2 also shows the tentatively selected route for a diversion channel for the purposes of this screening study. Figure 14.11 shows the ground contours along the selected diversion route, as well as the profile of the preferred option for the channel design.

Figure 14.11
Profile of Diversion Channel



14.3.2 General Arrangement of Diversion Concept

Diversion to Lake Winnipeg to relieve flood conditions on Lake Winnipegosis would require a channel approximately 55 km (34 mi) in length, with at least three relatively large drop structures to dissipate excess energy and prevent channel erosion.

Since the channel is expected to be excavated through clay (with some boulders) a channel cross section that is trapezoidal with side slopes of 1V: 5H was selected for planning and budgetary purposes. It is possible that further analysis with soil information from the area would reveal that this is too steep for long-term stability. However, at this point, given the tenuous economics of this scheme, an optimistic design was selected. It was recognized that if the economics continued to be unfavourable, introduction of flatter side slopes in the channel would only accentuate that outcome. Similarly, for the purposes of this screening level of studies, it was assumed that the diversion channel would be designed for open water conditions only. Design for operation in winter would almost certainly add cost to the scheme; and it was

intended that if this flood mitigation alternative showed merit, a more detailed assessment of the effects of ice would be needed in the next level of study.

Given the tenuous economic nature of this mitigation scheme, only one option was considered for cost estimation and evaluation in the next phase of economic assessment. It would have a potential maximum diversion capacity of 283 m^3/s (10,000 cfs). It was conceivable that additional capacity could show somewhat more favourable results, but it was expected a channel could convey 283 m^3/s (10,000 cfs), which would be sufficiently indicative of the concept to determine whether further analysis of other options would be warranted.

Figure 14.11 shows a profile of the concept devised for the case with a discharge capacity of 283 m^3/s (10,000 cfs). Details and estimated construction costs of the scheme have been summarized in Table 14.6. Detailed layout of the drop structures was not attempted, other than to roughly estimate the footprint area of a typical structure for purposes of establishing a basis for cost estimation. It was realized that the cost of the drop structures, regardless of their number and size, would be less than 10% of the overall cost of the scheme, so detailed attention to the design of those structures would not lead to significant improvement in the estimation of the cost of the overall scheme. For that reason, the budgetary estimates for the drop structures were considered representative and acceptable for the purposes of this screening study. Similarly, there would likely be a need for a control structure at Lake Winnipegosis, or at least an uncontrolled overflow weir, so that the diversion channel would only divert water when the water level of Lake Winnipegosis exceeded a specified threshold. Costs for an overflow weir are considered relatively minor compared to the overall cost of this scheme and are not included at this conceptual level of study.

Table 14.6
Summary of Design Parameters and Estimated Capital Costs for Diversion Capacity of 283 m^3/s

Item	Description	Cost (where pertinent)
Channel Invert El at Lake Winnipegosis	El 247.2 m (El 811.0 ft)	
Channel Invert El at Lake Winnipeg	El 210 m (El 689.0 ft)	
Channel Base Width	15 m (49.2 ft)	
Channel Side Slopes	1V:5H	
Bed slope	1 m per 10 km	
Design Depth of Flow (open water)	6.85 m (22.5 ft)	
Estimated Manning n-value of channel bed	0.03	
Maximum velocity	0.84 m/s (2.8 ft/s)	
Volume of Excavation	71,500,000 m^3 @ \$10/m^3	\$ 715,000,000
Drop Structure #1	6 m drop; 900 m^2 @ \$10,000/m^2	\$ 9,000,000
Drop Structure #2	6 m drop; 900 m^2@ \$10,000/m^2	\$ 9,000,000
Drop Structure #3	11 m drop; 1500 m^2 @ \$10,000/m^2	\$ 15,000,000
Drop Structure #4	8 m drop; 1200 m^2 @ \$10,000/m^2	\$ 12,000,000
Bridge at PTH 6	135 m long, deck area 1600 m^2 @ \$7,000/m^2 + \$3 M (approaches)	\$ 14,200,000
Revegetation	2,750 ha @ \$5,500/ha	\$ 15,125,000
Environmental Mitigation Costs	10% of direct costs	\$ 78,933,000
Indirect Costs (Owners Costs, Engineering, Administration, Approvals)	20% of direct costs	\$ 157,865,000
Contingency Allowance	30% of total direct and indirect costs	\$ 307,837,000
	Total Cost	**\$1,333,960,000**

Various base widths and depths of channel were investigated to determine whether a deep narrow channel could be significantly more economical than a wide shallow one. The selected design had a base width of 15 m (49.2 ft) and was considered the minimum that could feasibly be constructed using large scale efficient excavating equipment. This would require a design depth of flow of approximately 7 m (23 ft), assuming a channel slope of 1 m per 10 km (0.5 ft per 1 mi), and a design velocity (average for the cross section) of 0.84 m/s (2.75 ft/s). This channel was computed to require 71,500,000 m^3 (93,000,000 yd^3) of excavation.

Two other options were considered, but rejected, as follows:

- A 50 m (164 ft) wide base width with the same slope of 1 m per 10 km (0.5 ft per 1 mi). This would reduce the design velocity to approximately 0.8 m/s (2.6 ft/s) and the design depth of flow to approximately 4.9 m (16.1 ft). The estimated volume of excavation for this arrangement would be approximately 74,000,000 m^3 (96,000,000 yd^3), or about 6% more than the preferred alternative described above.
- A 15 m (49 ft) base width with a channel bed slope of 1 m per 7 km (0.8 ft per 1 mi). This would increase the velocity to approximately 0.95 m/s (3.1 ft/s) and reach the maximum that could be considered reasonably safe with respect to potential channel bed erosion. In spite of the smaller cross sectional area of the channel below the water line, the increased depth due to the steeper slope resulted in a volume of excavation of 73,700,000 m^3 (95,800,000 yd^3).

The intent of this diversion scheme would be to reduce high water levels on Lake Winnipegosis without substantially changing the lower end of the normal range of fluctuations of levels of that lake. The primary focus of the concept was to consider operation of the diversion only when the level on Lake Winnipegosis reached a clearly abnormally high level. The initial analyses were based on an assumed activation of the diversion when the water level of Lake Winnipegosis would reach the upper 90 percentile thresholds approximately El 253.4 m (El 831.5 ft) in winter and El 253.7 m (El 832.5 ft) in summer, and return the lake to, or below, those thresholds as quickly as possible. More aggressive schemes were also investigated, assuming an activation level on Lake Winnipegosis as low as El 253.1 m (El 830.5 ft) in winter (currently exceeded about 50% of the time), and El 253.4 m (El 831.5 ft) in summer (currently exceeded about 30% of the time).

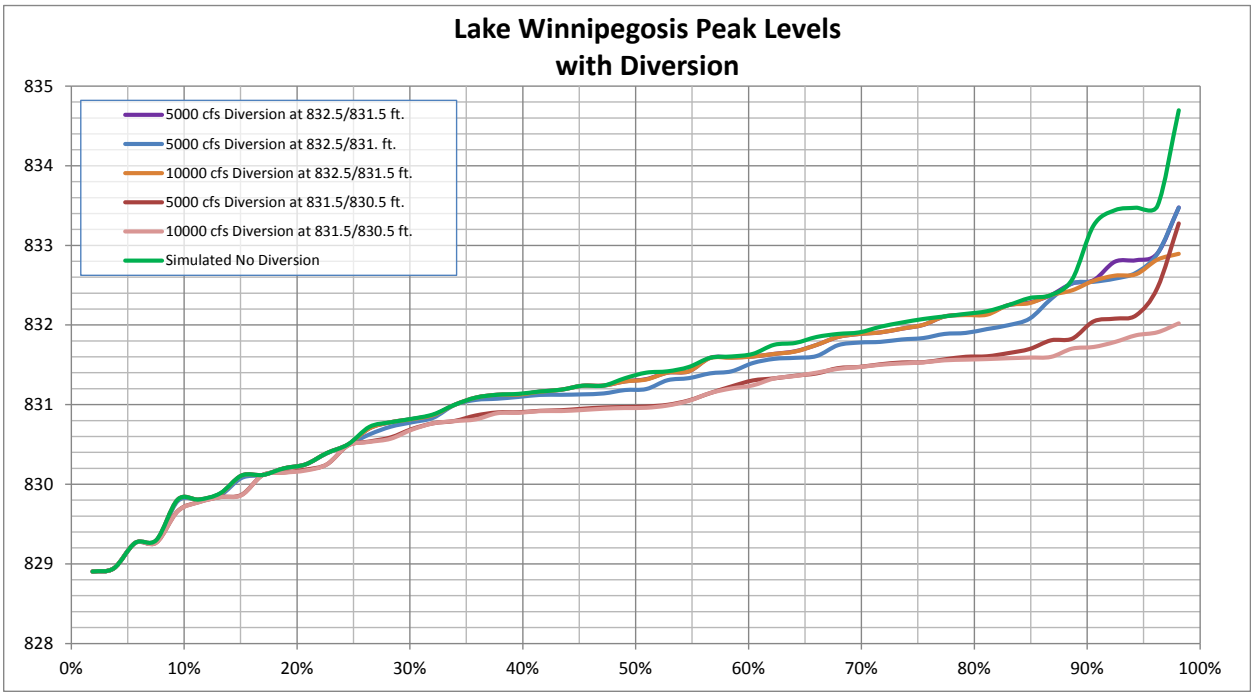
14.3.3 Hydrological Studies

As discussed above, a range of options was investigated from the hydrologic perspective to understand what the frequency of use of this diversion scheme would be and what potential reduction of lake levels would result in Lake Manitoba. Details of the hydrologic model are documented in Section 8.2.

Figures 14.12, 14.13 and 14.14 demonstrate the results of the hydrological studies in terms of effects on water levels on Lake Winnipegosis, Lake Manitoba and Lake St. Martin, respectively. These duration curves are based on hydrological records from 1961 to 2012, inclusive. They demonstrate what the water levels on the three lakes were historically, and what they would

have been if each of the diversion concepts had been installed for that period of record. It should be emphasized that, although two diversion capacities were analyzed from the hydrologic perspective, only the case with a capacity of 283 m^3/s (10,000 cfs) is addressed in Section 14.3.4 in terms of cost and development of design components.

Figure 14.12
Lake Winnipegosis Peak Levels with Diversion



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.13
Lake Manitoba Peak Levels with Diversion

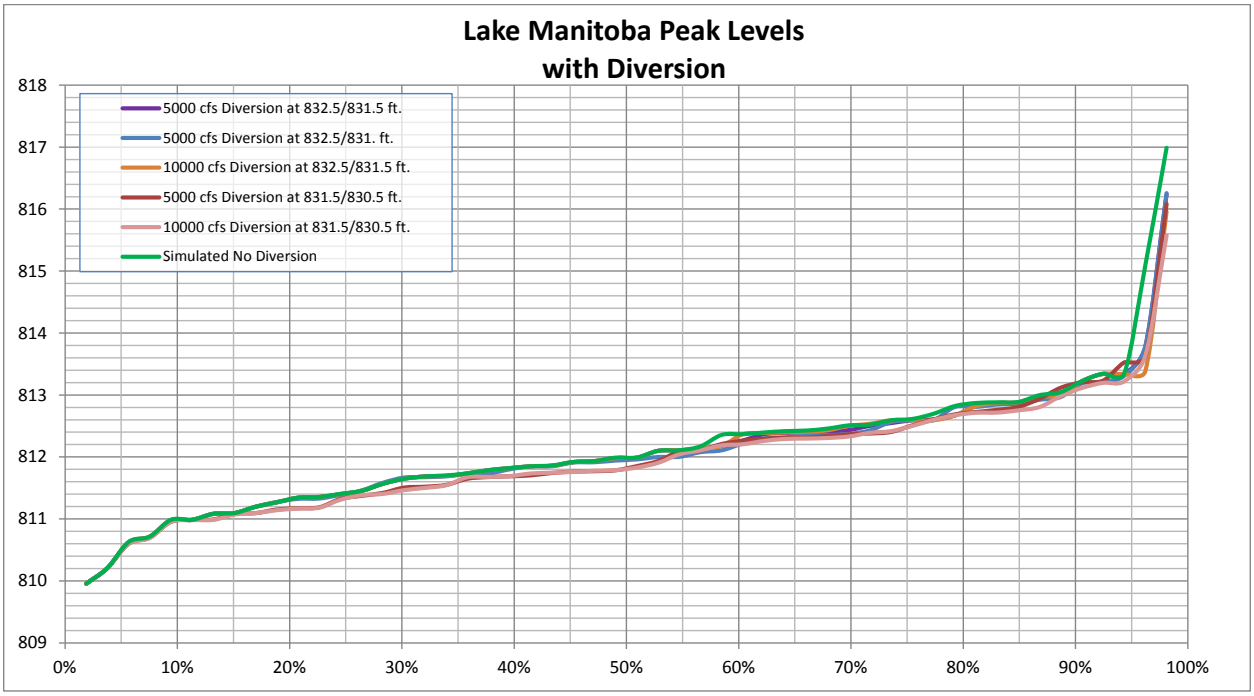
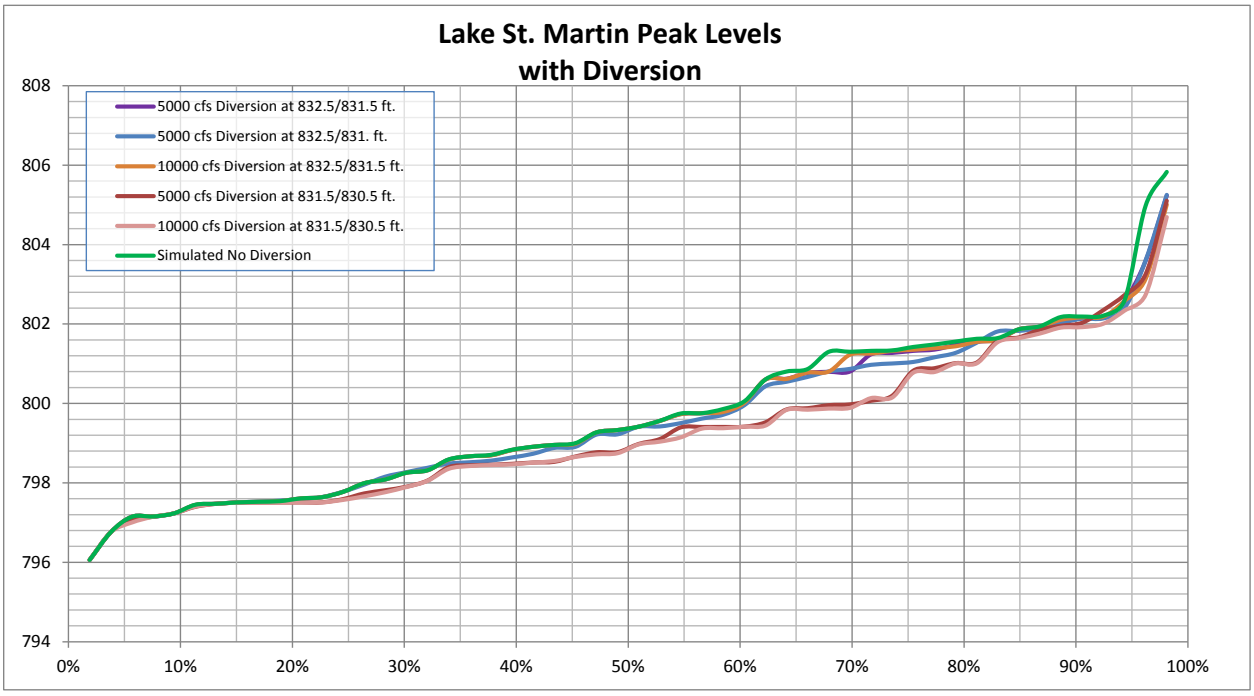


Figure 14.14
Lake St. Martin Peak Levels with Diversion



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

To put the performance of this scheme into perspective, the greatest reduction in water level of Lake Manitoba was shown to be in 2011. The diversion scheme with relatively aggressive activation levels on Lake Winnipegosis of El 253.4 m (El 831.5 ft) in the summer and El 253.1 m (El 830.5 ft) in the winter was estimated to cause a reduction by approximately 0.4 m (1.4 ft) on Lake Manitoba.

14.3.4 Cost Estimation

Cost estimation was only crudely carried out at this screening stage of analysis. Capital costs and other pertinent information have been summarized in Table 14.6. An allowance of cost per drop structure was included, based on the expectation of a cost of roughly $9,000 per square metre. If the scheme were to appear advantageous, it was possible that optimization of the design of the channel viz-a-viz the drop structures (e.g., more channel excavation, fewer, higher drop structures) could lead to some improvement in cost. However, it was clear that this would result in only minor changes in total cost and would be unlikely to affect the overall economic indicators for this scheme. It should be noted that the capital costs of this mitigation scheme were not affected by the operational assumptions of the trigger level on Lake Winnipegosis for initiation of diversion, as shown in Figures 14.12 to 14.14.

The cost estimate also included an allowance for environmental mitigation works that would be required to construct this scheme and address potential adverse environmental impacts. Due to the potential impacts of changes to the natural Lake Winnipegosis water level regime, and the presence of numerous stakeholders including First Nations communities around both lakes, environmental mitigation costs were assumed to be moderate.

The costs were significant. By far the single largest single cost component is for the excavation of the channel. That is driven by the length of the channel (55 km) (34 mi), as well as the depth of the excavation in the highest part of the land between the lakes (approximately 18 m (59 ft)). This raises the obvious question of whether pumping might be an option to reduce the excavation costs. It may have been considered in other circumstances. But if pumping were to be considered, it would be far more effective to apply it to the shorter channel that would divert water with pumping to Cedar Lake. No further consideration of pumping for the diversion to Lake Winnipeg was justified.

14.4 CONTROL STRUCTURE AT OUTLET OF LAKE WINNIPEGOSIS

A control structure at the outlet of Lake Winnipegosis could be used under conditions of excessive runoff to store flood waters in Lake Winnipegosis and reduce inflow to Lake Manitoba.

The concept was driven primarily by the existence of a large potential storage capacity that could be used to store excessive inflows to Lake Manitoba. It was recognized that there would be adverse conditions imposed under flood conditions on Lake Winnipegosis.

14.4.1 Location and Key Factors

Lake Winnipegosis is the second largest lake in Manitoba, the 11th largest lake in Canada, and the 27th largest lake in the world. It has a surface area of approximately 5,400 km^2 (2,085 mi^2) and a shoreline length of more than 600 km (373 ft). Lake Winnipegosis drains into Lake Manitoba via the Waterhen River. By comparison, the surface area of Lake Manitoba (third largest lake in Manitoba) is only approximately 4,625 km^2 (1,786 mi^2).

In addition, the total watershed area draining into Lake Winnipegosis (including the surface area of the lake) is approximately 55,000 km^2 (21,200 mi^2). This watershed forms approximately 70% of the total drainage area of 80,000 km^2 (30,888 mi²) to Lake Manitoba (again including the surface area of Lake Manitoba). It is obvious that retention of excess flood water in Lake Winnipegosis could have a sizeable effect on flood conditions downstream.

For the purposes of this study, location of a structure to control outflows from Lake Winnipegosis was restricted to being upstream of Waterhen Lake. Location of the structure downstream of Waterhen Lake would adversely affect the communities of Skownan and Waterhen, and significantly raise water levels on Waterhen Lake, and was, therefore, rejected.

Unfortunately, the river upstream of Waterhen Lake is split between two separate channels, the West Waterhen River, which carries the majority of flow, and the Little Waterhen River. Preservation of these waterways would require structures to be located on each waterway (or one control structure with excavation of a channel to join the two natural channels).

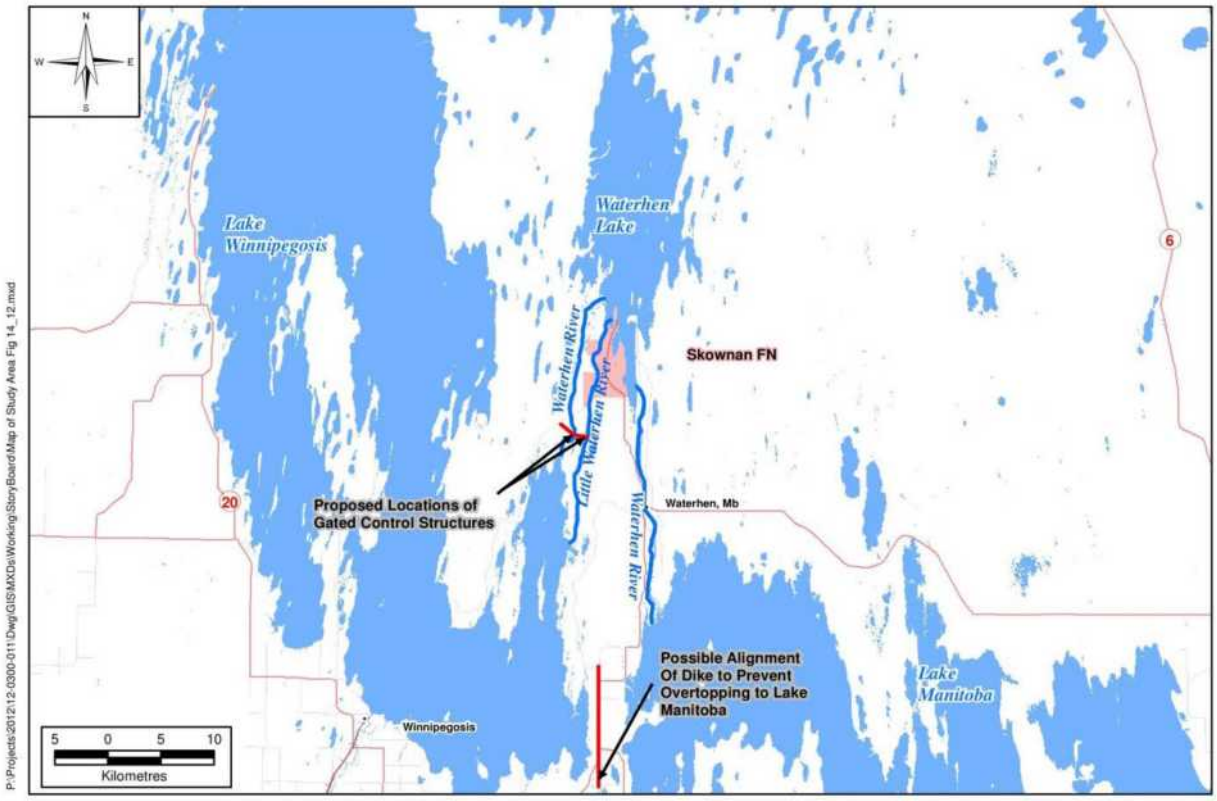
Increases in water level on Lake Winnipegosis would affect its significant shoreline length, as well as shoreline residents and communities. This impact is clearly a significant disadvantage of this scheme that would need to be addressed if the option advanced further.

14.4.2 General Arrangement of Control Structure and Related Works

Control of outflows from Lake Winnipegosis and storage of water above the normal range of the lake levels would require the following facilities:

- Gated control structures on each of the West Waterhen River and the Little Waterhen River. Possible advantageous locations of these structures are shown in Figure 14.15.
- Water retaining dikes that would extend from the structures laterally to connect to high ground levels.
- Channel improvements in the river channels downstream.
- Dikes along a narrow band of land approximately 25 km (15.5 mi) south of the control structures where the natural land retaining Lake Winnipegosis is low. Figure 14.15 also shows the general location of this low area. High water levels on Lake Winnipegosis, particularly during wind storms from the north could threaten to overtop this low area if left unprotected. Formation of a breach and a permanent channel at this location, which would drain down to Lake Manitoba, would have a significant impact on the natural non-flood regime of Lake Winnipegosis. This occurrence was considered unacceptable and must be prevented.

Figure 14.15
Map of Study Area



14.4.3 Design Details

14.4.3.1 Components and Dimensions of the Control Scheme

The structure(s) must be capable of releasing flow into the Waterhen River under normal conditions in such a manner and to the extent that lake levels would not be affected under normal non-flood conditions. The details of achieving this objective are difficult to define precisely because of the lack of bathymetric data for the waterways and the coarse topographic mapping (it has only 5 m (16.4 ft) contour intervals). However, in spite of this, approximations have been attempted, and the concept that would be expected to achieve control of the lake would include structures and facilities as follows:

- A reinforced concrete control structure with a floor slab invert at approximately El 252.1 m (El 827 ft), with eight bays of 5 m clear width, and 7 intermediate piers located in the West Waterhen River as shown in Figure 14.16. Figure 14.17 shows concept sketches of cross sections of the structure. The arrangement has been based

on judgment and experience with detailed design of similar structures elsewhere. No detailed stability analyses have been done. If this scheme advances to more detailed studies, stability analyses of the structure and its foundation could show that some increase in the length of the structure was required. However, the dimensions selected were considered adequate for this level of study.

- Control of outflow would be with steel gates controlled by wire rope hoists suspended from a hoist structure. The size of the control structure was selected so that, in combination with extensive excavation in the river, the outflow stage-discharge relationship for Lake Winnipegosis would remain as it now exists (i.e., no backwater effect under normal conditions). The need and extent for energy dissipation downstream of the gates is unclear at this time, and depends on the nature of the riverbed and the detailed plan of operating the gates. For the purposes of this study, it was optimistically assumed that energy dissipation works (stilling basins for example) would not be required, and that heavy riprap on the channel bed downstream of the structures would be sufficient for erosion protection.

Figure 14.16
Proposed Approximate Location of Control Structures

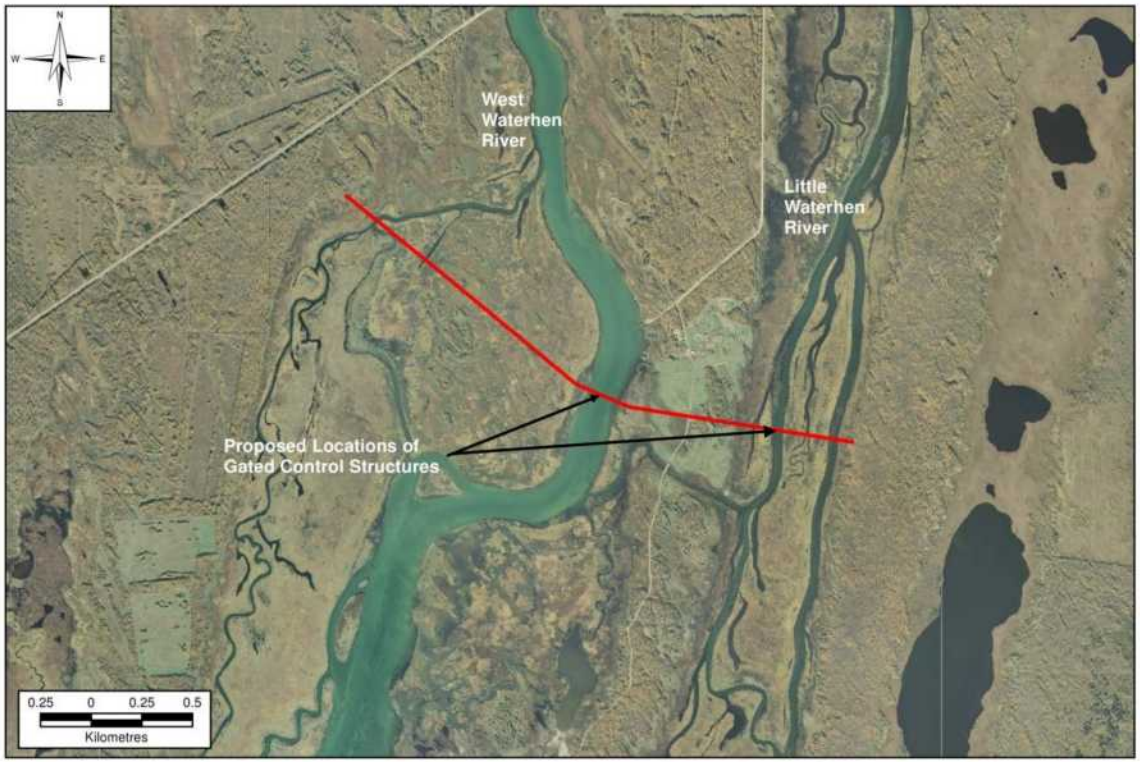
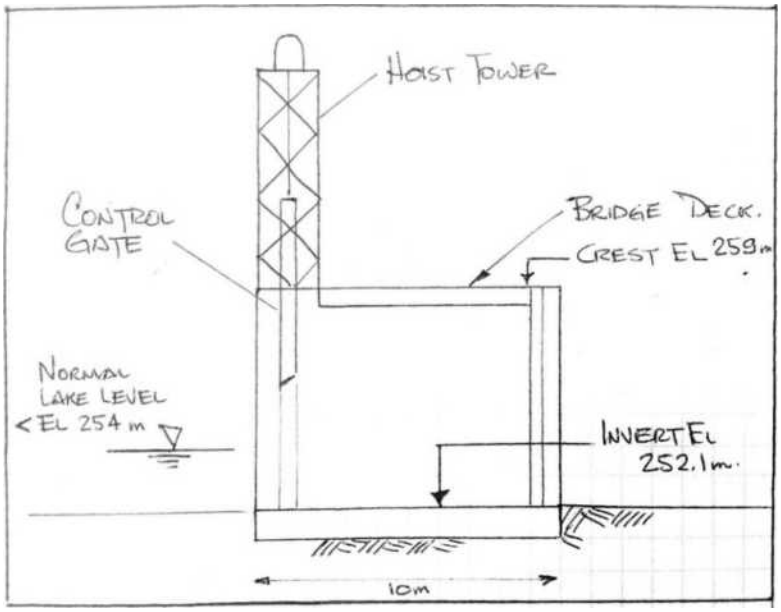


Figure 14.17
Conceptual Cross Section of Control Structure



- A similar control structure located in the Little Waterhen River, but with the same invert level and only three, 5-m wide bays. Both of these concrete structures would remain fully open during most of the year, and in most years. Only during extreme flood events would gates be required to be placed to hold back excess floodwaters from entering Lake Manitoba. This would occur on average roughly about once in 25 to 50 years, depending on the design parameters selected, as described in Section 14.4.3.2.

- The concept has included steel gates in both structures because of the potential for rapidly varying lake levels with strong north or south winds that may start or stop abruptly. If, for example, stoplogs were used to control flows and the sudden appearance of a south wind cause the lake level near the structures to decline, outflow through the structure might decline significantly for many hours, or even cease entirely for periods of time. Similarly, strong north winds could rapidly raise water levels at the structures and the flow over stoplogs could quickly increase many-fold in a matter of hours. Both effects would be environmentally undesirable. As a result of these realities, the concept has included steel gates, rather than stoplogs, to permit rapid response to changing water levels. It is possible that in final design a compromise could be made, and some combination of stoplogs and gates could be chosen. For budgetary purposes, however, gates have been assumed for this study.

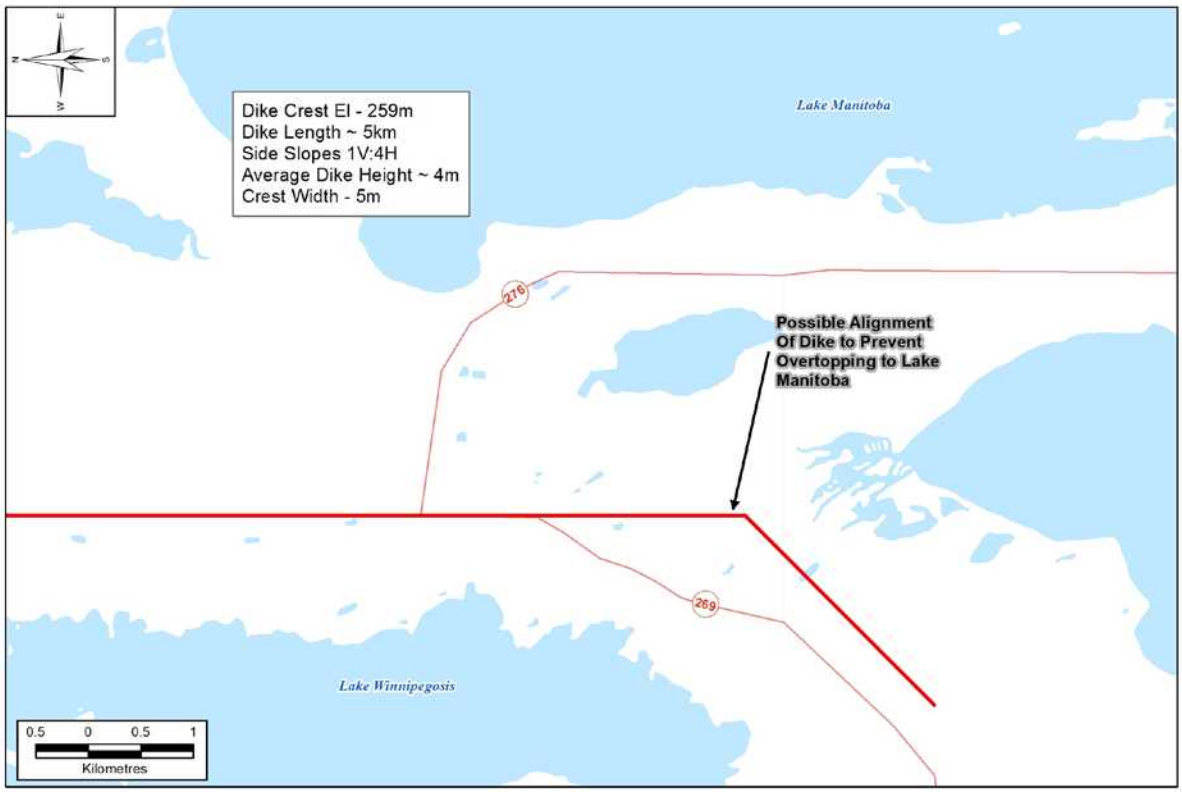
- Channel improvements in the river downstream of the main structure in the West Waterhen River to the extent necessary to compensate the minor loss in discharge capacity of the natural channel that would be imposed by the control structure. That excavation is difficult to define without knowledge of the channel bathymetry and hydraulic performance of the existing channel. However, it is estimated to involve some 5 km (3 mi) of river length with an excavation volume of about 15,000 m^3 (19,600 yd^3). Widening of the channel would be the preferred means of increasing the conveyance of the river. This volume of excavation is based on judgement and cannot be verified

without field surveys that are beyond the scope of this concept study. It should be noted that in final design it may turn out that construction of a larger structure than assumed in this study may eliminate the need for excavation in the river channel. That would need to be investigated when more detailed survey information in the river was available. For the purposes of this study, however, the combination of a structure and channel improvements was assumed.

- Earthfill water retaining dikes that would extend between the structures and to each side of the structures to tie into high ground. These would be dikes with an impervious central core, and with side slopes possibly as steep as 1V:5H, with erosion protection on the upstream face to protect against severe wave attack that would be expected under some conditions on Lake Winnipegosis. Explorations in the area of the structure and soil testing would be required to provide adequate information for a final design. These dikes would have to be high enough to contain a severe inflow design flood without causing overtopping and breaches. Further criteria used for selecting the height of these dikes and of the control structure have been described below.

- Earthfill water retaining dikes along approximately 5 km of low land where overtopping by high waters or by wind driven waves could initiate a breach that could cause a catastrophic release of some of the water stored in Lake Winnipegosis into Lake Manitoba. Figure 14.18 shows the estimated extent of diking superimposed on a drawing produced by the Manitoba Water Resources Branch in 1950 (MWRB, 1955). The dike would have a crest width of approximately 5 m (16.4 ft), and side slopes of 1V:4H. A total volume of approximately 420,000 m^3 (549,400 yd^3) is estimated to be required to provide the required protection in this area. Heavy riprap would be required to protect against wind driven waves, particularly from the northwest.

Figure 14.18
Area of Freeboard Dikes on Southeast Rim of Lake Winnipegosis



The configuration of these structures was contingent on the philosophy of operation of the structures and could change in future more definitive stages of design. For the purposes of this study, however, these structures were considered adequate to permit an approximate cost estimate to be made and to accompany discussions of the concept in broad terms.

14.4.3.2 Hydraulic Design Parameters

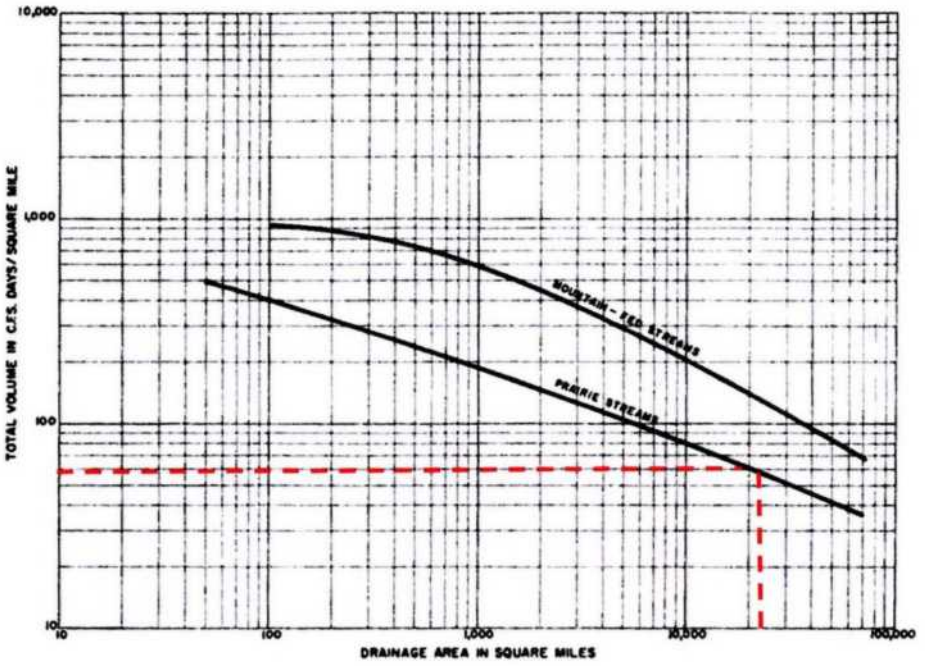
Inflow Design Flood

Retention of water in Lake Winnipegosis by damming at the outlet requires that the retaining structures would be high enough to prevent overtopping and breaching during extreme floods and/or due to overtopping by waves during severe wind storms. Detailed dam safety studies were not justifiable at this conceptual level of study, and have not been done. However, it was clear from rudimentary dam break analysis that a breach of the dikes or at the control structures at the lake outlet could be catastrophic. Sudden damaging floods would occur at the

communities of Skownan and Waterhen where the total population exceeds 1,400 residents. The extent of inundation in these communities due to a dam breach was not clear, but given the low relief in the land surrounding Lake Winnipegosis, it was reasonable to believe that the communities could be devastated by the ensuing flood inundation. In compliance with the Dam Safety Guidelines (CDA, 2009) of the Canadian Dam Association, an appropriate inflow hydrograph for Lake Winnipegosis would be the Probable Maximum Flood (PMF).

Estimation of a PMF normally requires a relatively elaborate engineering study with consideration of antecedent soil moisture conditions, maximum rainfall, snowfall, snowmelt amounts and rates, and runoff. Such a detailed analysis was not considered appropriate for this screening level of conceptual study. An approximate method was applied, based on the results of detailed PMF estimations of similar drainage basins. Such studies were done by the Saskatchewan Nelson Basin Board in 1970 (SNBB, 1972) and resulted in graphs of drainage area vs. flood peaks and flood volumes. The graph shown in Figure 14.19 shows the findings of that review with respect to flood volumes.

Figure 14.19
SNBB Graph of PMF Volume vs Drainage Area
for Prairie-Fed and Mountain-Fed Rivers in Western Canada



Application of the line of best fit to the drainage area of Lake Winnipegosis would suggest a total flood volume of 3,200,000,000 m^3 (2,600,000 ac-ft) for a prairie fed river, and approximately double that for a mountain fed river. Two other comparative values are of interest:

- The PMF estimated for the local watershed that releases into the Nelson River watershed between Jenpeg and Kettle Generating Station near Gillam, Manitoba. That watershed is approximately 85,000 km^2 (32,819 mi^2), compared to 55,000 km^2 (21,236 mi^2) for Lake Winnipegosis. Adjustment of the computed PMF volume for the upper Nelson River to better represent the smaller drainage basin of Lake Winnipegosis would suggest a PMF flood volume of 9,200,000,000 m^3 (7,500,000 ac-ft) for Lake Winnipegosis. However, it is likely that the runoff from the watershed surrounding Lake Winnipegosis would be less intense than on the rocky ground conditions that are prevalent on the Canadian Shield forming most of the catchment of the upper Nelson River.
- The volume of the 2011 Flood inflow to Lake Winnipegosis was an extreme event and calculations of inflows-available-for-outflow (IAO) for the flood between April and August indicated a flood volume of approximately 5,600,000,000 m^3 (4,500,000 ac-ft). That volume includes the effects of precipitation and evaporation over the lake.

The figures presented above are conflicting and make it difficult to confidently choose a reasonable inflow design flood event for the control structures on Lake Winnipegosis. For the purposes of this study, an inflow design flood equivalent to the IAO volume experienced in the extraordinary flood of 2011 has been selected. The worst case scenario of lake levels during the passage of that flood, as described in Section 14.4.4, showed a maximum water level of approximately El 255.6 m (El 838.5 ft). This includes the ponding effect due to the cutback in outflow caused by lowering of the gates. It has been assumed that operation of the structures could be implemented so that even if floods in excess of the 2011 event were to occur, additional release from Lake Winnipegosis could be permitted to prevent excessive water levels in that lake. Furthermore, this maximum assumed water level is a “static level”; and the effects of wind could temporarily increase the effective level at the structures. This issue has been accommodated in the conceptual design of the structures.

Freeboard Requirements

The CDA Dam Safety Guidelines recommend that adequate freeboard be provided so that water retaining structures would not be overtopped by waves or wind setup in either of the two following conditions:

1. A non-flood scenario with the reservoir at a normal level, combined with an extreme wind event with an annual probability of being exceeded of only 1:1000. For Lake Winnipegosis, this is estimated to be a sustained average wind speed for 6 hours of 80 km/h (50 mi/h) from the north.

2. A flood scenario with the reservoir at the peak during the passage of the IDF (in this case at El 255.6 m (El 838.6 ft), as described above), combined with waves and wind setup caused by a coincident occurrence of a wind with a 1 in 2 chance of being exceeded during the year. For Lake Winnipegosis, this is estimated to be a sustained average wind speed for 6 hours of 45 km/h (28 mi/h) from the north.

Lake Winnipegosis is a long narrow water body and is believed to be relatively shallow, on average, similar to Lake Manitoba. Wind setup can be a significant issue and has been estimated using the well-known Zuider Zee equation converted to metric measurements:

S = .000016 * F * U^2 / D

Where:

S = setup above still water level without wind in m.

V = wind speed in km per hour.

F = fetch length in km.

D = average depth of the water body in m.

There is no readily available bathymetric information for Lake Winnipegosis. However, it has been speculated that Lake Winnipegosis is similar in depth to Lake Manitoba, where an average depth of 5 m (~16 ft) has been used routinely for computation of wind setup values on that lake. That depth of 5 m has been adopted for Lake Winnipegosis, as well as a fetch length along the north / south axis of the lake of 240 km (~150 mi). With these assumptions, the wind speed for the non-flood scenario described above would generate a setup of approximately 4.9 m (~16 ft), and for the flood scenario, 1.5 m (~5 ft).

For the purposes of estimating wave heights and potential wave up-rush on the face of the structures, the methodology proposed by the James Bay Energy Corporation (endorsed by the CDA Dam Safety Guidelines) has been applied. Worst case scenarios for wave generation alone would be:

- Structures at the outlet of the lake:
  - For the non-flood condition and a wind from the south at 75 km/h (47 mi/h) the wave up-rush on a riprap sloped face with a 1V:4H side slope would be 2.5 m (8 ft).

- For the flood condition and a wind from the south at 40 km/h (25 mi/h), the wave uprush would be approximately 1 m (3.3 ft).

- Dikes along southeast rim of Lake Winnipegosis:
  - For the non-flood condition and a wind from the northwest at 90 km/h (56 mi/h) the wave up-rush on a riprap sloped face with a 1V:4H side slope would be 3.15 m (10.3 ft).
  - For the flood condition and a wind from the northwest at 52 km/h (32 mi/h), the wave uprush would be 1.5 m (5 ft).

The combination of wind setup and wave uprush must be considered carefully for each of the individual structures involved in this scheme. For example, a strong wind from the north would generate significant wind setup that would affect the structures on the West Waterhen River. However, that same wind should not be used for estimating a simultaneous wave uprush because the north wind would be blowing over and away from those structures. Similarly, the winds from the south described above would indeed generate high waves, but there would be a simultaneous wind set down (opposite to the wind setup described above) in the south end of the lake that would partially, or in some cases fully, negate the wave uprush above the still water level. All these factors have been considered in selecting the appropriate amount of freeboard required for each structure. The governing freeboards have been estimated and the crest elevation selected for each structure. The governing case is the freeboard required for non-flood conditions, which generates a relatively large setup with a wind from the north. For the purposes of this study, the crest elevation of all structures has been selected to be El 259 m (~El 849 ft).

More rigorous calculations of these freeboards would be required in advanced design of these structures; but for this study, the structure heights defined by the crest elevations listed above were considered adequate for a conceptual level of cost estimation.

14.4.4 Hydrological Studies

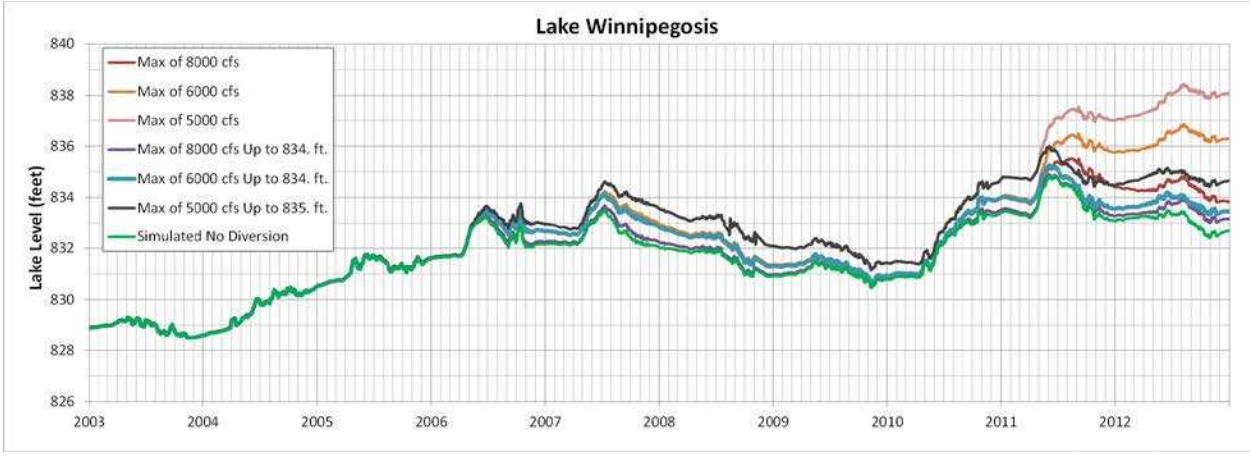
As discussed above, a range of options was investigated from the hydrologic perspective to understand what the frequency of use of this control structure would be and what the potential effects on the three affected lakes (Winnipegosis, Manitoba, St. Martin) would be.

Several control schemes assuming varying degrees of throttling outflow from Lake Winnipegosis were devised. They included:

- Maximum outflow of 227 m³/s (8,000 cfs) from Lake Winnipegosis.
- Maximum outflow of 170 m³/s (6,000 cfs).
- Maximum outflow of 142 m³/s (5,000 cfs).
- Maximum outflow of 227 m³/s (8,000 cfs), but keep Lake Winnipegosis at or below 254.2 m (834.0 ft).
- Maximum outflow of 170 m³/s (6,000 cfs), but keep Lake Winnipegosis at or below 254.2 m (834.0 ft).
- Maximum outflow of 142 m³/s (5,000 cfs), but keep Lake Winnipegosis at or below 254.5 m (835.0 ft).

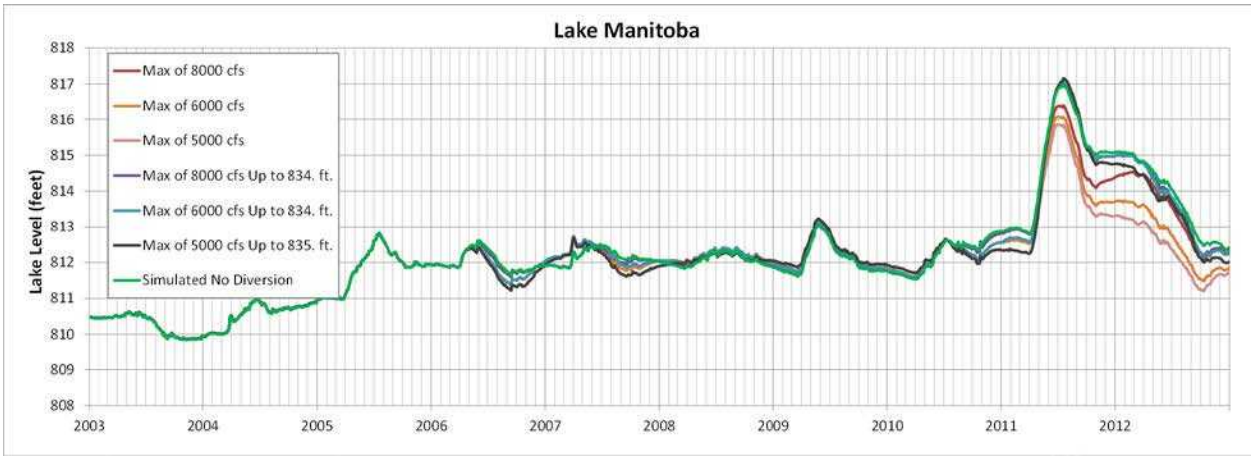
Figures 14.20 to 14.24 demonstrate the results of the hydrological studies in terms of effects on water levels. None of the operating scenarios would have any effect below a Lake Winnipegosis level of El 253.9 m (El 833 ft). In the historical record that dates back to 1961, Lake Winnipegosis has only exceeded this level since 2006. Consequently, the water levels for only the recent few years have been shown on Figures 14.20, 14.21 and 14.22. These graphs as a group demonstrate what the water levels on the three lakes were historically, and what they would have been if each of the options for control structures had been installed during that period. Figure 14.23 summarizes in a bar chart the peak water levels on Lake Manitoba for each year, for each of the schemes analyzed. Figure 14.24 depicts duration curves based on the hydrological records from 1961 to 2012 for Lake Manitoba for the 80% to 100% range. Since the operating scenarios only have effects when the lake levels are high, the duration curves are almost identical for frequencies below 80%.

Figure 14.20
Results of the Hydrological Studies of Effects on Water Levels – Lake Winnipegosis



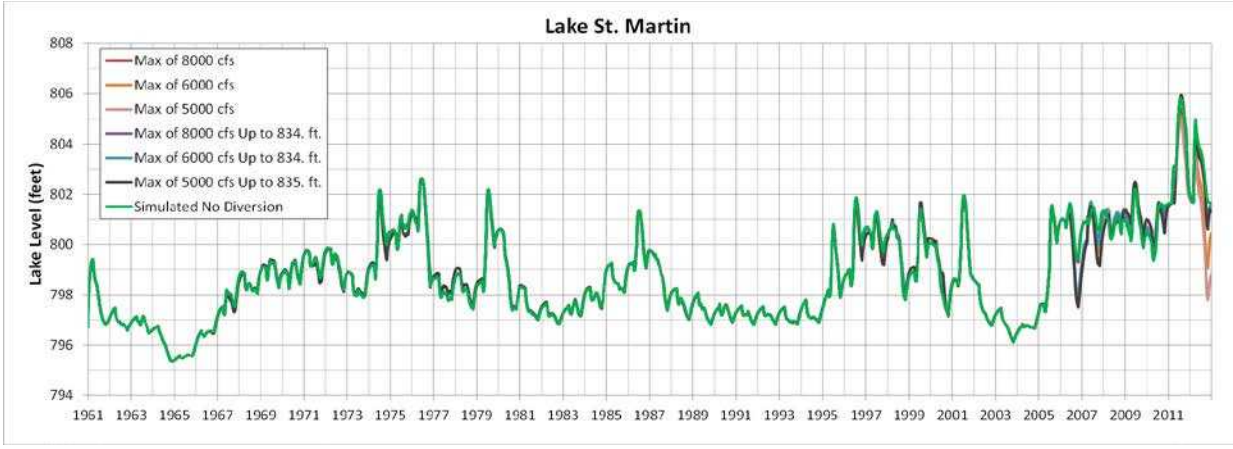
Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.21
Results of the Hydrological Studies of Effects on Water Levels – Lake Manitoba



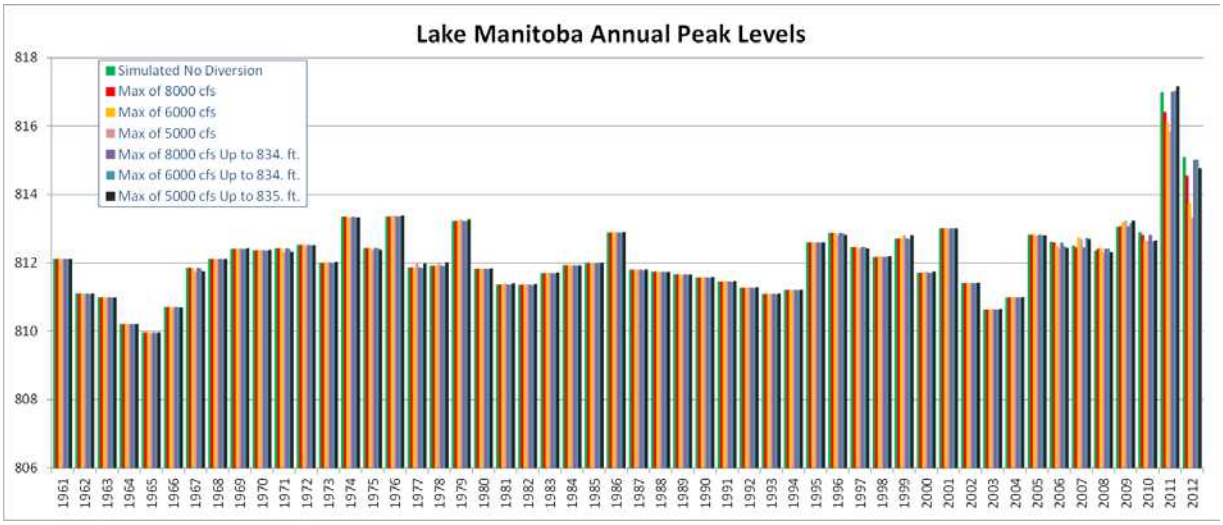
Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.22
Results of the Hydrological Studies of Effects on Water Levels – Lake St. Martin



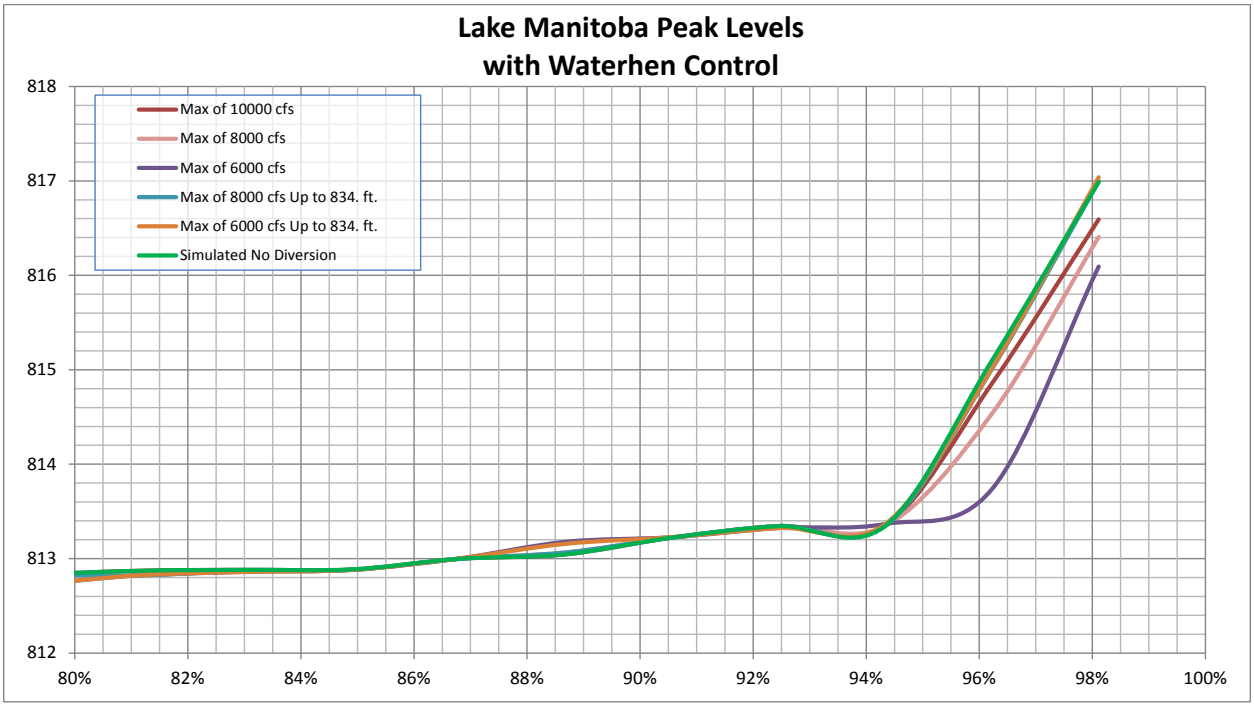
Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.23
Peak Water Levels on Lake Manitoba for Each Year for Each Scheme Analyzed



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

Figure 14.24
Lake Manitoba Peak Water Levels with Waterhen Control



Note: Water levels can be converted to metres by multiplying imperial values by 0.3048.

The key result is that for floods comparable to 2011, the water level of Lake Manitoba could have been controlled to be about 30 to 50 cm (1 to 1.6 ft) lower, if the water level in Lake Winnipegosis were allowed to rise by roughly twice that amount, 60 to 100 cm (2 to 3.3 ft). Lesser effects would of course be possible, with less aggressive cutbacks in outflow from Lake Winnipegosis. Similar effects would also be expected on both lakes for a 2014 type flood, however, to a lesser extent due to the lower inflows and peak water levels on the lakes as compared to the 2011 Flood, as discussed in Section 14.2.3.

The impact of this increase in lake levels during extreme events is a contentious issue, and would require a detailed investigation of the potential effects on the environment, shoreline residents, agricultural and other commercial interests along the lake, as well as the infrastructure in a variety of communities. Impacted communities could include Winnipegosis (population 700), Camperville (population 170), Duck Bay (population 408), Denbeigh Point (population 600), Pelican Rapids (population 80) and possibly Overflowing River depending on backwater effects along the lower reaches of the Overflowing River.

14.4.5 Cost Estimation

Cost estimation was conceptual at this screening stage of analysis. Estimated capital costs and other pertinent information have been summarized in Table 14.7. As discussed in Section 14.4.4, the increased water levels on Lake Winnipegosis could have significant impacts on stakeholders around the lake, including infrastructure in a variety of communities. Community dikes, individual flood protection and/or the purchase of vulnerable properties would likely be required at many locations to mitigate the risk of flooding due to the increased water levels. It was estimated based on judgement that a $10 million cost allowance would be a representative of this potential added expenditure. Detailed investigations beyond the scope of the current study would be necessary to determine whether this allocation was adequate.

The cost estimates also included an allowance for environmental mitigation works that would be required to construct this scheme and address potential adverse environmental impacts. Examples of potential environmental issues and effects associated with the increased water levels on Lake Winnipegosis, as well as the construction of a new dam across the Waterhen River include:

- Loss of terrestrial habitat and agricultural land due to increased inundated area.
- Alteration of aquatic life and habitat near the shorelines of the lake, along tributaries affected by backwater levels and also within the Waterhen River.
- Impeded navigation along the Waterhen River, as well as obstruction to the movement of fish.
- Changes to the quality of water both in Lake Winnipegosis and downstream of the new control structure.
- Increased ground water levels in the vicinity of the lake.
- Increased water levels and flooding around the lake.

These potential impacts could cause substantial resistance to the scheme from numerous stakeholders including First Nations communities residing around Lake Winnipegosis and the Waterhen River, or frequent users of the lake and river. On this basis, environmental mitigation costs were assumed to be major for this scheme.

It should be noted that the high uncertainties in quantities of material for dikes and in the height of the structures in general has suggested the need for a special allowance that was more than has been assumed for other more clearly defined options. A miscellaneous quantity allowance of 10% of the direct and indirect costs was selected for the purposes of this study to reflect the extraordinarily high level of uncertainty.

Table 14.7
Summary of Design Parameters and Estimated Capital Costs for a Control Structure

Item	Description	Costs
Control Structure in West Waterhen River	Invert El 252.1 m, 8 5-m wide bays, intermediate piers, with bridge deck and control gates– footprint area 500 m^2 @$10,000/$m^2$	$5,000,000
Control Structure in Little Waterhen River	Invert El 252.1 m, 3 5-m wide bays, intermediate piers, with bridge deck and control gates – footprint area 190 m^2 @ $10,000/$m^2$	$1,900,000
Channel excavation (improvements) in West Waterhen River	150,000 m^3 @ $10/$m^3$	$1,500,000
Additional cost of gates	11 steel gates, at a premium of $100,000 per gate	$1,100,000
Tie-in dikes	Crest El 259 m – 100,000 m^3 @ $20/$m^3$	$2,000,000
Freeboard dikes along southeast rim of Lake Winnipegosis	5 km in length, Crest El 259 m, 1V:4H side slopes 420,000 m^3 @ $20/$m^3$	$8,400,000
Allowance to Mitigate Increased Water Levels	Estimated at $10 Million	$10,000,000
Environmental Mitigation Costs	15% of total direct costs	$4,485,000
Indirect Cost (Owners Costs, Engineering, Administration, Approvals)	20% of total direct costs	$5,980,000
Miscellaneous Quantity Allowance	10% of direct and indirect costs	$4,037,000
Contingency Allowance	30% of all costs	$13,320,000
	Total Cost	**$57,722,000**

14.4.6 Other Disadvantages

In addition to the obvious disadvantages related to the impacts of high water levels around Lake Winnipegosis, there are other important disadvantages that would detract from the viability of this scheme. They include:

- The gates in the control structure, which would normally be left in the inoperative position for non-flood conditions. Only once in many decades (on average) would there be the need to use these gates. This would lead to a potential for maintenance to be neglected as the years go by, leading quite possibly to an inoperative structure many years in the future when the need for use may arise suddenly. This was a major concern to MIT and was agreed to be a major disadvantage in the comparison of the Ste Agathe Detention Scheme versus the expansion of the Red River Floodway in the studies that led up to the expansion of the Floodway.
- The temporary retention of floodwaters in Lake Winnipegosis would indeed lower the peak water level during extreme floods in Lake Manitoba. However, the later release of the stored water would result in an extended duration of above natural water levels on Lake Manitoba for many weeks or months after the peak occurred. This could have far-reaching impacts that would need to be thoroughly understood and addressed.

14.5 STORAGE UPSTREAM OF LAKE WINNIPEGOSIS

An option for reducing flood inflows to Lake Manitoba was identified in the Technical Workshop held for the AR&LMB Study in February, 2013. At that Workshop, it was suggested that implementation of a control structure on the Shoal River at the outlet of Swan Lake could have merit in reducing inflows to Lake Winnipegosis, thereby affecting inflows and flood levels on Lake Manitoba.

Unfortunately, there was limited hydrometric data on the Shoal River and Swan Lake. Review for the current study was based on a conceptual level of assessment.

Field surveys and more detailed hydraulic analyses than have been possible in this study would be required for meaningful development of the option. Nevertheless, there appears to be a modest potential for this scheme to provide benefits in reduced flood potential in Lake Winnipegosis and in Lake Manitoba. On the other hand, the effects of raising water levels in Swan Lake over what would have occurred in the natural state without the control structure were uncertain. The lake was originally named for the trumpeter swans that inhabit the area. The environmental impact of flooding in the wetlands surrounding the lake would clearly require further consideration.

14.5.1 Location and Key Factors

Figures 14.25 and 14.26 show the location of Swan Lake and its outlet to the Shoal River at the north end of the lake.

Figure 14.25
Location of Swan Lake

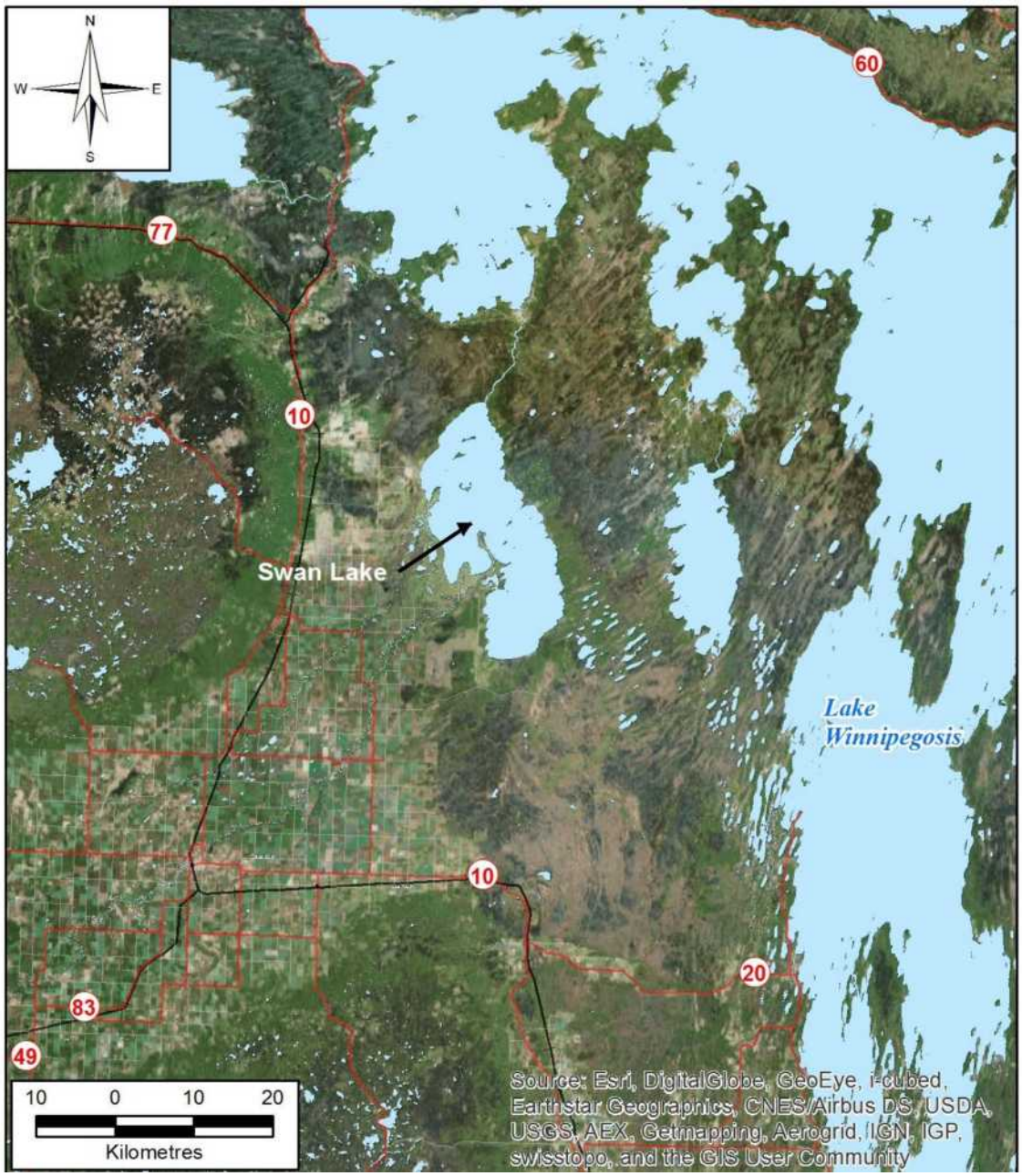
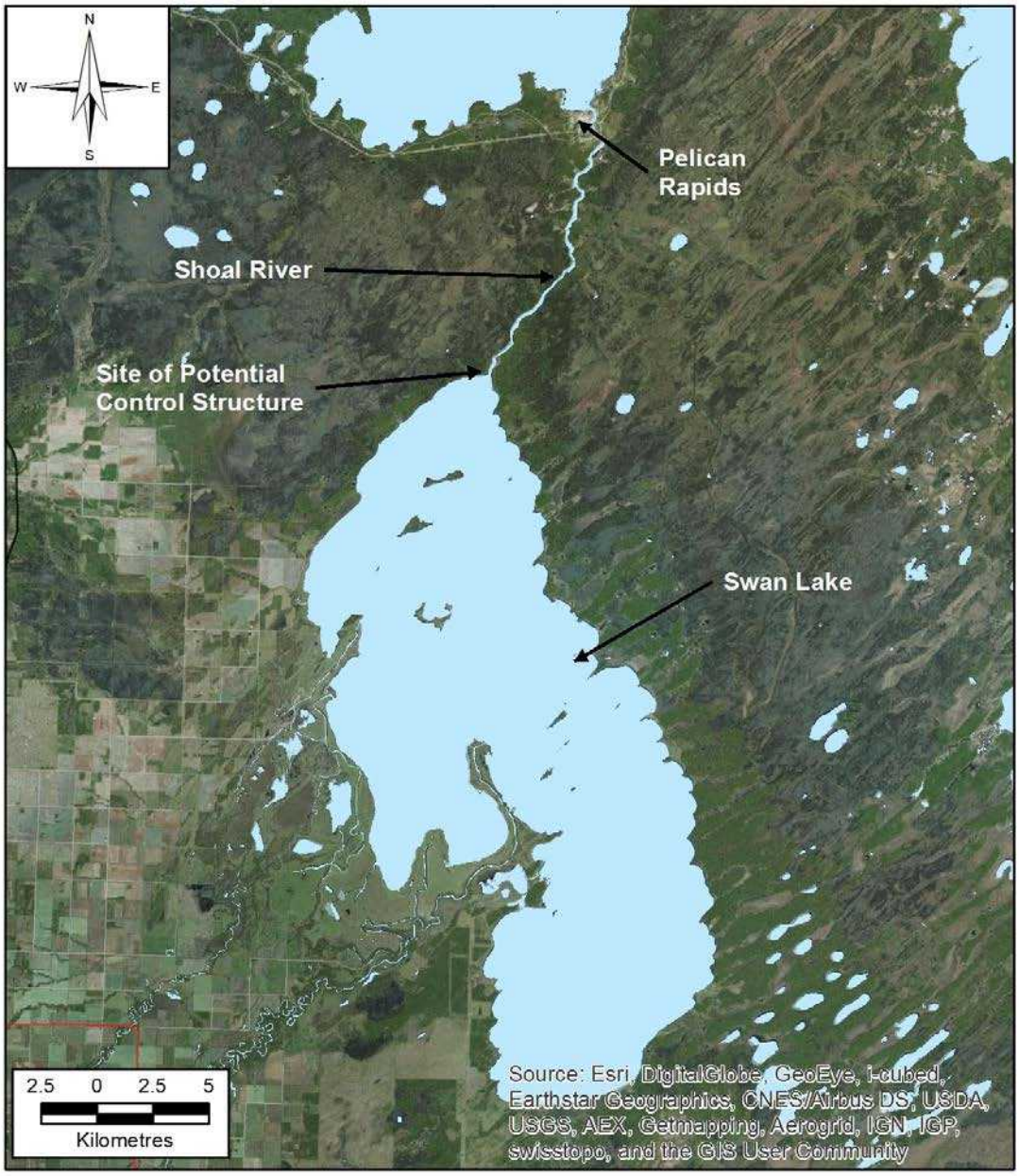
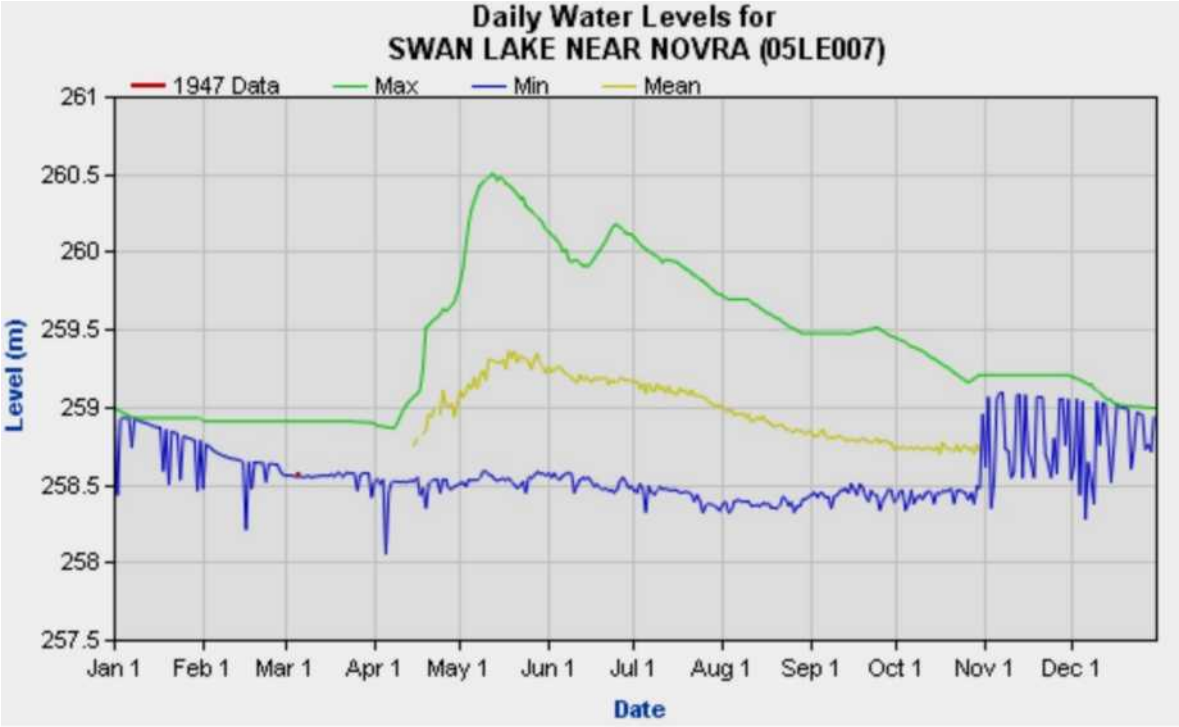


Figure 14.26
Location of Shoal River



The typical range of static water levels on Swan Lake (wind effects eliminated) is from approximately El 258.7 m (El 848.8 ft) up to El 259.4 m (El 851.1 ft), or a range of less than 1 m (3.3 ft). This is based on a 44-year record of water level measurements extending from 1947 to 1990 by Water Survey of Canada (Gauge #05LE007). Figure 14.27 shows the historical range of lake levels during this period.

Figure 14.27
Historical Water Level Summary for Swan Lake



The maximum lake level measured during the period of record occurred in the spring of 1988 (El 260.5 m) (El 854.7 ft) representing a rise of about 1.8 m (5.9 ft) from the level at the end of winter in that year.

The water level gauge was removed in 1990, and there has been no data since then, including the effects of the 2011 Flood, available for this study.

14.5.2 General Arrangement of Control Structure

The concept would involve the construction of a simple reinforced concrete structure with piers to enable the placement of stop logs as required in flood seasons to retain excess flood inflows. Development of preliminary designs has not been attempted.

14.5.3 Hydrological Studies

No detailed hydrological studies have been conducted, primarily because of the lack of hydrometric information. However, the following rationale can be considered in comparing this scheme with other options in the study area at this screening stage of study.

The drainage area of the watershed surrounding Swan Lake is estimated to be 10,100 km^2 (3,900 mi^2). This comprises approximately 18% of the drainage area to Lake Winnipegosis and 12% of the drainage area to Lake Manitoba. These low ratios alone suggest that there is limited capability for control of Swan Lake to assist significantly in preventing flooding downstream.

The surface area of the lake is approximately 310 km^2 (120 mi^2). If a control structure could be used to pond water up to 1 m (3.3 ft) (for example) above the water level that would have occurred naturally without that structure, then the flood inflow volume to Lake Winnipegosis could be reduced by about 310,000,000 m^3 (251,000 ac-ft). This represents approximately 6% of the inflow available for outflow (IAO) from Lake Winnipegosis in April/May/June 2011, and 3% of the IAO from Lake Manitoba in the same period. It was estimated that delay of the release of this small proportion of the inflow volume would have translated to a reduction in the peak water level in 2011 in Lake Winnipegosis by 8 cm (3.2 in), and in Lake Manitoba by 3 cm (1.2 in). Additional storage in excess of the example 1 m (3.3 ft) ponding in Swan Lake would of course provide more significant results downstream, but at the expense of increasing environmental impacts in and around Swan Lake.

The concept of a control structure on Swan Lake may have merit, but the potential benefits downstream would clearly be on a small scale. The downside would be the environmental impacts to the lake and its shoreline, as well as on the Shoal River.

14.6 CONCLUSIONS FROM THE STUDIES OF LAKE WINNIPEGOSIS

1. Inflow from Lake Winnipegosis forms a large proportion of the total inflow to Lake Manitoba. Temporary retention of excess floodwaters, or diversion to other water bodies could result in reductions in peak water levels on Lake Manitoba during extreme flood events.

2. Diversion of excess floodwaters from Lake Winnipegosis to Cedar Lake is feasible, but very costly. Diversion of floods would require pumping and excavation of a long, deep channel to extend between the two lakes. The estimated cost would range from approximately $160,000,000 to $320,000,000, depending on the discharge capacity selected. This range of investment would result in a reduction in the peak water level on Lake Manitoba of up to 0.4 m (1.3 ft) in a repeat of a flood event comparable to 2011.

3. The use of the diverted water in Cedar Lake to generate additional energy at Manitoba Hydro's Grand Rapids Hydroelectric Generating Station would only amount to a small financial benefit (less than 4% per year of the annual costs of the scheme).

4. Diversion of floodwaters from Lake Winnipegosis to Lake Winnipeg would avoid excessive inflow into Lake Manitoba and is feasible. However, it would be very costly, requiring an 80-km long channel with a series of concrete drop structures to dissipate the excessive energy from the 30+m differential between Lake Winnipegosis and Lake Winnipeg. The estimated cost of such a scheme would be in excess of $1.3 billion for a discharge capacity of 283 m^3/s (10,000 cfs). This would permit a reduction of approximately 0.4 m (1.3 ft) in a repeat of a flood event comparable to 2011.

5. Temporary retention of excess floodwaters in Lake Winnipegosis would be made possible by construction of a gated control structure on the upper reach of the Waterhen River. The gates would only be used in extreme floods and would be lowered to restrict the outflow from Lake Winnipegosis. It has been estimated that, in general, a one-half metre reduction in peak water level on Lake Manitoba would require retention of a volume of water in Lake Winnipegosis that would result in an increase in water level of 1 m (3.3 ft) on Lake Winnipegosis. The cost of the control structures required for such a scheme is estimated to be $43 million. Detailed investigations beyond the scope of the current study are necessary to determine whether the costs allocated, for mitigation works at existing infrastructure that would be affected by increased water levels, are sufficient. They are currently assumed to be approximately $10,000,000.

6. While the use of temporary storage of floodwaters in Lake Winnipegosis is theoretically possible, there are significant disadvantages including:

   a) The delay of release of the floodwaters would result in lower peak water levels on Lake Manitoba, but an extended duration of generally high water levels as the flood gradually works its way through the system.

   b) Infrequent use of the gates (at least 30 years between usages on average) would tend to leave the structure and its equipment vulnerable to neglect of maintenance and long-term understanding of the functioning of the control system(s).

   c) The potential significant environmental impacts associated with the increased water levels on the lake and the construction of a dam on the Waterhen River. These impacts would almost certainly be viewed as unacceptable by the local stakeholders. It would be expected that the local residents of the area would perceive that their interests were being sacrificed for the betterment of the users of Lake Manitoba.

   In the face of these serious disadvantages, further advancement of the option was considered unacceptable.

7. There appears to be some modest potential for control of outflow from Swan Lake to reduce flood flows into Lake Winnipegosis and in turn into Lake Manitoba. However, the effect would be small, and the environmental impact on Swan Lake and the Shoal River could be significant.

DRAWINGS

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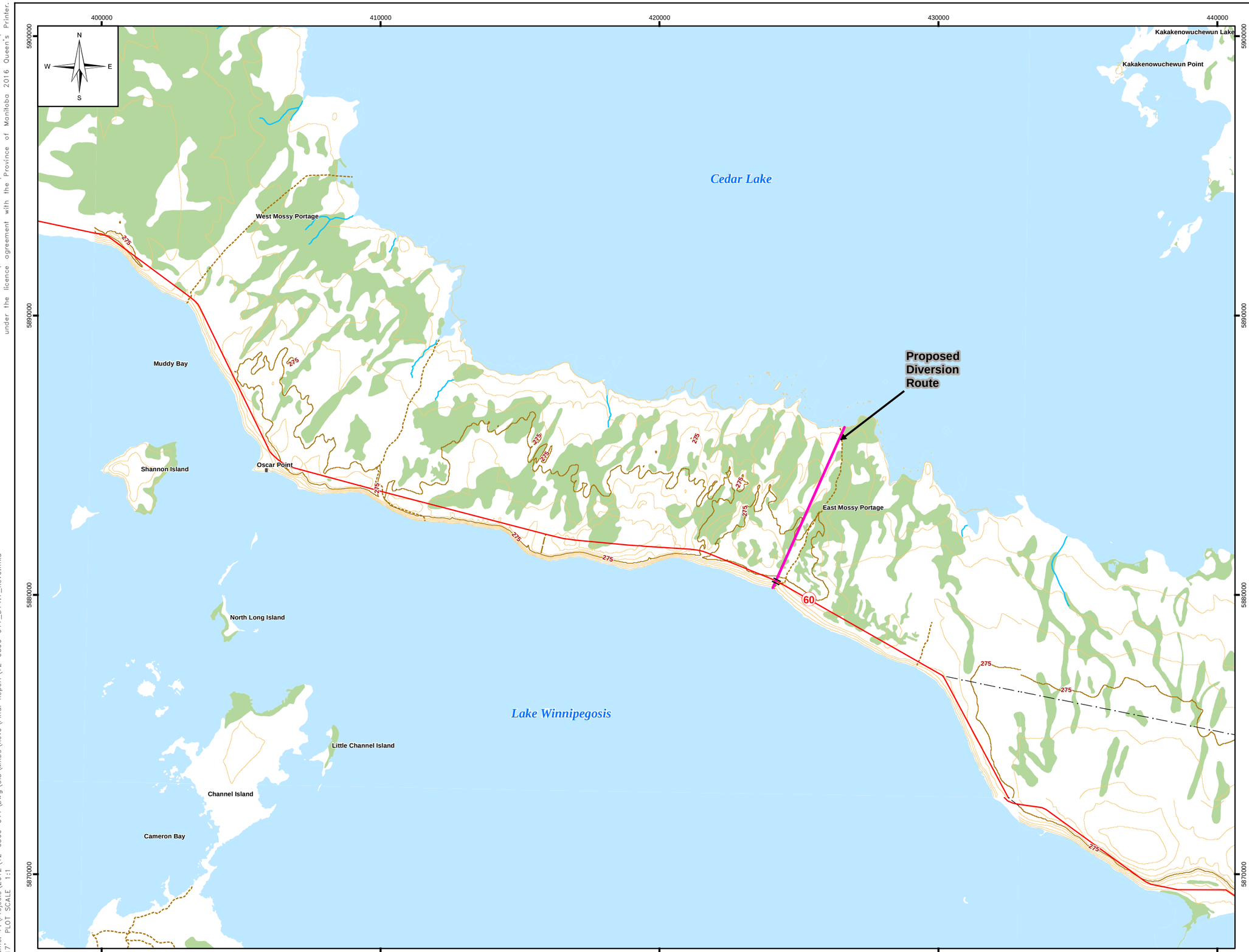

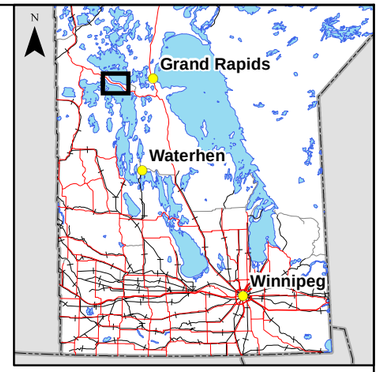

LEGEND:

- Road
- Limited–Use Road
- Trail
- Transmission Line
- Watercourse
- Waterbody
- Wetland
- 25m CDED Index Contour
- 5m CDED Contour
- Proposed Diversion Route
- Bridge

NOTE:

Topographic data is from the National Topographic Database, supplied by NRCAN.

1 0 1 2 3 4 5
Kilometres

SCALE: 1:130,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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0	16/01/04	ISSUED WITH FINAL REPORT	PAL

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

LOCATION PLAN OF FLOOD DIVERSION CONCEPT TO CEDAR LAKE

JANUARY 2016 | DWG 14.1 | REV: 0

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\Rev0\Final Report\12-0300-011_D14.1_Rev0.mxd
11"x17" PLOT SCALE 1:1

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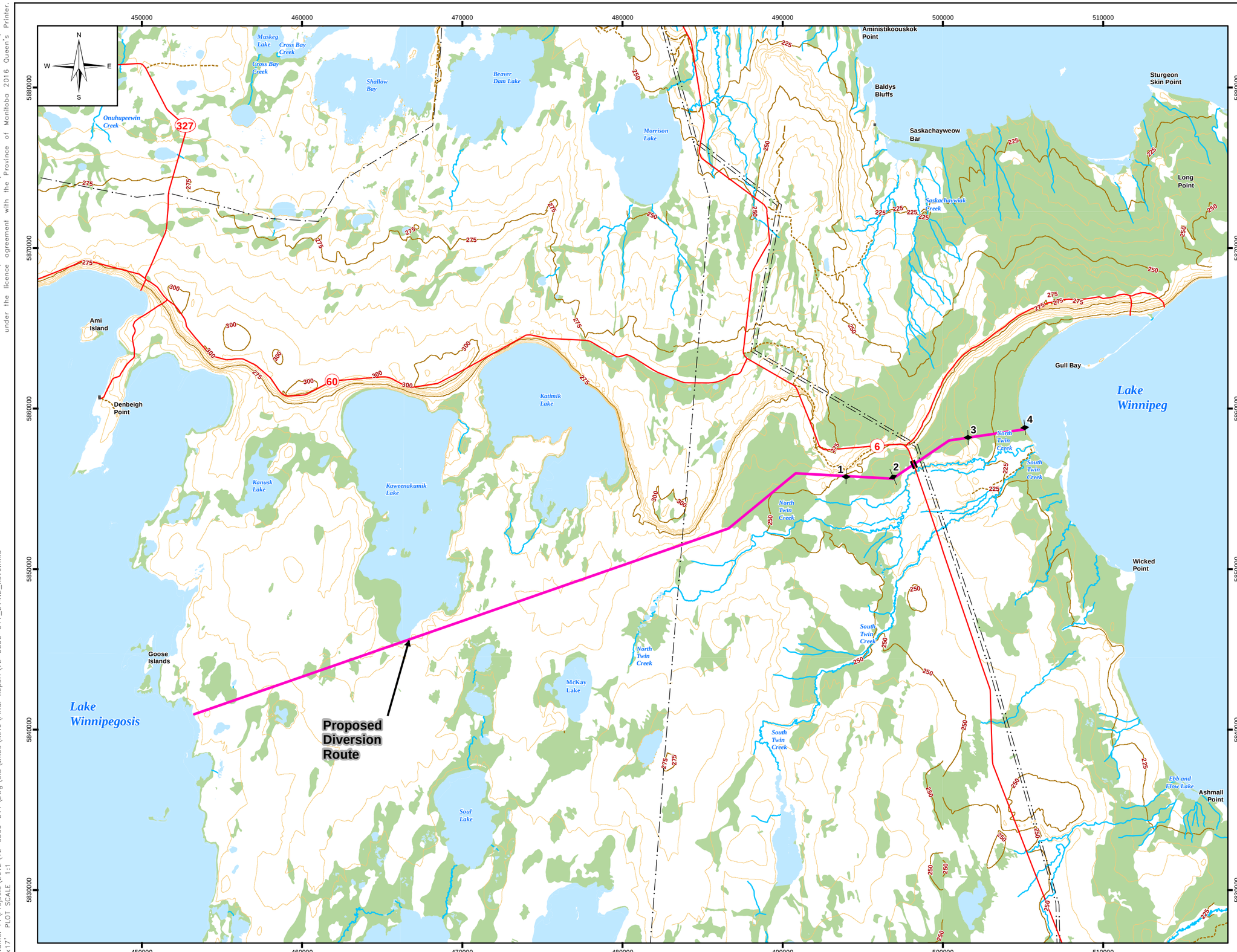

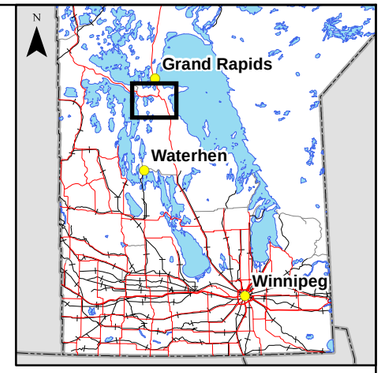

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- Bridge

NOTE:

Topographic data is from the National Topographic Database, supplied by NRCAN.

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Kilometres

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All units are metric and in metres unless otherwise specified.
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Elevations are in metres above sea level (MSL)

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REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

LOCATION PLAN OF FLOOD DIVERSION CONCEPT TO LAKE WINNIPEG

JANUARY 2016 | DWG 14.2 | REV: 0

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15.0 LAKE MANITOBA AND LAKE ST. MARTIN

15.1 BACKGROUND

In 2011, record, widespread flooding occurred across much of southern Manitoba resulting in unprecedented high inflows into Lake Manitoba through the Waterhen River, Whitemud River, Portage Diversion, from saturated groundwater storage and from local ungauged contributing drainage area. These high flows extended well into the summer and overwhelmed the capacity of the existing regulatory system. The result was that Lake Manitoba crested at 249.1 m (817.2 ft), which was 1.43 m (4.7 ft) above the desirable range of 247.0 to 247.7 m (810.5 ft to 812.5 ft). Flooding around Lake Manitoba caused significant damage to hundreds of properties around the lake, particularly during a storm in late May when winds reached over 100 km/h (62 mi/h), which raised the lake by 2.1 m to 2.7 m (7 to 9 ft).

The inflow to Lake St. Martin from Lake Manitoba was greater than the natural outflow capacity, causing Lake St. Martin to crest at 245.6 m (805.5 ft), which was 1.68 m (5.5 ft) above the desirable operating range of 242.9 to 243.8 m (797.0 to 800.0 ft). Flooding on Lake St. Martin prompted the emergency construction of dikes up to 2.4 m (8 ft) high with a top elevation of 246.6 m (809 ft). Road access was severely limited to several communities and widespread long-term evacuation from the four First Nations around Lake St. Martin and the Dauphin River was required.

In June 2011, the Province of Manitoba commissioned KGS Group and AECOM (the “2011 Team”) to urgently explore options to bring the levels of Lake St. Martin and Lake Manitoba down to the desirable range on an emergency basis. The Province sought a broad review of potential options to achieve this objective in a timely and cost-effective manner while also minimizing the potential impact on other areas of the Province. The scope of services performed in 2011 included:

- Identification and screening level assessment of over 11 diversion channel options and 4 historic mitigation options (e.g., Holland Dam).
- Geotechnical field investigations (peat probes, test pits, boreholes and geo-seismic) to estimate rock and overburden excavation quantities.
- Acquisition and processing of GIS data (survey, LiDAR, air photos, and sonar).

- Hydrodynamic and ice modeling to assess the potential effects of ice on the performance of the various channel options.
- Preliminary assessment of costs and constructability for each of the alternatives.

A report was prepared summarizing the results of the investigation titled, "*Analysis of Options for Emergency Reduction of Lake Manitoba and Lake St. Martin Levels*" (KGS Group and AECOM, 2011), which identified the Dauphin River as the hydraulic restriction in the system; and on July 22, 2011 the team recommended the following:

- Begin immediate construction of a 140 m^3/s (5,000 cfs) emergency outlet channel from Lake St. Martin to Buffalo Lake and Creek, on route to Lake Winnipeg to address the hydraulic flow restrictions out of Lake St. Martin and to accommodate additional Lake Manitoba outflows over the winter.
- Allow unrestricted outflow of water from Lake Manitoba through the FRWCS during the winter of 2011/2012, allowing several times more outflow than past winters.

Subject to the successful implementation of the above recommendation, the team also conditionally recommended that the Province of Manitoba consider:

- Construction of a 70 m^3/s (2,500 cfs) bypass channel around the north side of the FRWCS to allow additional outflow from Lake Manitoba.
- Expansion of the emergency outlet channel from Lake St. Martin by 70 m^3/s (2,500 cfs) to offset the additional inflows from Lake Manitoba on the principle of no net additional flow to Lake St. Martin.

The construction of Reach 1 of the Lake St. Martin Emergency Outlet Channel (LSMEOC) was completed on November 1, 2011; but due to time constraints and construction challenges, the original design channel base width was reduced, resulting in a design capacity of approximately 106 m^3/s (3,750 cfs; actual capacity was 113 m^3/s (4,000 cfs) based on hydrometric data collected during operation of the LSMEOC in 2011 and 2012). The proposed 70 m^3/s (2,500 cfs) FRWCS bypass channel was not constructed, as an expansion of the LSMEOC by 70 m^3/s (2,500 cfs) to offset the additional flow was not feasible in 2011. Now that the 2011 emergency has passed, the conditional recommendations noted above must be re-evaluated.

During operation, the water from the LSMEOC flowed into a bog area around Big Buffalo Lake before flowing into Buffalo Creek and finally discharged into the lower Dauphin River upstream of Sturgeon Bay (Lake Winnipeg). During the early stages of the project, there was concern that

the hydraulic restriction at the upstream end of Buffalo Creek would result in extensive flooding of the bog area during operation of Reach 1. The concern was that the water would potentially spillover and move overland towards the Dauphin River and lower portions of Buffalo Creek. To eliminate this potential flow restriction, an option was considered which involved excavating a natural segment of the upstream end of Buffalo Creek (Reach 2); however, it was later determined that the natural creek section could convey the peak flow in 2011, without requiring any modifications.

As the Reach 1 project developed, hydraulic modeling of potential water levels at the mouth of the Dauphin River indicated that there was a significant risk of major flooding of the Dauphin River communities in the spring of 2012 due to ice jam formations. This was largely due to the unprecedented winter flows with the FRWCS running at full capacity all winter. Therefore, it was determined that construction of the Reach 3 Emergency Channel (Reach 3) would be required to divert flows away from the Dauphin River prior to spring break up. The Reach 3 channel, in combination with dikes being constructed along the banks of the Dauphin River, would significantly reduce the risk of flooding for the Dauphin River communities. KGS Group was directed by MIT to proceed with the development of an appropriate design for Reach 3 in August 2011. The Reach 3 Emergency Channel was partially constructed between January and March 2012. However, with extremely mild winter conditions, ice jams on the Dauphin River did not develop as predicted. This ultimately precluded the need for Reach 3 to be operated in the spring of 2012. The Reach 3 Emergency Channel currently remains in an incomplete condition particularly with respect to some hydraulic structures.

The Reach 1 Emergency Outlet Channel was operated beginning in November 2011 and closed in November 2012, as required under the federal government terms and conditions for emergency operations. A longer-term environmental monitoring program is being implemented to ensure the full impacts of the emergency channel are documented and mitigation requirements identified. Unless required again for emergency operation, the channel will not be re-opened until it complies with the normal approval and licensing requirements under *The Environment Act*.

As a follow-up to the 2011 emergency flood mitigation activities for the Assiniboine River & Lake Manitoba Basins, the Province of Manitoba commissioned two independent review groups: the

Lake Manitoba and Lake St. Martin Regulation Review Committee (the Committee) chaired by Harold Westdal; and the Manitoba 2011 Flood Review Task Force (the Task Force) chaired by David Farlinger.

The Committee made the following recommendations with respect to the LMB and LSM Outlet Channels:

- *"The Committee recommends that the Lake St. Martin Emergency Channel be made permanent. The capacity should be sufficient such that Lake St. Martin can be maintained within the desirable range of 242.9 to 243.8 m (797 to 800 ft) at least 90 percent of the time.*
- *Consideration should be given to the inclusion of a control structure at the mouth of the permanent outlet channel to limit outflow at times of low lake levels.*
- *The current Reach 3 should be redirected so that the channel outlets south of Willow Point on Lake Winnipeg.*
- *The Committee recommends that a second channel be constructed between Lake Manitoba and Lake St. Martin that would provide the total outlet capacity to meet the original design criteria for the Fairford Control Structure. If the Province builds a new outlet to Lake Manitoba as recommended, it should take that opportunity to design the new outlet in such a way as to provide unrestricted fish passage between Lake Manitoba and Lake St. Martin".*

The Task Force chaired by David Farlinger made the following recommendations with respect to LMB & LSM Outlet Channels:

> *"Adopt the recommendations of the Lake Manitoba and Lake St. Martin Regulation Review Committee regarding additional outlet capacity requirements for Lake Manitoba and Lake St. Martin, with due consideration for the engineering studies that are being conducted".*

15.2 DATA AVAILABLE

Stage 1 of the current Conceptual Design Study is primarily a desktop review based on existing available data described in the following paragraphs (no new field programs were included in the scope).

The key reports that were reviewed in detail to understand the issues and options addressed previously included the following:

- Analysis of Options for Emergency Reduction of Lake Manitoba and Lake St. Martin Levels Report, Executive Summary (KGS Group and AECOM, 2011).
- Lake Manitoba and Lake St. Martin Regulation Review Committee Report (LM&LSMRC, 2013).
- The Manitoba 2011 Flood Review Task Force Report (MFRTF, 2013).
- Flood Mitigation Study for First Nations along Fairford River, Lake St. Martin, and Lake Pineimuta (AECOM, 2012).
- Regulation of Water Levels on Lake Manitoba and along the Fairford River, Pineimuta Lake, Lake St. Martin, and Dauphin River and Related Issues (LMRRAC, 2003).
- Lake St. Martin and Pineimuta Lake Regulation (Manitoba, 1978).
- Lake Manitoba Regulation (Manitoba, 1972 to 1984).
- Lake Manitoba Regulation – Volume 1 and 2 (Manitoba, 1973).
- Lake Manitoba Regulation Operating Rules of the Fairford River Dam (Manitoba, 1961).
- Report on Measures for the Control of the Waters of Lake Winnipeg and Manitoba – Supplementary Volume II (LWMB, 1958).
- Investigation of Means of Lowering Lake Manitoba – Report on Auxiliary Channel Scheme No. 2 on Fairford River (Verner, 1954).

As part of the overall assessment of channel options a geotechnical investigation was completed in 2011 along most of the proposed emergency channel routes to supplement existing data available from the Province of Manitoba, Conservation and Water Stewardship, Water Well Records (GWDrill). The primary focus of the 2011 geotechnical investigation program was to collect subsurface information in the vicinity of the proposed routes to allow the preliminary channel designs to be refined with the development of subsurface profiles to identify major design constraints such as excessive bedrock excavation requirements. The geotechnical investigation consisted of test pits, boreholes, monitoring wells, peat probes, lake bottom soundings, laboratory testing and geo-seismic surveys. The data obtained from the 2011 geotechnical investigations is documented in an appendix within the Analysis of Options for Emergency Reduction of Lake Manitoba and Lake St. Martin Levels Report (KGS Group and AECOM, 2011) and includes the following:

- Detailed test pit logs, to determine the depth to bedrock to refine cost estimates for proposed channel construction.
- Detailed overburden and bedrock core logs to classify the materials (stratigraphy) and depth to bedrock along the proposed channel routes.
- Piezometric elevations in the bedrock to interpret groundwater conditions along the proposed channel routes.
- Summary table of peat depths and consistency of materials beneath the peat along sections of proposed routes that traverse peat bogs.
- Summary table of approximate depth of water and lake bottom sediments on lakes within bog areas situated on and adjacent to proposed channel routes.
- Seismic refraction surveys to provide an estimate of the bedrock surface elevation in select areas where test pitting was insufficient to confirm the depth to bedrock, and where drill access was problematic.

The primary GIS data used for the project consisted of terrestrial and bathymetry surveys collected by KGS Group, the Province of Manitoba and other consultants, as well as data from LiDAR and aerial and satellite imagery. Large area terrain models from Natural Resource Canada (NRCAN) and Shuttle Radar Topography Mission (SRTM) were used initially to develop project wide contour datasets at larger scales. The data and information collected during the 2011 emergency work program was also used in the current Stage 1 phase of the Conceptual Design Study for the evaluation of outlet channel options. ATLIS Geomatics acquired and classified LiDAR point cloud data for approximately 1,960 km² (760 mi²) during July 2011. The LiDAR data provided key information for all of the potential channel options and surrounding area including Birch and Buffalo Creeks, Fairford and Dauphin Rivers, as well as Lake Pineimuta and Lake St. Martin. The LiDAR data was processed by KGS Group to produce a bare earth Digital Elevation Model (DEM) and contours at a 0.25 m vertical interval.

In addition to the data described above, bathymetric surveys were conducted on Lake Manitoba of the proposed outlet channel inlet locations. The data collected during the surveys was used to supplement the hydraulic analyses and to help better define the outlet channel rating curves. The bathymetric data is provided in Appendix P.

15.3 DESCRIPTION OF ALTERNATIVES AND HYDRAULIC DESIGNS

It is clear from presentations made to the Lake Manitoba / Lake St. Martin Regulation Review Committee, comments from the public, and survey responses that there is an overwhelming interest and demand for an additional outlet from Lake Manitoba. To ensure that flooding would not be aggravated on Lake St. Martin, a similarly-sized channel would be required from Lake St. Martin to Lake Winnipeg. Many have suggested that additional outlet capacity must be sufficient to avoid the flooding that would be experienced in a repeat of a 2011-sized flood. However, it will be difficult to justify the addition of a second outlet from Lake Manitoba sized to accommodate such a rare event. Regardless, the Review Committee encouraged the Province to undertake the necessary engineering studies and to develop cost estimates to provide the information necessary to address the public concerns. A preliminary analysis suggested that an increase of about 35% in the FRWCS capacity would be required to maintain Lake Manitoba levels within the target range set by the designers of the FRWCS in all years except 2011. Therefore, the Committee recommended that a second channel be constructed between Lake Manitoba and Lake St. Martin to provide the total outlet capacity required to meet the original design criteria for the FRWCS.

As part of the Analysis of Options for Emergency Reduction of Lake Manitoba and Lake St. Martin Levels Study, five emergency outlet channel routes (and several variations in capacity for each) were investigated to bypass the FRWCS and convey additional discharge to Lake St. Martin. The alignment of these channel options (A through E; Drawing 15.1 located at the end of Section 15.0) have not changed significantly from those considered in 2011. For the current study, the level of design was advanced for each of the routes, a new route (Option F) was added and additional capacities were evaluated that were not previously considered. The current study also examined various capacities and options for the Lake St. Martin outlet channels to achieve no net additional inflow into Lake St. Martin meeting additional requirements that were suggested by the Lake Manitoba / Lake St. Martin Regulation Review Committee. The channels are expected to operate over a range of water levels on Lake Manitoba and Lake St. Martin. However, for purposes of the design, the nominal channel capacities have been established for lake levels of 248.1 m (814 ft) on Lake Manitoba and 244.1 m (801 ft) on Lake St. Martin.

15.3.1 Lake Manitoba Outlet Channels

The Lake Manitoba outlet channel options include the five alignments previously examined along with one new channel option (F). These are shown on Drawing 15.1 (located at the end of Section 15.0) and described as follows:

- Option A – Twinning the Fairford River.
- Option B – Channel south of Pinaymootang First Nation.
- Option C – Channel slightly less south of Pinaymootang First Nation.
- Option D – Channel following Birch Creek.
- Option E – Bypass channel north of the FRWCS.
- Option F – Expansion of the Fairford River and FRWCS (including dredging of the inlet channel).

The assessment of options included evaluation of additional channel capacities that were not previously considered for several of the original alignments. Except for Option E, a conceptual design was completed for each option for a capacity of 140 m^3/s (5,000 cfs) at a Lake Manitoba elevation of 248.1 m (814 ft). Additionally a conceptual design was completed for Options A, B, C and D for a capacity of 283 m^3/s (10,000 cfs) and 424 m^3/s (15,000 cfs). For Option E, a conceptual design was completed for capacities of 70 m^3/s (2,500 cfs) and 106 m^3/s (3,750 cfs) at a Lake Manitoba elevation of 248.1 m (814 ft). These additional channel capacities for each outlet channel option have been designated numerically (e.g., A1, A2, A3, B1, etc.).

The channels were designed with a trapezoidal cross section with 4:1 side slopes in overburden (till) and near vertical 1:4 side slopes in bedrock. The channel slope for most options was 0.013%, although it was 0.032% for Option A and 0.030% for Option E. The channel roughness used for all options was 0.028 in till and 0.032 in bedrock, except for Option F, which ranged from 0.025 to 0.031 (till). The depth of flow was 3.4 m for Options B and C, 3.5 m for Options A and E, 4.0 m for Option D and varied for Option F. The base width and average top width for each option varied depending on the material and design flow as described in the following sections for each option. The design maximum flow velocity for the channels was generally 1.5 m/s (5.0 ft/s) in till. For Option A, the average flow velocities ranged from 1.1 to 1.2 m/s all in till (3.6 to 3.9 ft/s), and for Option D, they ranged from 0.5 to 0.7 m/s all in till (1.6 to 2.3 ft/s), whereas for Option E the velocities were 0.9 m/s in till (3.0 ft/s). The flow velocities in the various channel alternatives varied due to the available hydraulic gradient from the downstream end of the channel to the upstream lake and from channel hydraulic efficiencies due to different

width to depth ratios. Detailed information for each option is provided in the sections that follow and is also summarized in Table 15.1 (located at the end of Section 15.0).

Table 15.1
Design Criteria for Outlet Channels on Lake Manitoba and Lake St. Martin
(This table is located at the end of Section 15.0.)

Excavation volumes for each option were estimated using the design channel geometry, the ground surface based on Shuttle Radar Topography Mission (SRTM) data and LiDAR data, the bedrock surface based on well drilling data base (GWDrill), geophysical exploration, and test-hole data.

The following sections provide a description of each of the Lake Manitoba outlet channel alternatives and the basis for the development of the associated excavation volumes. The calculated hydraulic impact of the outlet channels for the various capacities is provided in Section 15.4, while cost estimates for each of the options are provided in Section 15.5.

15.3.1.1 Option A

The Option A outlet channel, which is comprised of the twinning of the existing Fairford River from just upstream of the FRWCS to Lake St. Martin, crosses through the Fairford River at two locations as shown in Drawing 15.2 (located at the end of Section 15.0). One of the design criteria for this option included increasing the capacity of the Fairford River without increasing the surface level elevation at any location in the river. The proposed route is approximately 12.5 km (7.8 mi) long and located primarily within the Pinaymootang First Nation, with a portion on privately held lands. This option would require construction of two new control structures, with one incorporated into a new bridge where the channel crosses Provincial Trunk Highway 6 (PTH 6). This upstream control structure would regulate discharges from Lake Manitoba for the purpose of maintaining target levels on the lake (i.e., supplement the FRWCS when lake levels are high). The second control structure would be located downstream of Lake Pineimuta and would be incorporated into a new bridge where the channel crosses the municipal road. This downstream control structure would be required to maintain Lake Pineimuta levels. Without the second control structure, Lake Pineimuta levels would be reduced in an uncontrolled manner during periods in which the channel was not required to be operated. The Option A channel

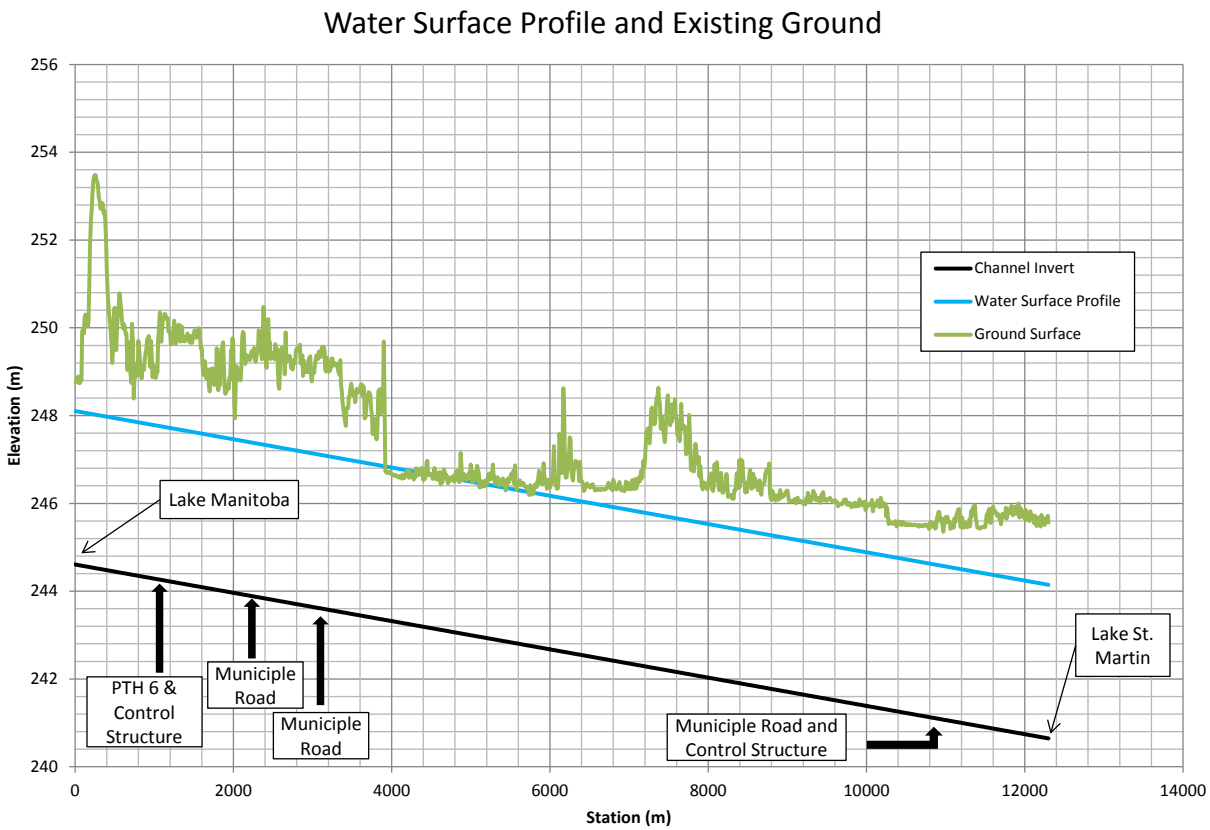
crosses another three municipal roads, with new bridges proposed at two of these locations to minimize disruption of road access. It could be possible to reduce the number of new bridge crossings required by localized realignment of roads.

The excavation for Option A was assumed to be primarily in till. The bedrock surface elevation was assumed to lie below the level of the channel invert based on GWDrill data and from bedrock surface elevation DEM maps compiled by Natural Resources Canada. The channel lengths, widths and estimated excavation volumes have been summarized in Table 15.2. A profile of Option A showing existing ground elevation, channel invert, water surface and key infrastructure is provided on Figure 15.1.

Table 15.2
Option A Excavation Volume Estimates

Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
A1	5000	Till	12300	26	4:1	64	4.7	2.63
A2	10000			64		83		4.83
A3	15000			112		131		7.61

Figure 15.1
Profile of Lake Manitoba Outlet Channel Option A



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

15.3.1.2 Option B

The Option B outlet channel south of Pinaymootang First Nation was considered to bypass First Nation land and to improve construction conditions by going through an area that was not impacted by flooding. This route also bypassed Lake Pineimuta, which was seen as an advantage since flooding on this lake affects First Nation communities and agricultural land. As well, bypassing Lake Pineimuta eliminates the need for a downstream control structure. Option B would connect Portage Bay on Lake Manitoba to Lake St. Martin, south of the Pinaymootang First Nation as shown in Drawing 15.2 (located at the end of Section 15.0). The proposed route is approximately 11.5 km (7.1 mi) long and is situated on privately held lands and crown leased lands.

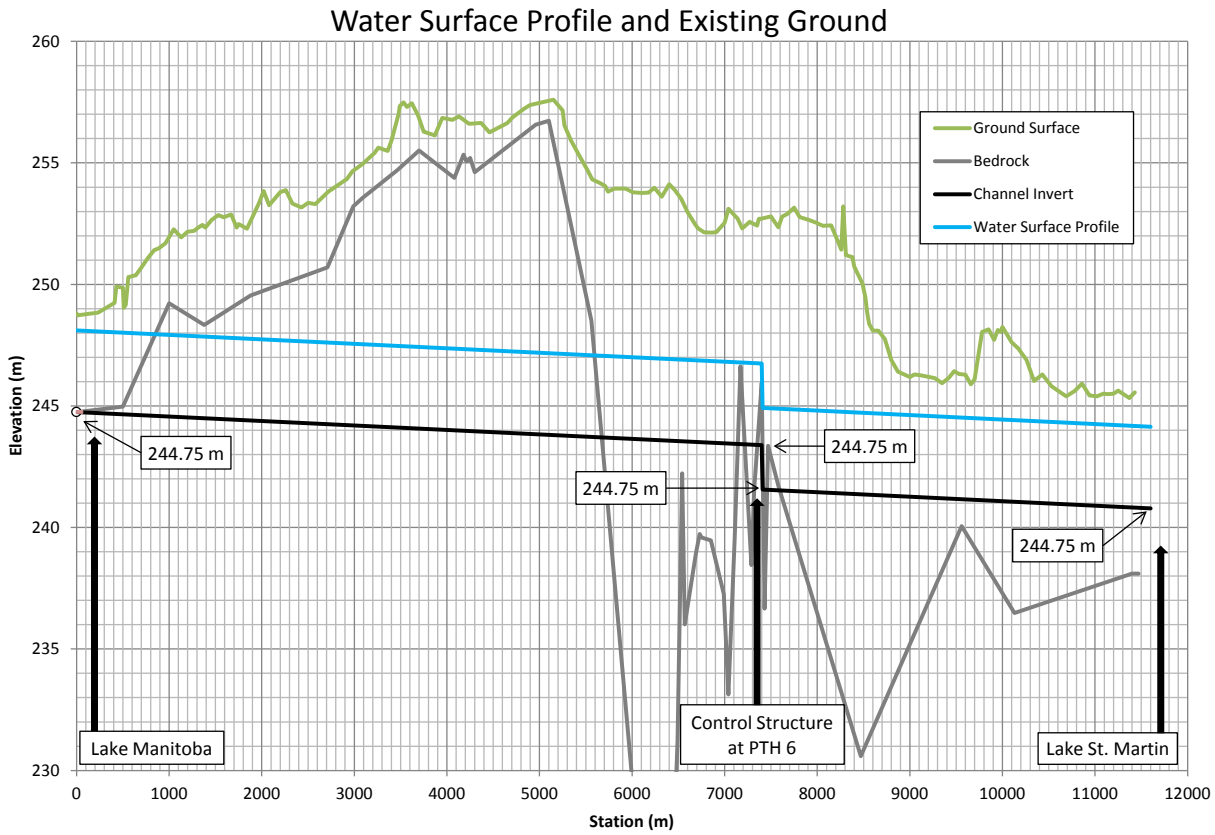
This option would require construction of new bridge crossings for one municipal road and for PTH 6. The bridge at PTH 6 will be incorporated into construction of a new control structure and drop structure (Figure 15.2). The control, drop and bridge structures have all been combined into one structure to reduce construction costs. The control structure was located near the mid-point of the channel length to allow for a narrower and less costly structure than if it was located at the upstream municipal road near Lake Manitoba. The proposed location also helps reduce the potential groundwater impacts caused by the channel since having the structure located further downstream results in higher water levels in the channel upstream when the structure is closed. For these reasons, the proposed location of the control structure at PTH 6 formed the base case for this option. However, the proposed location should be further considered at the next stage of design to confirm its preferred location.

Option B would require excavation of a significant bedrock ridge in the first half of the channel and for the channel to have rock at its base for roughly half of the total length. The channel lengths, widths and estimated excavation volumes have been summarized in Table 15.3. A profile of Option B showing existing ground elevation, channel invert, water surface and key infrastructure is provided on Figure 15.2.

Table 15.3
Option B Excavation Volume Estimates

Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
B1	5000	Til	5760	31	4:1	82	8.5	4.13
		Rock	5740	47	1:4			1.98
B2	10000	Til	5760	72	4:1	125		6.85
		Rock	5740	92	1:4			3.77
B3	15000	Til	5760	113	4:1	167		9.58
		Rock	5740	136	1:4			5.56

Figure 15.2
Profile of Lake Manitoba Outlet Channel Option B



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

15.3.1.3 Option C

The first half of the Option C outlet channel is situated slightly north of Route B and just south of Pinaymootang First Nation. This alignment was chosen to reduce the amount of rock excavation required in comparison to Option B, while still bypassing First Nation land. Option C would connect Portage Bay on Lake Manitoba with Lake St. Martin, as shown in Drawing 15.2 (located at the end of Section 15.0). The proposed route is approximately 11.6 km (7.2 mi) long and situated on privately held and crown leased lands.

Similar to Option B, this option would require construction of new bridge crossings at two municipal roads and at PTH 6, with the bridge at PTH 6 combined with a new control structure and drop structure (Drawing 15.2). The control, drop and bridge structures have all been combined into one structure to reduce construction costs. The control structure was located

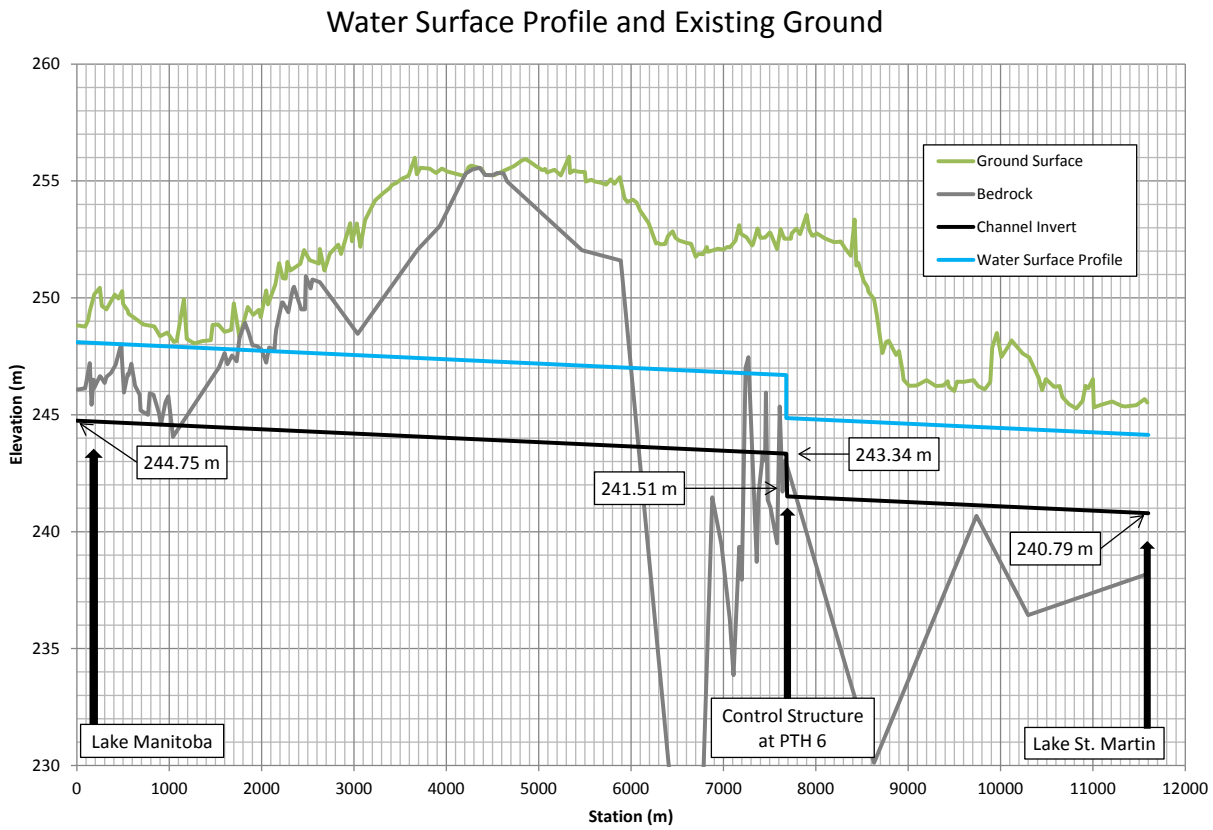
near the mid-point of the channel length to allow for a narrower and less costly structure than if it was located at the upstream municipal road near Lake Manitoba. The proposed location also helps reduce the potential groundwater impacts caused by the channel since having the structure located further downstream results in higher water levels in the channel upstream when the structure is closed. For these reasons, the proposed location of the control structure at PTH 6 formed the base case for this option. However, the proposed location should be further considered at the next stage of design to confirm its preferred location.

The geophysical exploration found that a significant bedrock ridge must still be excavated in the first half of the channel and that the channel would have rock at its base for over half of the total length. However, estimated rock and till construction volumes were slightly lower than Option B. The channel lengths, widths and estimated excavation volumes are summarized in Table 15.4. A profile of Option C showing existing ground elevation, channel invert, water surface and key infrastructure is provided on Figure 15.3.

Table 15.4
Option C Excavation Volume Estimates

Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
C1	5000	Til	5470	31	4:1	79	7.9	3.47
		Rock	6090	47	1:4			1.74
C2	10000	Til	5470	72	4:1	122		5.84
		Rock	6090	92	1:4			3.32
C3	15000	Til	5470	113	4:1	165		8.18
		Rock	6090	136	1:4			4.90

Figure 15.3
Profile of Lake Manitoba Outlet Channel C



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

15.3.1.4 Option D

The Option D outlet channel route connects Watchorn Bay on Lake Manitoba to the outlet of Birch Creek on Lake St. Martin as shown in Drawing 15.3 (located at the end of Section 15.0). This proposed route alignment is adjacent to low-lying terrain between Lake Manitoba and Lake St. Martin along which numerous marshes and small lakes exist. The proposed route is approximately 22.8 km (14.2 mi) long and situated on privately held and crown leased lands.

This option would require construction of new bridge crossings at three municipal roads, Provincial Road (PR) 239 and PTH 6. The bridge at PTH 6 would be combined with a new control structure and drop structure (Drawing 15.3). The control, drop and bridge structures have all been combined into one structure to reduce construction costs. The control structure was located at PTH 6, which is near the mid-point of the channel length to allow for a narrower

and less costly structure than if it was located at the upstream municipal road near Lake Manitoba. The proposed location also helps reduce the potential groundwater impacts caused by the channel since having the structure located further downstream results in higher water levels in the channel upstream when the structure is closed. For these reasons, the proposed location of the control structure at PTH 6 formed the base case for this option. However, the proposed location should be further considered at the next stage of design to confirm its preferred location.

Local land drainage from the west may be impeded along the upstream half of the channel. This issue can be addressed by the construction of an outside drain with the flow directed into the channel downstream of the control structure at PTH 6. Land drainage along the downstream portion may be improved as it can be redirected directly into the new channel. Similarly, land drainage along the upstream half could be improved by constructing an outside drain along the channel and directing the flow downstream of PTH 6 to Lake St. Martin.

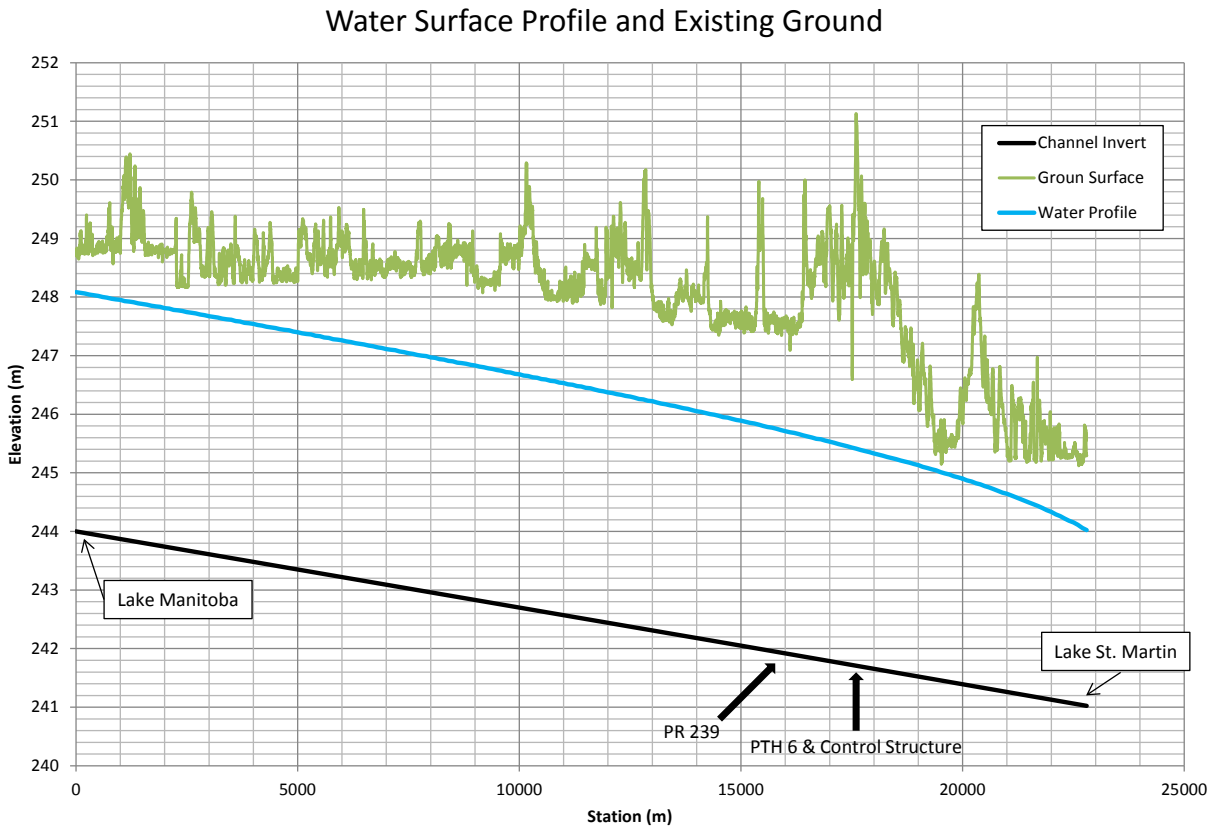
While the channel crosses a fourth municipal road just south of Lake St. Martin, a minor road realignment would be constructed rather than constructing another bridge crossing. It may be possible to further reduce the number of new bridge crossings required by localized realignment of roads, in particular PR 239 could be re-routed north to avoid crossing the new channel alignment.

While Option D is longer than Options B and C, the excavation depths and total volume of excavation required is less. Information from GWDrill and from preliminary test hole data also showed that by shifting the channel alignment to the north side of Birch Creek, costly bedrock excavation would likely be avoided. Locating the channel north of the existing marshes and lakes would also avoid wet conditions and improve the constructability; however, the route may still be impacted by flooding during construction if lake levels are high. The channel lengths, widths and estimated excavation volumes are summarized in Table 15.5. A profile of Option D showing existing ground elevation, channel invert, water surface and key infrastructure is provided on Figure 15.4.

Table 15.5
Option D Excavation Volume Estimates

Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
D1	5000	Till	22800	27	4:1	72	5.53	6.29
D2	10000			64		108		10.86
D3	15000			100		144		15.37

Figure 15.4
Profile of Lake Manitoba Outlet Channel Option D



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

15.3.1.5 Option E

The Option E outlet channel was considered as a short bypass of the FRWCS to add additional outlet capacity to Lake Manitoba. The channel alignment is similar to the beginning portion of

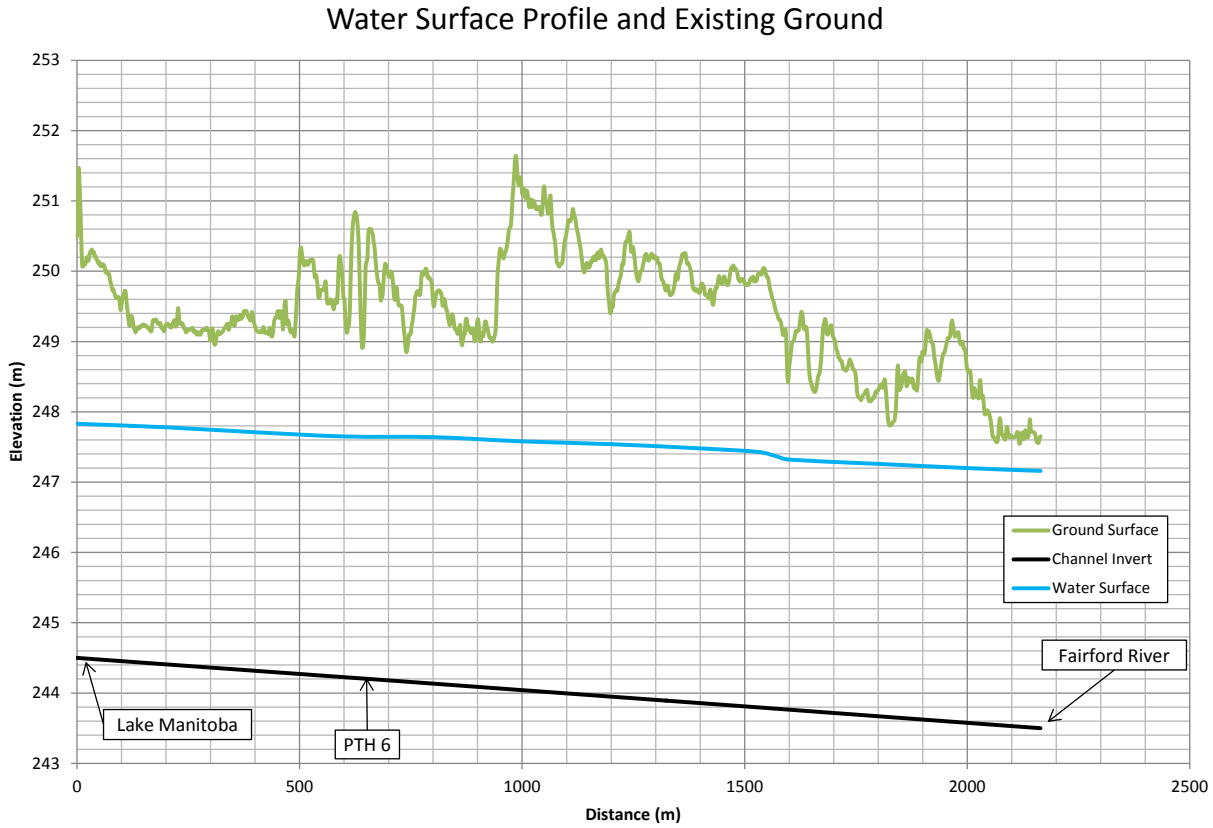
Option A, bypassing the FRWCS to the north and then merging with the Fairford River a short distance downstream of the existing outlet from Lake Manitoba as shown on Drawing 15.2 (located at the end of Section 15.0). However, unlike Option A, the increased capacity from this outlet will result in increases in the surface elevations in the downstream reaches of the Fairford River. As previously noted, two capacities were evaluated for this option 70 m^3/s (2,500 cfs; Option E1) and 106 m^3/s (3,750 cfs; Option E2). The capacity for this option is limited as it does not consider channel twinning for the full length of the Fairford River. The proposed route is approximately 2.2 km (1.4 mi) long and situated on privately held lands. This option would require construction of one new bridge and a control structure at PTH 6, with the bridge incorporated into construction of the new control structure (Drawing 15.2).

The excavation for Option E is assumed to be primarily in till. The bedrock surface elevation is assumed to lie below the level of the channel invert based on GWDrill data and previous geotechnical investigations. The channel lengths, widths and estimated excavation volumes are summarized in Table 15.6. A profile of Option E showing existing ground elevation, channel invert, water surface and key infrastructure is provided on Figure 15.5.

Table 15.6
Option E Excavation Volume Estimates

Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
E1	2500	Till	2150	9	4:1	54	5.4	0.37
E2	3750			21		65		0.50

Figure 15.5
Profile of Lake Manitoba Outlet Channel Option E



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

15.3.1.6 Option F

Option F is comprised of widening the Fairford River and FRWCS (including dredging of the inlet channel) as shown on Drawing 15.2 (located at the end of Section 15.0). A stipulation for the design criteria of Option F was to increase capacity without increasing the surface elevation in the river at any location. Widening of the Fairford River to increase the capacity would require removal of the abandoned railway bridge. Construction of two additional 5.84 m (3.6 mi) wide bays would be required at the FRWCS, and a new control structure / bridge would need to be constructed downstream of Lake Pineimuta to replace the existing municipal road bridge. The downstream control structure was required to maintain Lake Pineimuta levels. Without the second control structure, Lake Pineimuta levels would be reduced in an uncontrolled manner during periods in which it was not necessary to operate the channel.

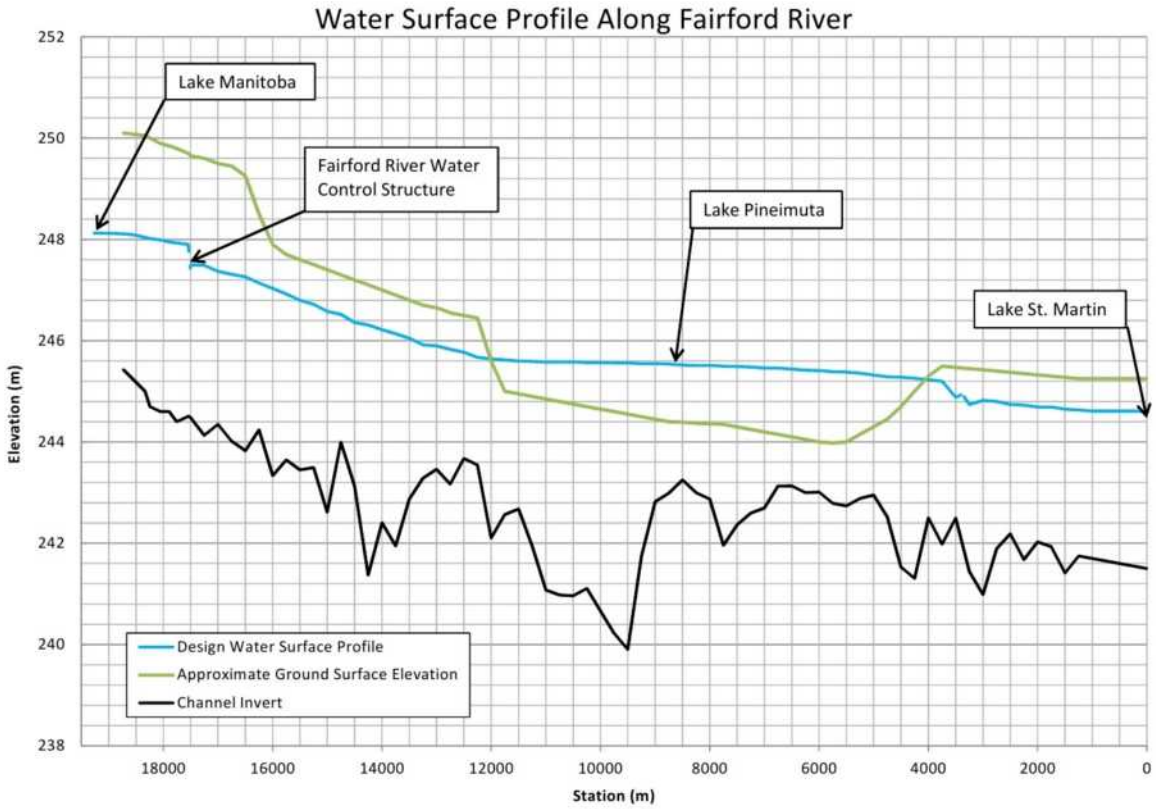
To facilitate construction, it has been assumed that Option F will require temporary fill for machinery access parallel to the river and staging areas to excavate the existing river channel to the invert. The fill volume required for construction along the river reaches would be removed as the below water excavation proceeded. There would also be an above water cut volume required to excavate the existing riverbank. The temporary fill, below water and the above water excavation material would be deposited in a spoil pile offset a safe distance from the top of the riverbank.

The excavation for Option F is assumed to be primarily in till. The bedrock surface elevation is assumed to lie below the level of the channel invert based on GWDrill data and previous geotechnical investigations. The channel lengths, widths and estimated excavation volumes are summarized in Table 15.7. A profile of Option F showing existing ground elevation, channel invert, water surface and key infrastructure is provided on Figure 15.6.

Table 15.7
Option F Excavation Volume Estimates

Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
F1	18500	Til	18700	95	4:1	n/a	3.0	4.88

Figure 15.6
Profile of Lake Manitoba Outlet Channel Option F



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

15.3.2 Lake St. Martin Outlet Channels

All water that exits Lake Manitoba ends up in Lake St. Martin. The only natural outlet from Lake St. Martin is the Dauphin River that flows into Sturgeon Bay (Lake Winnipeg). The development of the FRWCS (Manitoba, 1961) and associated channel works in 1961 resulted in the Fairford River having a greater capacity than the Dauphin River with Lake St. Martin at normal levels. As such, the Dauphin River is the critical restriction of flow between Lake Manitoba and Lake St. Martin, particularly during winter months due to frazil ice formations further reducing the capacity of the Dauphin River. In addition, the storage capacity of Lake St. Martin is significantly less than that of Lake Manitoba. As a result, Lake St. Martin often rises to a level above its flood stage to balance inflows and maximize natural outflows. The outlet capacity of Lake St. Martin must be increased to allow maximum outflows from Lake Manitoba through the FRWCS without

causing any additional flooding on Lake St. Martin compared to unregulated conditions (prior to the construction of the FRWCS).

The Lake St. Martin Emergency Outlet Channel (LSMEOC) was divided into three separate sections (Reach 1, Reach 2, and Reach 3) when it was originally developed (Drawing 15.4 located at the end of Section 15.0). As previously discussed, Reach 1 was constructed and operated from November 2011 through to November 2012. Reach 2 was not designed or constructed as it was determined in the early stages of the project that it was not required for the 2011 / 2012 flows. Reach 3 was only partially constructed as extremely mild winter conditions limited ice staging on Lake St. Martin and ultimately precluded the need for operation of Reach 3 in the spring of 2012. Since Reach 3 was not operated in 2011/2012, flows from operation of Reach 1 followed the entire length of Buffalo Creek and discharged into the lower Dauphin River. However, utilizing the lower portion of Buffalo Creek on a permanent basis is not recommended due to environmental impacts, increased potential for flooding of Dauphin River communities and opposition from stakeholders. Therefore, for the current study, it has been assumed that all Lake St. Martin outlet channels would include Reach 3 and that only riparian flow would be permitted into Buffalo Creek.

During construction, the Dauphin River First Nation community noted a preference to have the Reach 3 outlet located on the east side of Willow point rather than the designed alignment, which ended at Johnson Beach. While this was not feasible during the time restrictions in 2011, the east of Willow Point option was re-examined as a part of this Stage 1 study. The Willow Point route was also recommended by the recent Lake Manitoba / Lake St. Martin Regulation Review based on feedback received from First Nations.

The Lake St. Martin outlet channel options being evaluated include a combination of the following reaches:

- Reach 1 (R1) – Re-opening and/or expansion of Reach 1.
- Reach 2 (R2) – Expansion of Buffalo Creek (Reach 2) downstream of Buffalo Lake.
- Reach 3 (R3-JB) – Completion and/or expansion of Reach 3 to Lake Winnipeg at Johnson Beach.
- Reach 3 (R3-WP) – Completion and/or expansion of Reach 3 to Lake Winnipeg east of Willow Point.

Note that unlike the Lake Manitoba outlet channel options, R1, R2 and R3 are separate reaches downstream of each other; and combinations of the reaches are required to form a complete Lake St. Martin outlet channel system (i.e., they cannot be implemented independently). Therefore, since only one route was considered for both Reach 1 and Reach 2, the two possible Lake St. Martin outlet channel options being evaluated include the following:

- Option JB – Reach 1, Reach 2 and Reach 3 to Johnson Beach.
- Option WP – Reach 1, Reach 2 and Reach 3 east of Willow Point.

Reach 1 had originally been designed in 2011 for a discharge of approximately 140 m^3/s (5,000 cfs) at a Lake St. Martin elevation of 244.1 m (801 ft); however, due to construction delays, the channel was constructed to approximately 75% of the design flow, and currently has a capacity of approximately 113 m^3/s (4,000 cfs). The original design capacity for Reach 3 was 120 m^3/s (4,250 cfs). However, as the role of the Reach 3 channel was to divert the added flows in Buffalo Creek from Reach 1 away from the lower Dauphin River, it was required to support flow capacities equivalent to Reach 1. Likewise, to convey flows from Reach 1, the capacity of Reach 2 also needed to be equivalent to Reach 1. As such, the existing (base) capacity for Reach 1, 2 and 3 was listed as 113 m^3/s (4,000 cfs).

As outlined in Section 15.1, the Lake Manitoba and Lake St. Martin Regulation Review Committee, and the Manitoba 2011 Flood Review Task Force recommended that the Lake St. Martin Emergency Channel be made permanent. This recommendation was adopted; therefore, the existing capacity for Reach 1, 2 and 3 of 113 m^3/s (4,000 cfs) formed the base condition for this study. Furthermore, as also outlined in Section 15.1 and recommended in the report "Analysis of Options for Emergency Reduction of Lake Manitoba and Lake St. Martin Levels" (KGS Group and AECOM, 2011), the governing principal for the design of the Lake Manitoba and Lake St. Martin channel was that there should be no net increase in flow to Lake St. Martin above the base condition of the existing Reach 1 channel. On this basis, the current assessment included evaluation of increased capacities, in addition to the base capacity of 113 m^3/s (4,000 cfs), to account for increased flows from the Lake Manitoba outlet channel options of 140 m^3/s (5,000 cfs), 283 m^3/s (10,000 cfs) and 424 m^3/s (15,000 cfs). This would result in additional Lake St. Martin outlet channel alternatives with capacities of 255 m^3/s (9,000 cfs), 396 m^3/s (14,000 cfs), and 538 m^3/s (19,000 cfs), respectively, at a Lake St. Martin elevation of 244.1 m (801 ft).

Alternatives to handle the additional capacities corresponding to the Lake Manitoba channel Option E (increased capacity by 70 m^3/s (2,500 cfs) or 106 m^3/s (3,750 cfs)) were not assessed because, on their own, they would not provide the flood protection required to meet the goals of the project.

The outlet channels were designed as a trapezoidal cross section with near vertical 1:4 side slopes in bedrock (Reach 3 only) and 3:1 side slopes in overburden (till). The Lake St. Martin outlet channels were designed with 3:1 side slopes, to match the conditions of the existing constructed channels rather than the 4:1 side slopes used to develop the Lake Manitoba options. Future detailed design could consider 4:1 side slopes to provide more acceptable conditions for revegetation of the channel. The channel roughness used was 0.028 in till and 0.032 in bedrock. The average flow velocity, channel slope, depth of flow, base width and average top width varied depending on the Reach, material and design flow, as described in the following sections. Detailed information for each option is provided in the sections that follow and is also summarized in Table 15.1 (located at the end of Section 15.0).

During consideration of emergency channel measures in 2011, the flow through the Buffalo Lake Bog was considered to be a potential problem resulting from organic material dislodging and floating to the surface thus restricting downstream flow passage. Downstream of the bog where Buffalo Creek begins, the available cross section area was also a concern in that it would not have the capacity to discharge the flow from the Reach 1 channel, which would result in a backwater effect in Reach 1. Additional field surveys conducted in 2011 and, from experience gained during the operation of the Reach 1 channel in 2011 / 2012, aided in determining that these concerns were not a problem for the limited flow capacity of Reach 1 and the corresponding relatively low discharges that occurred. As a result, it has been concluded that Reach 2 channel excavation would not be required for lower design discharge similar to the existing capacity of the Reach 1 channel.

The need for channel improvement for Reach 2 was reexamined for the enlarged channel alternative being examined in this study. Detailed backwater model runs with design discharge greater than approximately 280 m^3/s (10,000 cfs) resulted in water levels in the bog that would result in water escaping from the bog and overflowing to the Dauphin River. The corresponding

high levels would also result in backwater to Reach 1, which would reduce flows in Reach 1 and require increased excavation to offset the backwater effects.

Excavation volumes were generally estimated using the design channel geometry, the ground surface based on Shuttle Radar Topography Mission (SRTM) data and LiDAR data, and the bedrock surface based on well drilling data base (GWDrill), geophysical exploration and test-hole data.

The following sections provide a description of the various reaches of a Lake St. Martin outlet channel together with the associated excavation volumes. The predicted hydraulic impact of the outlet channels for the various capacities is provided in Section 15.4. Cost estimates for each of the options are provided in Section 15.5.

15.3.2.1 Reach 1

Reach 1 of the Lake St. Martin outlet channel connects Lake St. Martin to the bog complex surrounding Big Buffalo Lake and is approximately 6 km (3.7 mi) in length as shown in Drawing 15.4. The route is located entirely on crown land.

When the LSMEOC was initially opened, an earth plug was removed at the inlet, which was replaced with an earth fill cofferdam when the channel was subsequently closed. This method of operation is not recommended for a permanent channel, as it is not practical in the long-term. As well, it has environmental concerns with respect to erosion and sedimentation. As such, two concepts, an overflow weir and a control structure, were considered for a permanent inlet to Reach 1.

An overflow weir would operate in an uncontrolled manner as lake levels fluctuate. Therefore, it would have to be constructed at an elevation near the upper limit of the desirable operating range for Lake St. Martin because a lower weir elevation would draw down Lake St. Martin excessively during dry periods. To achieve the required discharge capacity, the overflow weir would have to be significantly wider than the outlet channel. In comparison, a control structure could be operated strategically in response to actual conditions, which would make the outlet channel more efficient, particularly at lower lake levels. Making inlet improvements and adding a

control structure would also increase the existing outlet channel capacity. A control structure could also possibly be strategically operated to help deal with ice formation issues within the outlet channel and the Dauphin River.

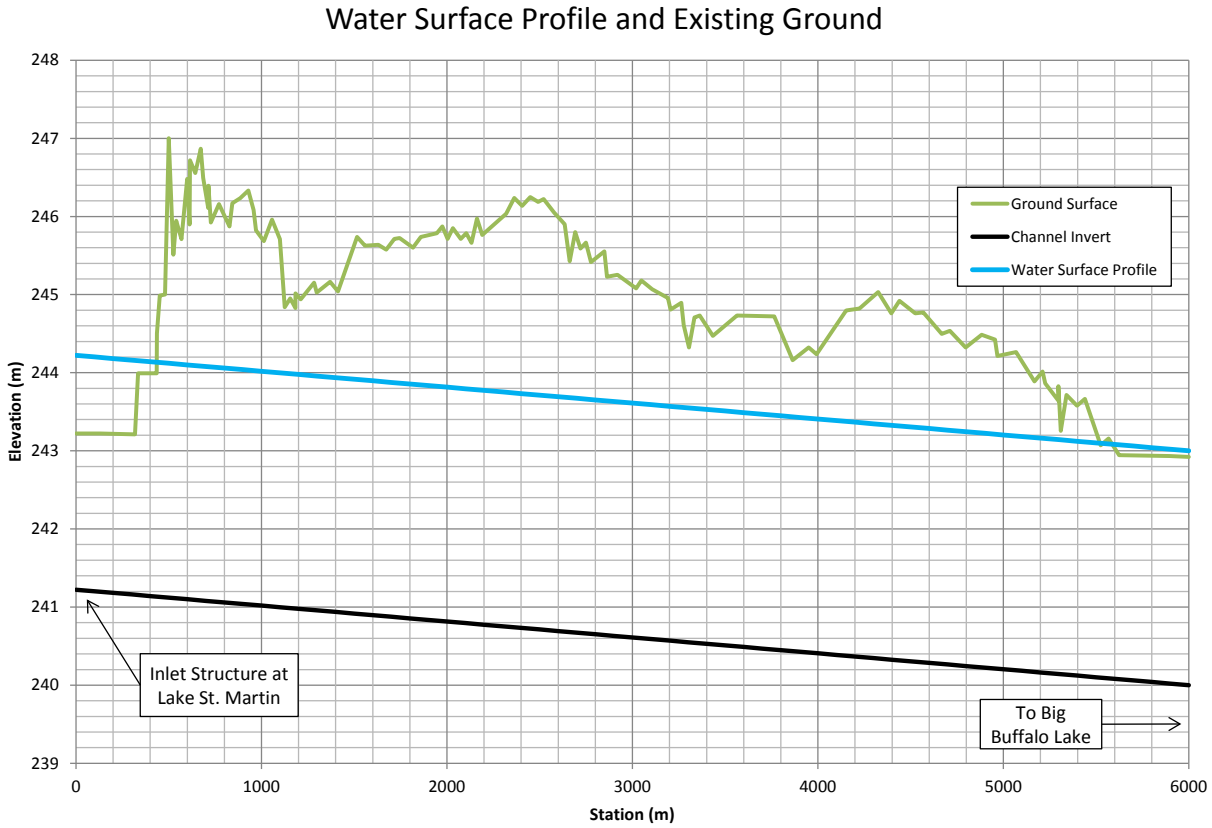
Based on the above, all Lake St. Martin outlet channel design capacities considered in this report incorporate an inlet control structure at Reach 1, rather than an overflow weir. The control structure is envisaged to be a simple sluiceway structure with stoplogs similar to the FRWCS. It would have a horizontal apron slab at an elevation of 241.0 m (790.7 ft) with no overflow crest (no elevation drop downstream). The structure would have piers spaced at approximately 6 m (20 ft), and a bridge deck complete with hoisting structure from which the stoplogs could be removed manually when required to release flow into Reach 1 of the outlet channel. The bridge deck would also be used as a crossing for maintenance access to the other side of the outlet channel.

The excavation for Reach 1 was assumed to be primarily in till. The design was based on a channel slope of 0.020% with an average depth of flow of 2.8 m (9.2 ft) and a design average flow velocity of 0.9 m/s (3.0 ft/s). The channel lengths, widths and estimated excavation volumes are summarized in Table 15.8. A profile of Reach 1 showing existing ground elevation, channel invert and water surface is provided on Figure 15.7.

Table 15.8
Lake St. Martin Outlet Channel – Reach 1 Excavation Volume Estimates

Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
R1 Base	4000	Til	6000	45	3:1	70	4:2	0.55
R1+5000	9000			81		106		1.46
R1+10000	14000			128		153		2.68
R1+15000	19000			176		201		3.90

Figure 15.7
Profile of Lake St. Martin Outlet Channel Reach 1



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

15.3.2.2 Reach 2

Reach 2 of the Lake St. Martin outlet channel is located along Buffalo Creek beginning at the edge of the bog complex surrounding Big Buffalo Lake and is approximately 2.9 km (1.8 mi) in length as shown on Drawing 15.4. The route is located entirely on crown land.

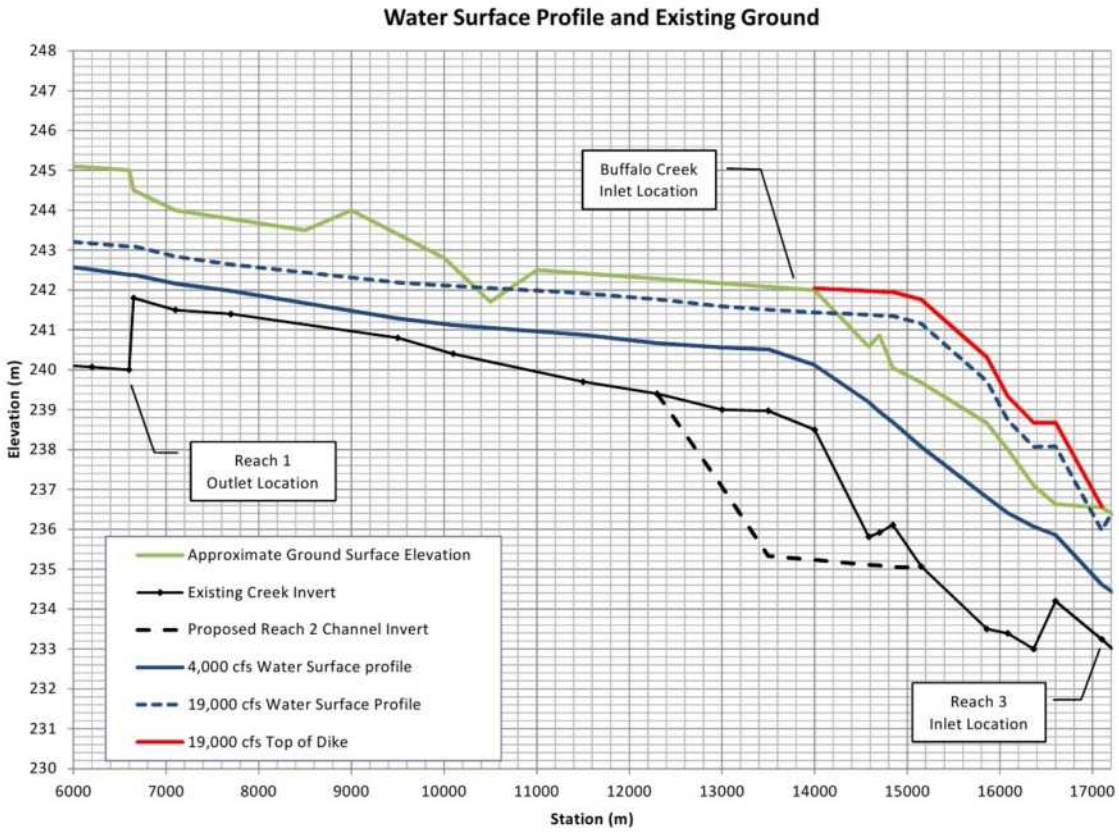
Reach 2 involves the widening and deepening of a natural segment of the upstream end of Buffalo Creek to increase the capacity and ensure water levels on Buffalo Lake are contained by the surrounding topography and to reduce backwater effects to the Reach 1 channel. The concern was that the water could potentially spillover from Buffalo Lake and travel overland towards the Dauphin River and lower portions of Buffalo Creek in an uncontrolled manner. Reach 2 was also required to ensure water levels on Buffalo Lake at the exit of Reach 1 were low enough not to reduce the capacity of Reach 1 due to backwater effects.

The excavation for Reach 2 was assumed to be primarily in till. Some of the excavation material will be used to construct containment dikes along the creek, and the rest will be deposited in spoil piles. The channel was designed with a slope of 0.013% with an average depth of flow of 7 m (23 ft) and a design average flow velocity of 0.5 m/s (1.6 ft/s). The channel lengths, widths and estimated excavation volumes are summarized in Table 15.9. A profile of Reach 2 showing existing ground elevation, channel invert, dike elevations and the water surface is provided on Figure 15.8.

Table 15.9
Lake St. Martin Outlet Channel – Reach 2 Excavation Volume Estimates

Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
R2 Base	4000	Til	2900	0	3:1	n/a	4:2	0.00
R2+5000	9000			10		35		0.10
R2+10000	14000			80		105		0.75
R2+15000	19000			140		165		1.30

Figure 15.8
Profile of Lake St. Martin Outlet Channel Reach 2



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

15.3.2.3 Reach 3

Reach 3 of the Lake St. Martin outlet channel would divert flood flows from Buffalo Creek directly to Lake Winnipeg, rather than following the natural drainage path along lower Buffalo Creek to the Dauphin River and then to Lake Winnipeg as shown in Drawing 15.4. An earth dam structure would be constructed across Buffalo Creek to divert the flood flows into Reach 3. The earth dam would include culverts so that riparian flow in Buffalo Creek due to local runoff would be allowed to remain in the creek and continue downstream of Reach 3.

In addition to evaluating different capacities for Reach 3, the evaluation considered two alternative locations where Reach 3 outlets to Lake Winnipeg including the Johnson Beach location and the east side of Willow Point location. The total length of Reach 3 is approximately 9.7 km (6.0 mi) for the Johnson Beach outlet location and approximately 11.4 km (7.1 mi) for the

Willow Point outlet. Both routes are located entirely on crown land and both currently only have winter road access. Aside from at the proposed new Reach 1 inlet control structure, bridge crossings would not be necessary as there are no existing permanent roads in this area.

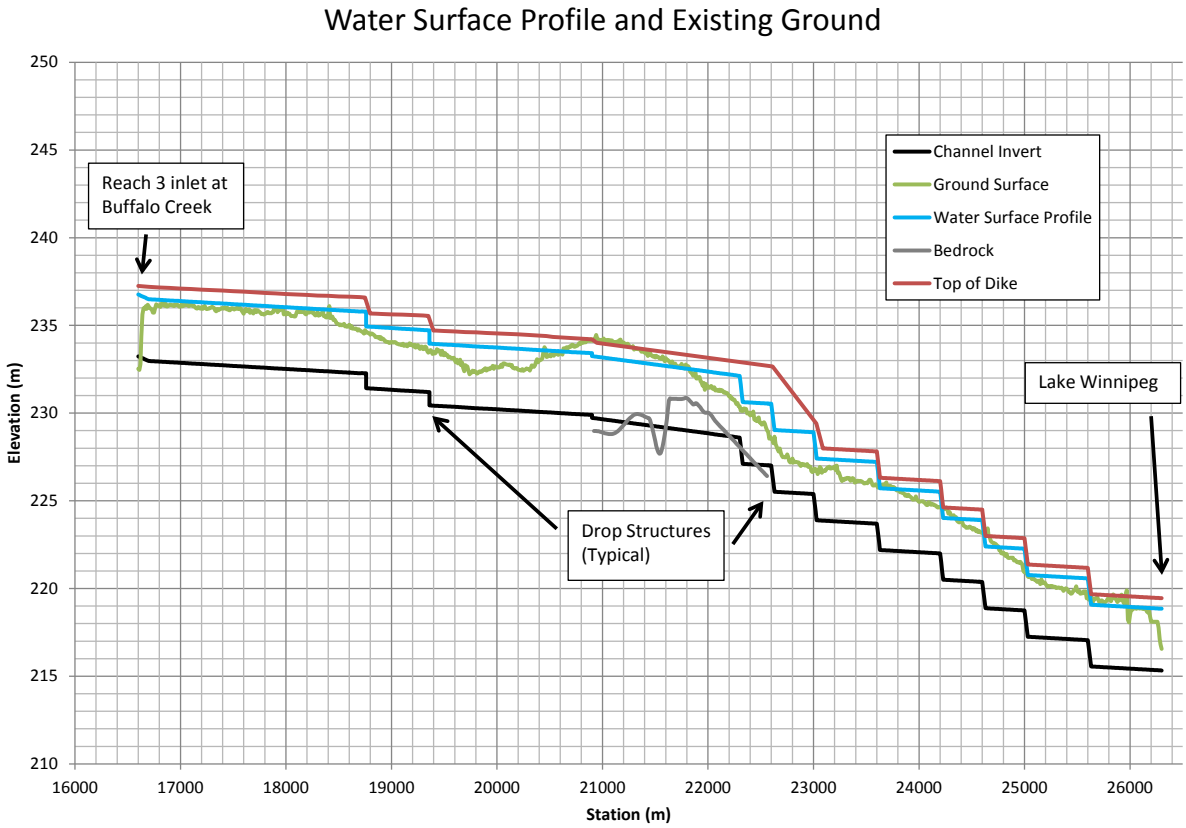
To reduce the gradient and flow velocities in the channel, multiple rock fill drop structures would be required. As shown in Drawing 15.4, the Johnson Beach outlet option requires 11 drop structures and the Willow Point outlet option requires 12 drop structures. For both outlet options, three of the drop structures would be located within the portion of the Reach 3 channel that was partially constructed in 2012.

The excavation for Reach 3 is primarily in till with a portion in bedrock. Some of the excavation material will be used to construct containment dikes along the creek, and the remainder will be deposited in spoil piles. The design channel slope is 0.034 % in till and 0.074% in bedrock. The average depth of flow is 3.5 m (11.5 ft) in till and 2.0 m (6.6 ft) in bedrock with average flow velocities of 1.1 m/s (3.6 ft/s) and 1.6 m/s (5.2 ft/s), respectively. The channel lengths, widths and estimated excavation volumes are summarized in Table 15.10. Profiles of the two Reach 3 options showing existing ground elevation, channel invert and water surface are provided on Figures 15.9 and 15.10.

Table 15.10
Lake St. Martin Outlet Channel – Reach 3 Excavation Volume Estimates

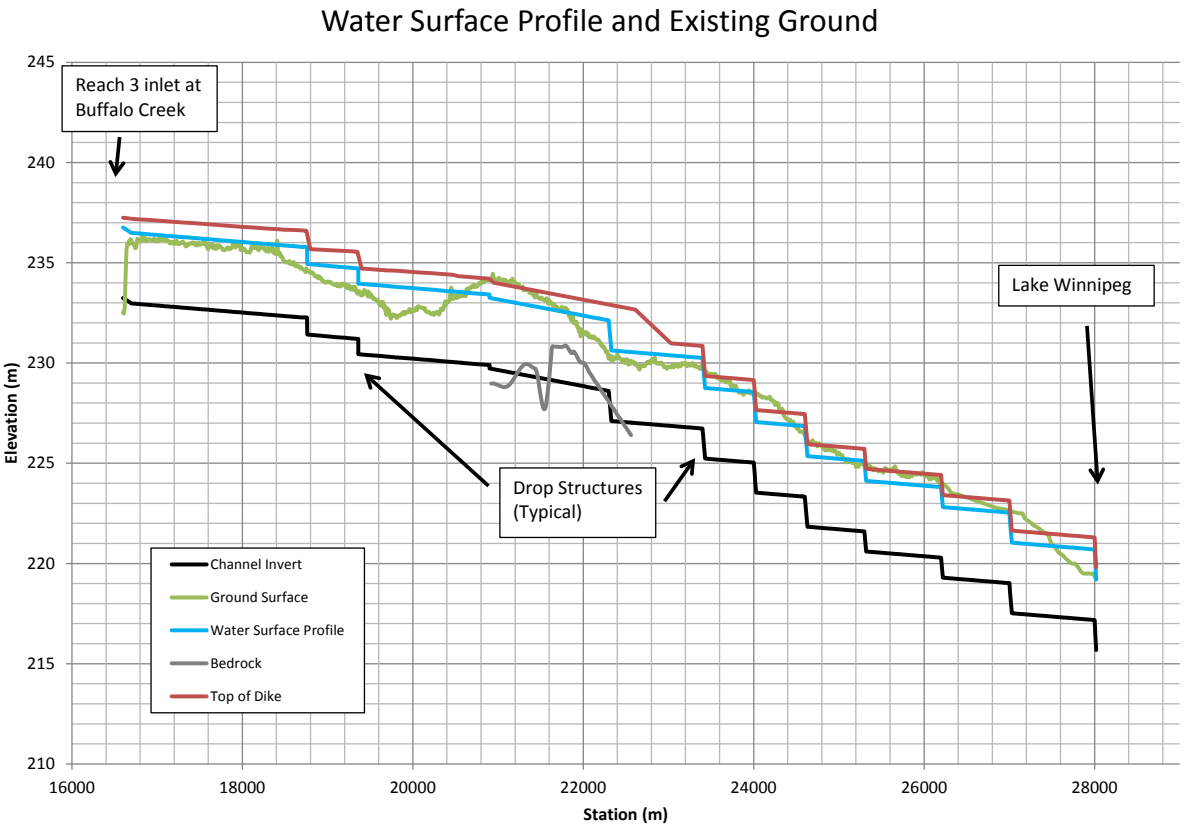
Design Option	Design Flow (cfs)	Material	Channel Length (m)	Base Width (m)	Channel Side Slopes	Average Top Width (m)	Average Depth of Excavation (m)	Overall Excavation Quantity (mil.cu.m)
R(JB) Base	4000	Till	8040	21	3:1	39	3.0	0.36
		Rock	1660		1:4			0.00
R3(JB) +5000	9000	Till	8040	52	3:1	70	3.0	1.47
		Rock	1660		1:4			0.03
R3(JB) +10000	14000	Till	8040	82	3:1	101	3.0	2.35
		Rock	1660		1:4			0.06
R3(JB) +15000	19000	Till	8040	113	3:1	131	3.0	3.22
		Rock	1660		1:4			0.09
R3(WP) Base	4000	Till	9760	21	3:1	41	3.4	0.69
		Rock	1660		1:4			0.00
R3(WP) +5000	9000	Till	9760	52	3:1	72	3.4	2.11
		Rock	1660		1:4			0.03
R3(WP) +10000	14000	Till	9760	82	3:1	103	3.4	3.27
		Rock	1660		1:4			0.07
R3(WP) +15000	19000	Till	9760	113	3:1	133	3.4	4.43
		Rock	1660		1:4			0.08

Figure 15.9
Profile of Lake St. Martin Outlet Channel Reach 3 to Johnson Beach



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

Figure 15.10
Profile of Lake St. Martin Outlet Channel Reach 3 to East of Willow Point



Note: Elevations can be converted to metres by multiplying imperial values by 0.3048.

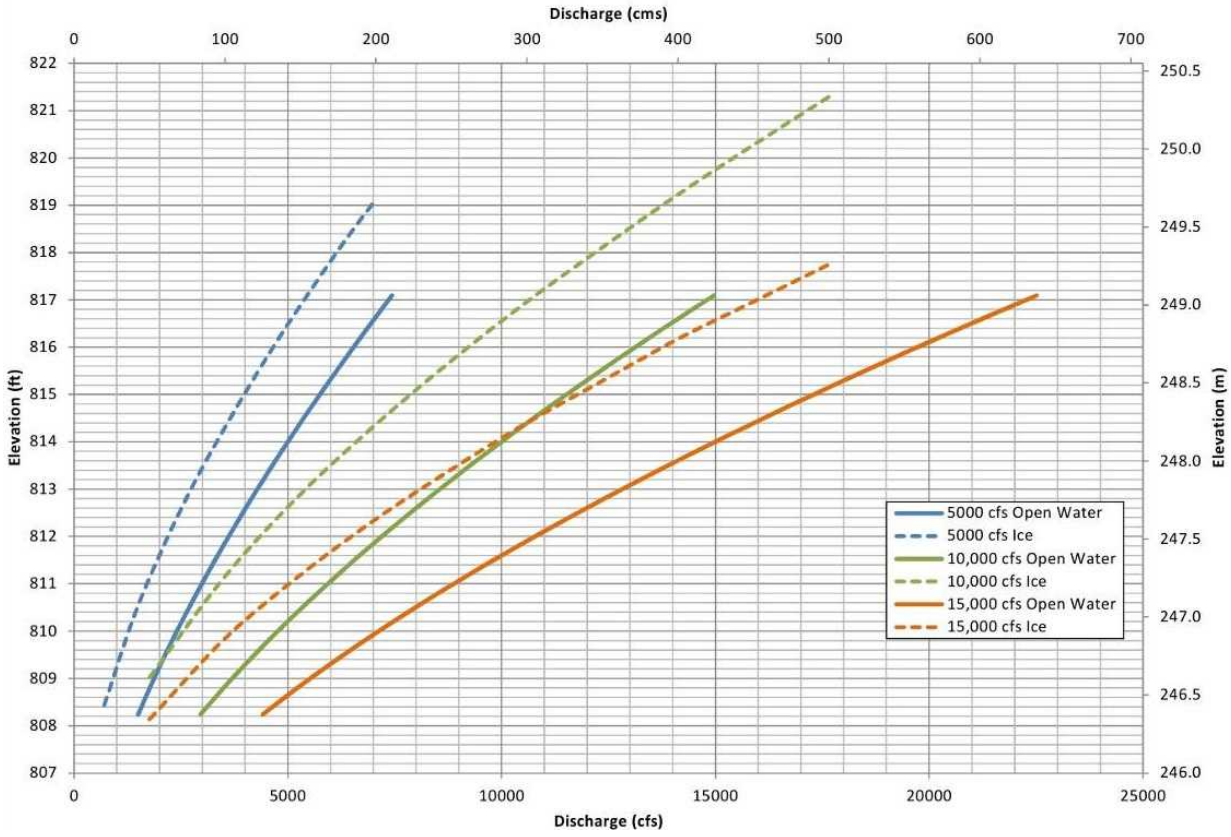
15.4 EFFECTS OF CHANNELS ON WATER LEVELS

15.4.1 Flood Routing

The flood routing model for Lake Manitoba and Lake St. Martin discussed in Section 8.2.2 formed the basis for computing the effects of the outlet channels on the lake levels. Stage-discharge relationships for the Lake Manitoba and Lake St. Martin outlet channels were developed and incorporated into the routing model. The stage-discharge relationships for the Lake Manitoba Outlet channels are provided on Figure 15.11 and were developed based on the results from backwater models of the channels. Only one representative stage-discharge curve was used for the different outlet channel options. The justification for this simplification is that the basis of design for all channels required that the design discharge be provided at a Lake Manitoba elevation of 814 ft (248.1 m), which resulted in similar rating curves for all channels. It

was recognized that the stage-discharge curves could vary slightly depending on the route option selected due to differences in channel geometry and conditions at the channel inlet. Nevertheless, for this concept phase study, the curves shown on Figure 15.11 were considered representative of the increase in outflow from Lake Manitoba for all of the alternatives. It should be noted that at the elevation of 814 ft (248.1 m) on Lake Manitoba, the FRWCS already has a capacity of about 380 m^3/s (13,400 cfs).

Figure 15.11
Lake Manitoba Outlet Channel Stage-Discharge Relationships



The stage-discharge relationship for the Lake St. Martin Outlet Channel, which has a design flow of 113 m^3/s (4,000 cfs), is provided on Figure 15.12. Its derivation was based on the hydrometric data collected on Reach 1 during operation of the Lake St. Martin Emergency Outlet Channel in 2011 and 2012. For larger channels, the stage-discharge curves were based on the results of backwater models of the channels and are provided on Figure 15.13.

Figure 15.12
Lake St. Martin 4,000 cfs Outlet Channel Stage-Discharge Relationship

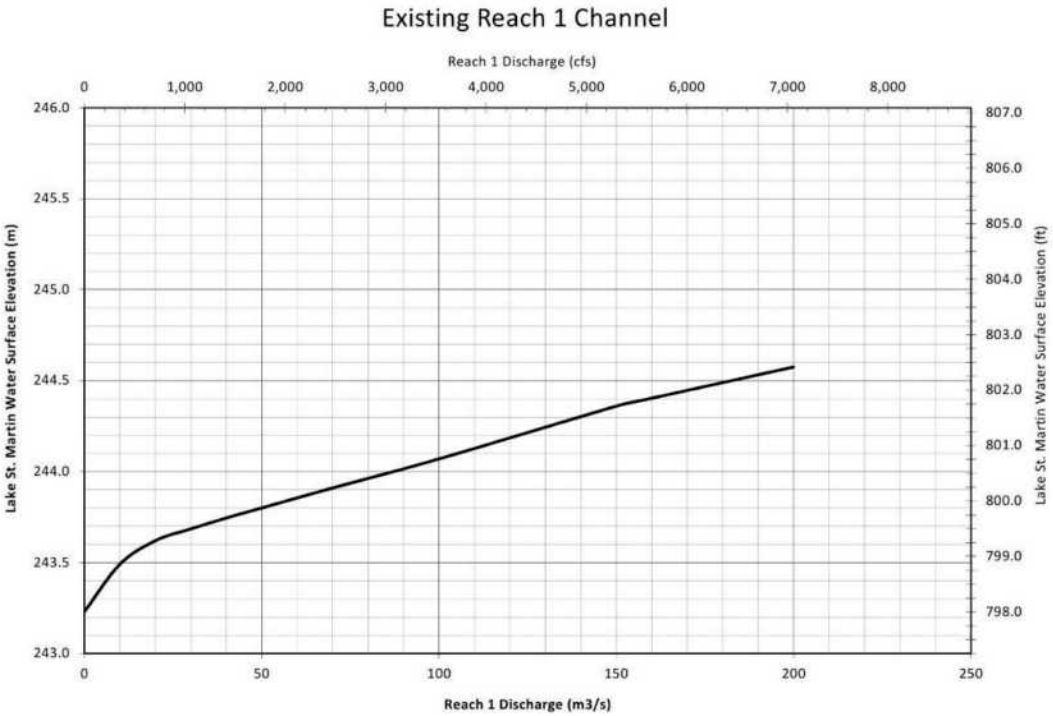
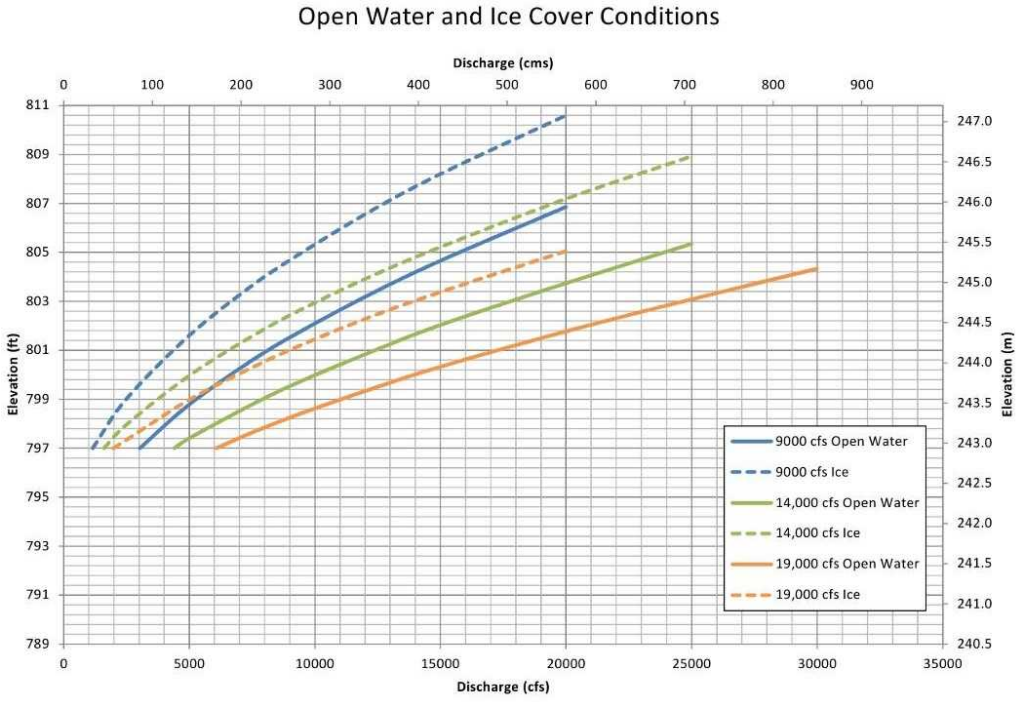


Figure 15.13
Lake St. Martin Outlet Channels Stage-Discharge Relationships



As depicted by the stage-discharge curves for the different outlet channel alternatives, the formation of ice cover in the channels can reduce the outflow capacity from lakes Manitoba and St. Martin. Based on experience from the 2011 Flood, it was assumed as the base condition for the study that ice would not form in the Lake Manitoba and Lake St. Martin outlet channels. However, an analysis was also conducted assuming a worse case ice cover would form early in the winter with a maximum ice cover thickness of 0.6 m (2 ft). A sensitivity analysis was conducted to determine the effects to the computed lake levels of a change to the ice conditions in the model and the results are documented in Section 15.4.2.

As discussed in Section 8.2.2.3, a starting water level of El 247.65 m (El 812.5 ft) was assumed for Lake Manitoba and El 244.15 m (El 801 ft) was assumed for Lake St. Martin for the baseline conditions without the outlet channels. By selecting those water levels, it was assumed that the two lakes were already at the upper limits of their operating ranges prior to the flood. With the outlet channels in place, a starting water level of El 247.5 m (El 812.0 ft) was assumed for Lake Manitoba and 243.84 m (800 ft) was assumed for Lake St. Martin. In this case, it was assumed that the outlet channels were effective in maintaining the lake levels below the upper limit of the operating range before the start of the flood. A sensitivity analysis was conducted to determine the effects to the computed lake levels of a change to the starting water level and has been documented in Section 8.2.2.4.

Each combination of Portage Diversion inflow hydrographs and local Lake Manitoba IAO hydrographs discussed in Section 8.2.2.2 were routed numerically through the model of Lake Manitoba and Lake St. Martin. Resultant hydrographs for the two lakes for a scenario, which results in a peak water level equivalent to a flood event with an approximate return period of 1:200 years, have been provided in Figures 15.14 and 15.15.

Figure 15.14
Computed 1:200 Year Lake Manitoba Water Level Hydrographs with an Outlet Channel

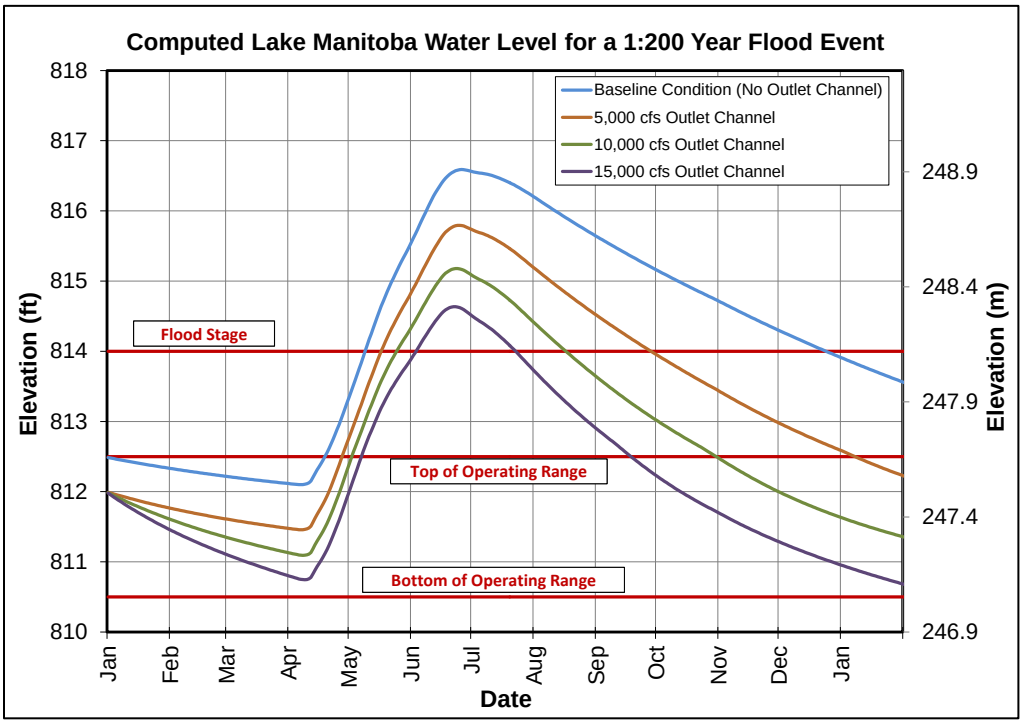
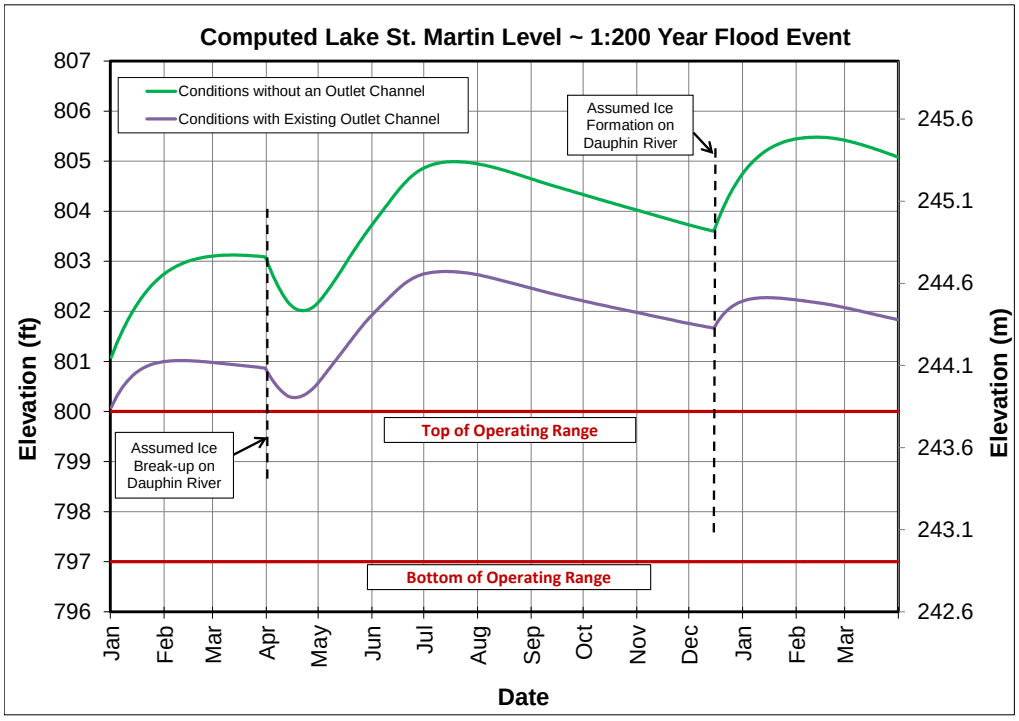
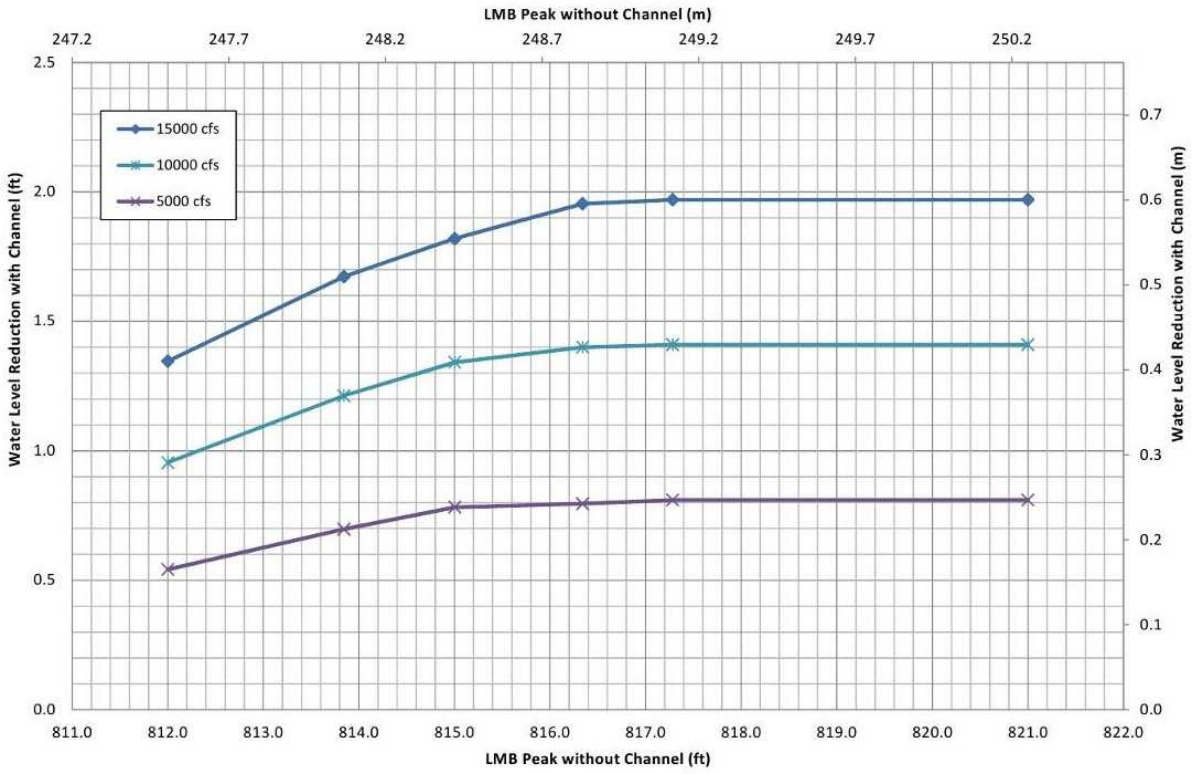


Figure 15.15
Computed 1:200 Year Lake St. Martin Water Level Hydrographs with an Outlet Channel



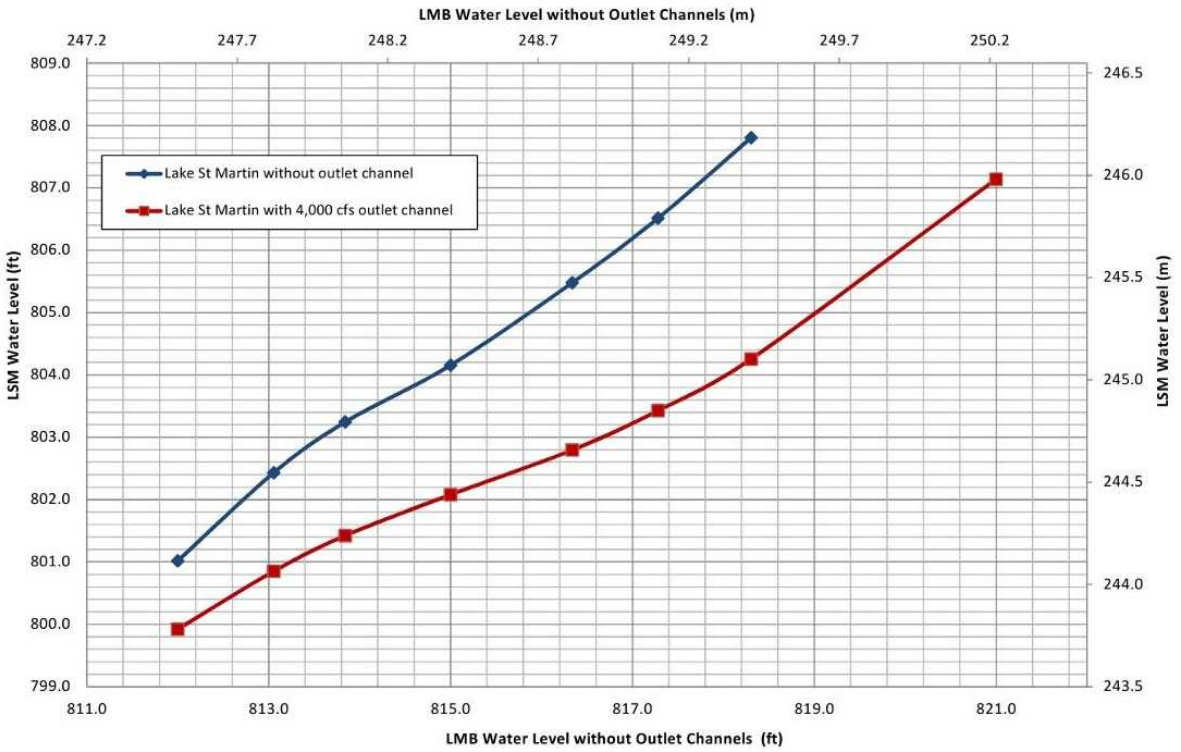
Based on the results of the analysis, a relationship was developed to determine the reduction in water levels on Lake Manitoba due to operation of an outlet channel as shown on Figure 15.16 By interpolating between lines on the figure, the reduction in Lake Manitoba water levels can be obtained for any size of outlet channel, and for any peak water level for the base case condition.

Figure 15.16
Reduction in Water Levels on Lake Manitoba due to Operation of an Outlet Channel



A relationship was developed between Lake Manitoba and Lake St. Martin to determine the peak water levels on Lake St. Martin with and without the base case for the permanent Reach 1 channel with a design flow of 113 m^3/s (4,000 cfs). This is shown on Figure 15.7. Without the Reach 1 channel, the peak water level on Lake St. Martin was computed to occur in the summer for small flood events. However, for large floods, the peak water level was computed to occur in the winter due to discharge restrictions caused by ice in the Dauphin River. By contrast, with a permanent Reach 1 channel in operation, the peak water level on Lake St. Martin was computed to always occur in the summer, regardless of whether a discharge restriction due to ice was assumed in Reach 1. A sensitivity analysis of assumed ice conditions is provided in Section 15.4.2.

Figure 15.17
Relationship between Peak Water Levels on Lake Manitoba and Lake St. Martin



As discussed in Section 15.3.2, the governing principal for the design of the Lake Manitoba / Lake St. Martin channel was that there should be no net increase in flow to Lake St. Martin above the base condition of the existing Reach 1 channel. All Lake St. Martin outlet channel alternatives, therefore, assumed the design flows were in addition to the outflow capacity of the existing Reach 1 of 113 m^3/s (4,000 cfs). For this reason, the relationship developed with a permanent Reach 1 channel was also used for the other Lake St. Martin outlet channel alternatives. It was recognised that due to the routing effects through Lake St. Martin, and the reduced peak water level on the lakes from operating the outlet channels, the actual relationship between Lake Manitoba and Lake St. Martin would vary between alternatives; however, at this concept phase of study, the computed relationship for Reach 1, which has a design flow of 113 m^3/s (4,000 cfs) was considered representative.

15.4.2 Sensitivity to Ice Effects

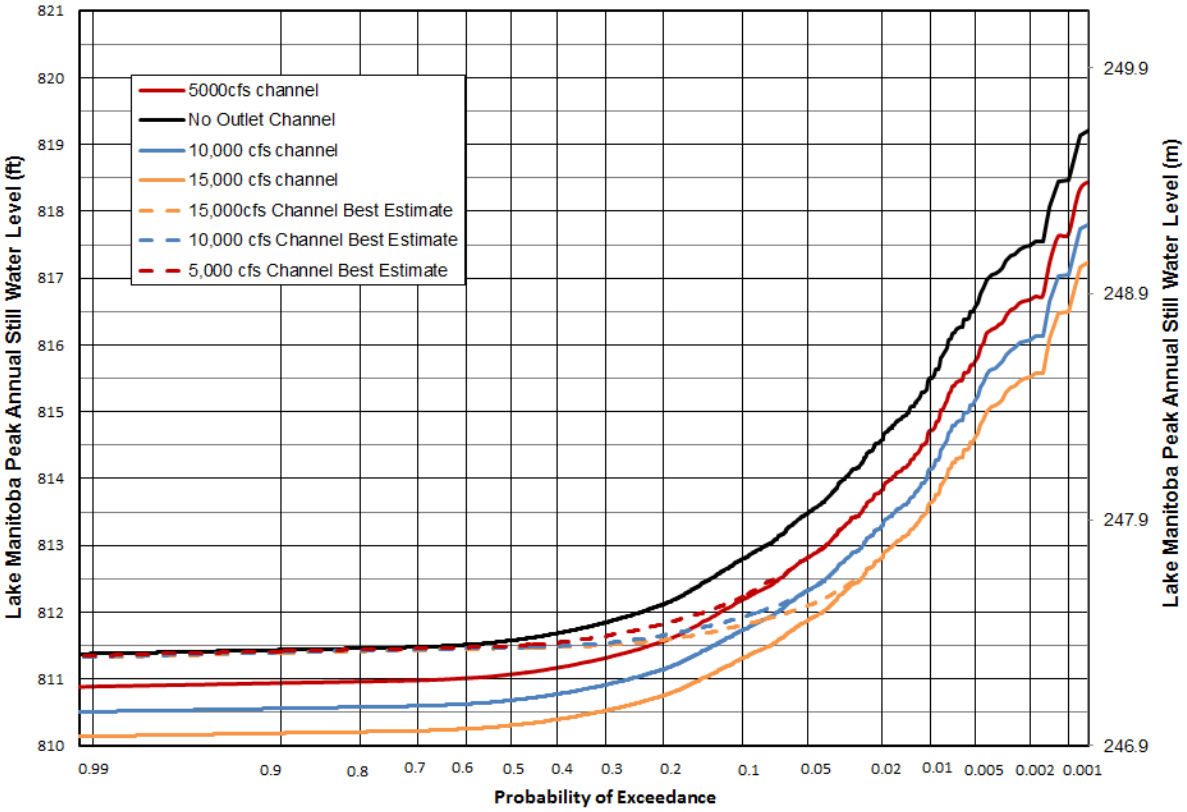
As discussed in Section 15.4.1, it was assumed for the base conditions that ice would not form in the outlet channels. This assumption was based on experience from the 2011 Flood. The effects of an ice cover on the Lake Manitoba and Lake St Martin outlet channels to peak water levels was determined using a stage-discharge relationship representing the best estimate of the worst case ice conditions. The stage-discharge relationships of the different channel alternatives were provided in Section 15.4.1. Discharge restrictions due to ice were assumed to occur between January 1 and March 31.

For Lake Manitoba, if ice were assumed to form on the outlet channel, this was computed to cause an increase of approximately 0.03 m (0.1 ft) to the peak water level of the lake. Conversely, on Lake St. Martin, assumption of formation of ice on the outlet channel was shown to cause an increase of approximately 0.3 to 0.6 m (1 to 2 ft) to the peak winter water level of the lake. However, the computed peak winter level did not exceed the computed peak summer level. As discussed in Section 15.4.1, with a permanent Lake St. Martin Outlet Channel, the peak water level on Lake St. Martin was computed to always occur in the summer.

15.4.3 Lake Manitoba Frequency Curve with Outlet Channels

Sections 8.51 and 8.52 described the methods and results of the “Monte Carlo” analyses for Lake Manitoba that formed the basis of computing frequency curves for the still water level and (SWL) and water levels that combine the effects of wind (CWL). The frequency curve of the Lake Manitoba SWL with the different outlet channel alternatives was determined by adjusting the stochastic SWLs discussed in Section 8.5.1 based on the relationships developed between peak Lake Manitoba water levels and reduction in water levels due to operation of an outlet channel provided in Figure 15.16. The resultant frequency curves are shown on Figure 15.18.

Figure 15.18
Frequency Curves of Still Water Level on Lake Manitoba with an Outlet Channel

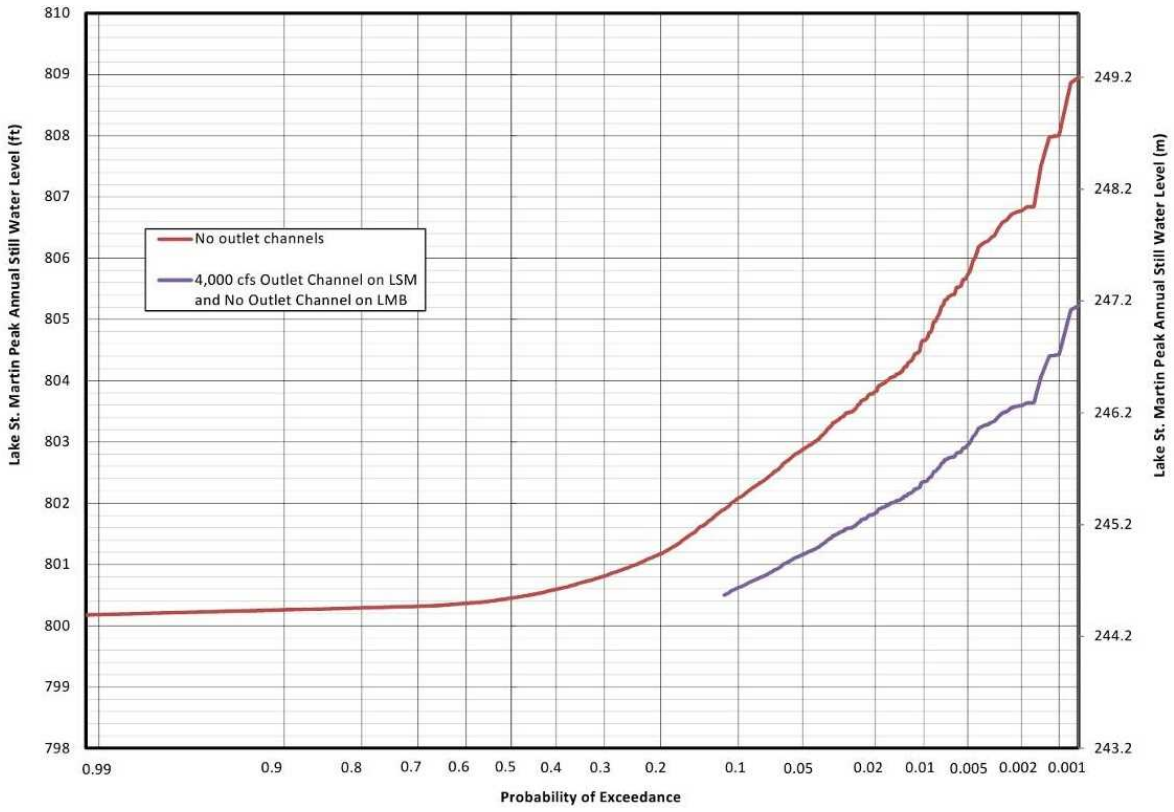


Since the flood routing scenarios from which the relationships were developed do not consider changes in gate operation at the FRWCS or closure of the Lake Manitoba outlet channel when Lake Manitoba levels are within the normal operating range of El 247.04 m and El 247.65 m (El 810.5 ft to El 812.5 ft), the frequency curves for high frequency events (i.e., events where the peak water level is below El 247.65 m or El 812.5 ft) are conservatively high. In all likelihood, the FRWCS and Lake Manitoba outlet channel would be operated to control the lake level within its normal operating range during such routine flood events. A best estimate of what the actual frequency curve at the lower range would be has been included on Figure 15.18. A sensitivity analysis of a change to the frequency curve at high frequencies to the results of the benefit / cost analysis has been included in the section of this report that describes the economic analysis.

The frequency curves of Lake St. Martin SWL, with and without a 113 m^3/s (4,000 cfs) Reach 1 outlet channel were determined using the frequency curve of Lake Manitoba SWLs and the

relationships developed between Lake Manitoba peak water levels and Lake St. Martin peak water levels provided in Figure 15.17. The resultant frequency curves are provided in Figure 15.19.

Figure 15.19
Frequency Curve of Still Water Levels for Lake St. Martin



15.5 COST ESTIMATION

An integral element in selecting a preferred alternative is a definition of the project components and the subsequent funding that would be required to develop each channel option. The following section describes the basis for the costing and provides a breakdown of the major items in the cost estimate for each of the proposed outlet channel options for Lake Manitoba and Lake St. Martin. The project requires a combination of one Lake Manitoba option and one Lake St. Martin option (Reach 1, Reach 2, and Reach 3) to effectively mitigate flooding on the two lakes. However, at this time, summary costs have not been provided for combinations of the more probable channel options.

15.5.1 Basis of Estimates

The cost estimates proposed in the following section are based, in part, on the preliminary estimates included in the report, "*Analysis of Options for Emergency Reduction of Lake Manitoba and Lake St. Martin Levels"* (KGS Group and AECOM, 2011). The cost estimates include many of the same components previously defined in the Analysis of Options report including control structures, bridges, excavation volumes and land acquisition. The standard widths of control structures as well as PTH and municipal bridges were assumed to be the same as those defined in the Analysis of Options report (14 m, 12 m, and 10.8 m, (46 ft, 39 ft and 35 ft), respectively). However, the dimensions of the channels and corresponding excavation volumes and structure lengths have been substantially updated in this report based on a more advanced hydraulic assessment of each option. Additional project components included in the current cost estimate table that were not previously examined in detail include rock riprap, upgrades to the FRWCS fish passage, a diversion structure and inlet weir at Buffalo Creek and the inlet of Reach 3, diking (for Reach 2), permanent access roads, rail bridge removal, construction camp, revegetation and mitigation costs.

The basis for the unit measures and prices for preliminary use in this study are documented in Section 7.8 of the report. The following section describes how the costs for each component of the outlet channels were developed for the various options and includes unit measures and pricing for updated project components that were not discussed in previous studies.

15.5.2 Cost Estimate Description

The estimated construction costs for the Lake Manitoba and Lake St. Martin outlet channel options are provided in Table 15.11. These were calculated based on the quantities for each channel option previously described in Section 15.3 and the unit pricing described in Section 7.8. Details and clarifications for calculating quantities and/or unit pricing that were not previously described are provided in the following sections.

Table 15.11
Estimated Construction Quantities and Costs
(Included at the end of Section 15.0)

Excavation

Earth and rock excavation costs were calculated by multiplying the estimated volumes by the unit pricing. Option A was unique in that a premium was added to the unit pricing for earth excavation to account for the two locations where the alignment crosses the Fairford River, as this will require excavation in the wet and localized modification of the riverbanks. As such a blended rate of $12/m^3 was used for this option.

Option F was also unique in that it required fill for machinery access to excavate the existing river channel down to the invert level. The fill volume required for construction was estimated to be 1,200,000 m^3 at $25/m^3. The cut volume below the water was estimated to be 1,800,000 m^3, plus an additional 1,200,000 m^3 to remove the temporary fill, at a unit price of $25/m^3. The cut volume above the water was estimated to be 680,000 m^3 at $10/m^3. The costs for the fill volume, and the cut volume above and below the water level for Option F were combined and input as a single entry into the cost table (Table 15.11 – located at the end of Section 15.0) within the excavation category.

Rock Riprap

Rock riprap is required to armour the downstream side of concrete control structures and bridges. It will also be used for construction of rock fill gradient control drop structures along Reach 3, as previously described in Section 15.3. Rock riprap will also likely be required to stabilize a section of bank along Buffalo Creek within the downstream end of Reach 2 that is highly erodible. The cost for riprap armouring has been incorporated as part of the cost estimate for each concrete structure. Whereas, the cost estimate for the rock fill gradient control drop structures is based on an estimated quantity of 11 gradient control structures for Willow Point, 12 structures for Johnson Beach, and the footprint area of each structure. Quantities for bank stabilization in Reach 2 were not calculated at this conceptual stage of analysis. Rather, an allowance of $2 million was included in the estimate.

The unit rate for the rock riprap cost estimate ($90/m^3) was based on average unit prices in the vicinity of Winnipeg where rock riprap must be hauled from existing off-site quarries. This assumption was based on analysis of bedrock samples collected from the rock ridge at the

Reach 3 site in 2012, which suggested that the bedrock at site was not suitable for use as riprap. The bedrock samples were submitted to MIT testing laboratories for LA abrasion, sulphate soundness, and specific gravity and absorption testing. The bedrock was found to be highly susceptible to mechanical breakdown (56% losses in the LA abrasion test) and freeze-thaw action (45% to 46% losses in the sulphate soundness test). The limestone had a specific gravity that ranged from 2.55 to 2.59, and was also found to be highly absorptive (3.0% to 3.4%). The results of the testing indicated that the local bedrock did not meet MIT Riprap Standard Specification 1297 and would only be considered for temporary erosion protection (5 years or less) and emergency usage. It is possible that other sources of bedrock in proximity to the Lake St. Martin and Lake Manitoba channel options may be discovered and developed into quarries. However, a conservative value for the unit price of riprap was used to account for the uncertainty in terms of bedrock quality.

Buffalo Creek Diversion Structure and Reach 3 Inlet Weir

Any Reach 3 option will require a diversion structure in Buffalo Creek downstream of the inlet location to divert flows into Reach 3, while maintaining riparian flows in Buffalo Creek. As part of this design, a weir structure would have to be developed at the inlet to Reach 3. At this conceptual stage of analysis, an estimated lump sum value has been allotted of $2.5 million.

Diking

Diking is required within Reach 2 to contain flows within Buffalo Creek. Fill quantities for the dikes were calculated based on assumed dike geometry with a 4 m (13 ft) top width and 4:1 side slopes. The base of the dike was assumed to be 1 m (3.3 ft) below the top of peat. The dikes also incorporated a 0.6 m (2 ft) height allowance for freeboard, and a 2 m^2 per linear meter allowance for a cutoff trench. The base of the dike was also assumed. In Table 15.11, the line item for diking is a combined value for dike fill and clearing and grubbing. The assumed unit price for dike fill and clearing and grubbing was $20/$m^3$, and $16,500/ha, respectively.

Control Structures

The estimated concrete control structure costs for each option were calculated based on the footprint area of each structure and the number of structures required. This was estimated using the lengths that were calculated to provide the required capacity and a standardized 14 m (46 ft) width, which is based on the dimensions of the FRWCS. The control structure would have piers spaced at approximately 6 m (20 ft) and a bridge deck complete with hoisting structure from which the stoplogs could be removed manually when required to release flows. While the height and subsequent width of the control structure for the Reach 1 options may not be required to be as wide as the Lake Manitoba options, there is an unknown in terms of the footing requirements (will be built on till rather than bedrock). Therefore, a conservative value was considered more appropriate at this time.

Option F is unique in that it requires the addition of two bays to the existing FRWCS, which will require a cofferdam, temporary traffic control, and relocation of the existing abutment and approach. Therefore, the actual footprint area of the additional two bays was increased for this option to account for the associated extra costs.

Increased Fish Passage at FRWCS

As indicated in Section 15.1 (Background), the Lake Manitoba and Lake St. Martin Regulation Review Committee recommended that: *"If the Province builds a new outlet to Lake Manitoba as recommended, it should take that opportunity to design the new outlet in such a way as to provide unrestricted fish passage between Lake Manitoba and Lake St. Martin"*. However, as indicated Section 15.6.4, unrestricted fish passage would likely be ineffective if included as part of a newly developed Lake Manitoba outlet channel. Instead, improvement for fish passage at the current FRWCS will be included as part of the proposed project, regardless of the Lake Manitoba outlet channel selected. As such, an estimated lump sum value of $2 million was included for each of the Lake Manitoba outlet channel options, as a cost allowance for a new or improved fish passage structure at the FRWCS. Improving fish passage at the FRWCS with the construction of a new external fishway would also provide an opportunity to decommission the existing fishway, which is located within one of the FRWCS bays. This could improve the

hydraulic capacity of the structure at a relatively low cost and should be considered further in future stages of the project.

Bridges

The estimated bridge costs for each option were calculated based on the footprint area of each bridge and the number of PTH and municipal road bridges required. The footprint area was estimated using the lengths that were calculated for the PTH bridges and the municipal bridges, and a standard width of 12 m (39 ft) and 10.8 m (35 ft), respectively. Because the PTH bridges will be built over the proposed control structures, the bridge lengths were based on the length of the control structures, whereas, the lengths for the municipal bridges were based on the average top width of the channel (in till sections). The standard widths of the bridges were based on the information developed for the Analysis of Options report (KGS Group and AECOM, 2011). While Option D crosses PR 239, for the purposes of the cost estimate, it was treated as a municipal road. As previously noted, it may be possible to reduce the number of new bridge crossings required for some options by localized realignment of roads. A lump sum of $3 million was added to each PTH bridge to account for approaches / traffic control, as discussed in Section 7.8 of the report.

Option F would require removal of the abandoned railway bridge to increase the capacity of the Fairford River. A lump sum allowance of $1.0 million was assumed, which includes the salvage value of the steel superstructure.

Land Acquisition

The area of land acquisition required for each channel option was estimated by multiplying the channel length by the width of an appropriate Right-of-Way (ROW). The ROW for each channel option was estimated by adding 140 m (460 ft) to the channel base width, which is comparable to distances used in the design of Reach 1 and Reach 3. The ROW represents the area of the channel top width, berms, spoil piles, drainage ditching and a buffer zone. The calculation of estimated land acquisition required for the Lake Manitoba Options is summarized in Table 15.12.

Table 15.12
Estimated Land Acquisition

Option	Base Width (m)	ROW (m)	Channel Length (m)	Land Acquisition (ha)
A1	26	166	12300	204
A2	64	204	12300	251
A3	112	252	12300	310
B1	31	171	11500	197
B2	72	212	11500	244
B3	113	253	11500	291
C1	31	171	11600	198
C2	72	212	11600	246
C3	113	253	11600	294
D1	27	167	22800	381
D2	64	204	22800	464
D3	100	240	22800	546
E1	9	149	2150	32
E2	21	161	2150	35
F	95	180	18700	336

Permanent Access Roads

The Lake Manitoba outlet channel options will not require significant development of permanent roads for construction purposes, and access to the control structures for operation will be via PTH 6. The Lake St. Martin outlet channel options could be accessed along winter road systems for construction purposes; however, this would restrict all construction activities to winter months (late December to mid-March). The cost estimate assumes that an all-season road will be required to permit year round access to the project site for construction and also in the future for the purpose of operating the control structure, maintaining revegetation components of the project, and general maintenance to the structure and channel. All-season access to Reach 2 and Reach 3 would be required only if access to Reach 1 was already established, therefore, the cost estimates for Reach 2 and 3 include only the incremental cost to extend access from Reach 1.

There is an existing forestry road approximately 61 km (38 mi) in length extending from Spearhill Manitoba that was used for access during the construction of Reach 3 in 2012. At the end of the forestry road, a winter road was developed extending the remaining distance to the Reach 1 inlet and Reach 3 project site. The Reach 1 winter road access is approximately 19 km (12 mi). The winter road access to Reach 2 and 3 from the Reach 1 access road would require an additional 16 km (10 mi) road to Reach 3 and a further 3.7 km (2.3 mi) to Reach 2. It is estimated that upgrading the current 61 km (38 mi) forestry road to a single-lane gravel road would cost approximately $100,000/km for a total of $6.1 million. It is estimated that upgrading the current winter road access to a single-lane gravel road would cost approximately $750,000/km. Therefore, access to Reach 1 would cost a total of $20.35 million ($6.1 million to upgrade the forestry road plus $14.25 million to upgrade winter road). The estimated incremental cost to upgrade the winter road to Reach 3 would be $12.0 million with a further incremental cost to Reach 2 of $2.78 million.

Construction Camp

The construction camp estimates are based on the size of the construction camp used for the development of Reach 3 in 2012. A daily cost per person was estimated at $102/person to cover food and lodging for onsite staff. A mobilization and demobilization fee was included, which covers the costs of provincial permits (permit for storage and handling of petroleum products, explosive magazine permits, and an onsite wastewater management systems permit), and the installation and removal of trailers for lodging, washhouses, dining area and office space ($2,800/unit). These prices assume that the camp equipment / infrastructure is owned by the contractor that will be administering the camp rather than renting this required infrastructure from an outside source.

The estimated construction camp costs vary based on the maximum number of staff proposed to be in the camp at one time, which is a function of the project magnitude and construction schedule. A construction camp cost for the Lake St. Martin Reach 3 outlet channel to Johnson Beach, with an increased capacity of 142 m^3/s (5,000 cfs; R3-JB +5000), was estimated and assumed a base requirement of 50 staff for a period of 270 days (nine months) and 14 units. This option was deemed to be the most comparable to the construction camp operated during the development of Reach 3 in 2012, which was used as a basis for the cost estimate. The

construction camp costs for the remaining options were estimated by prorating the cost relative to the estimate for R3-JB +5000, based on the subtotal of all direct costs (excluding the construction camp). The prorated value is intended to reflect the appropriate construction schedules and size of the construction camp relative to the size of each option and the complexity due to additional structures, rock excavation, or underwater excavation.

Revegetation

The estimated cost to provide appropriate weed control and a vegetation cover for any of the outlet channel options at Lake Manitoba or Lake St. Martin is based on costs for conducting similar work along East Side Road, which was developed and administered by Scatliff Miller Murray Inc. For East Side Road, revegetation costs ranged from approximately $8,000/ha to $15,000/ha. For this study, an average value of $11,500/ha was assumed. This price assumed:

- Specifications will be developed prior to construction activities to allow for appropriate stockpiling of overburden and site / seed bed preparation (slopes, organic cover, mechanically preparing soils, etc.).
- All shaping of embankments and site / seed bed preparation is included in the construction costs.
- Appropriate site access is available for all options.
- Appropriate site conditions for seeding requirements.
- No significant delays or hindrances to revegetation work.

Total revegetation costs were calculated based on the total hectares estimated for the land acquisition (ha) multiplied by the unit price/ha. This estimate is likely conservative, since revegetation will not be required over the length of the rock sections that are included in the total area. This will require further discussions prior to developing the specifications for weed control and vegetation cover.

Mitigation Costs

Costs associated with proposed best practices to mitigate effects during construction, such as control of surface water and groundwater (dewatering, ditching, silt fences and settling ponds) have been included in the construction cost estimate as part of the unit price for excavation.

Design measures such as riprap to mitigate the effects of erosion and the use of control structures to reduce velocities during operation have been incorporated in the channel design and the costs included as part of the cost estimate for structures. Likewise the costs associated with land acquisition to compensate people for land required as part of the channels has been included, as previously described.

Mitigation and compensation costs that have not been accounted for in the cost estimate include costs for long-term monitoring, groundwater mitigation costs, wells affected by the project (Lake Manitoba routes), compensation for damage to fish habitat, compensation to fishermen or any additional mitigation and compensation measures that may come from Regulator requirements determined during the advancement of the licensing and permitting process for the channel. These costs have been estimated as a percentage of the direct project costs with the percentage relative to the potential environmental effects as described in Section 7.0. The percentages used in the cost estimate for each option were as follows:

- Option A - 10%.
- Option B - 5%.
- Option C - 5%.
- Option D - 7%.
- Option E - 8%.
- Option F - 15%.
- Option R1 - 5%.
- Option R2 - 4%.
- Option R3 (JB) - 6%.
- Option R3 (WP) - 6%.

Miscellaneous Allowance

A miscellaneous allowance was added to the cost estimate, assumed at 5% of the total construction costs for each option. This value was added to account for additional fees that will likely be required for the project that have not been quantified at this conceptual phase of

analysis. This includes, but is not limited to, mobile batch concrete facilities, bringing in electricity to the site (Reach 1 and possibly Reach 3), unit cost premiums, etc.

15.5.3 Cost Summary of Outlet Channel Options

The total costs for each of the Lake Manitoba and Lake St. Martin outlet channel options, for the various capacities assessed are summarized in Table 15.13. This cost summary is based on the project components for each option as described in Section 15.3, the unit pricing and the basis for the costing described in Sections 15.5.1 and 15.5.2, as well as the 30% contingency and 20% fees for engineering, administration and environmental approvals.

Table 15.13
Cost summary of Outlet Channel Options

Design Alternatives	Total Cost ($Mil CAD) by Capacity (cfs)				
Lake Manitoba Channels	**2,500**	**3,750**	**5,000**	**10,000**	**15,000**
Option A – Twinning the Fairford River	-	-	100.9	168.8	252.2
Option B – Channel South of Pinaymootang FN	-	-	186.5	324.1	461.8
Option C – Channel Slightly Less South of Pinaymootang FN	-	-	167.7	291.7	415.2
Option D – Channel Following Birch Creek	-	-	158.0	259.0	360.3
Option E – Bypass Channel North of FRWCS	22.2	30.0	-	-	-
Option F – Expansion of Fairford River / FRWCS	-	-	267.6	-	-
Lake St. Martin Channels		**4,000**	**9,000**	**14,000**	**19,000**
Reach 1		62.1	88.6	123.5	154.6
Reach 2		6.0	13.5	29.1	41.2
Reach 3 to Johnson Beach		38.4	65.6	88.3	110.8
Reach 3 east of Willow Point		45.2	78.1	107.0	134.3
Option JB – Reach 1, 2 and 3 to Johnson Beach		106.5	167.6	240.9	306.6
Option WP – Reach 1, 2 and 3 east of Willow Point		113.3	180.1	259.6	330.1

Notes: "-" indicates alternative was not assessed for this capacity.

Option E with a 71 m^3/s (2,500 cfs) capacity was the lowest estimated cost for the Lake Manitoba outlet channel options as shown in Table 15.13. For the outlet channels that considered larger capacities (larger than 142 m^3/s (5,000 cfs)), Option A was nearly half the cost

compared to the other options. The remaining options from least expensive to most were Options D, C, B and F.

For the Lake St. Martin outlet channels, the Reach 3 to Johnson Beach option was approximately 7 to 8% lower in cost compared to the Reach 3 east of Willow Point option as shown in Table 15.13.

Cost estimates for channel capacities that differ from those evaluated in Section 15.3 can be interpolated from the values presented in Table 15.13. For example, the estimated cost for a Lake Manitoba Outlet Channel with a capacity of 212 m^3/s (7,500 cfs) would be $229.5 million for Option C and $208.5 million for Option D. On Lake St. Martin, the estimated cost for an Outlet Channel with a capacity of 326 m^3/s (11,500 cfs) would be $204.3 million for Option JB and $219.9 million for Option WP.

15.6 POTENTIAL ENVIRONMENTAL CONCERNS

The proposed construction and future operation of an outlet channel between Lake Manitoba and Lake St. Martin and the subsequent modifications to the existing outlet channel system between Lake St. Martin and Lake Winnipeg will provide a large scale positive effect by helping to alleviate flooding in the region. However, they will also likely result in a number of environmental concerns. Biophysical attributes that may be affected include, but are not limited to, surface water quality, groundwater, terrestrial environment, aquatic environment and fish resources. Social attributes and concerns that could affect the project include land use and ownership, access, and unresolved First Nation litigation claims. While these and other potential environmental concerns will be examined in detail during the development of future licensing and permitting for the proposed project, this report acknowledges the likelihood of particular environmental concerns from a general perspective and focuses on the localized effects along each of the individual outlet channel options to better assess the relative difference between them.

A description of the general environmental concerns is provided in the following sections along with a relative ranking of the channel options from best to worst (on a scale from 1 to 10 with 10 being the best indicating the smallest effect). The relative ranking is qualitative, and the criteria

have not been weighted based on the potential level of effect for this preliminary assessment. For example, criteria that may be assigned more importance could include erosion / sedimentation, fish resources, land use / ownership and First Nation litigation; however, this was beyond the scope of this conceptual study. The ranking is based on what is known about a project of this nature and the existing information about the current proposed locations for the outlet channels, as well as our understanding of certain environmental impacts likely to occur. The information herein will be used to gauge the extent of potential impacts for each of the outlet channel alignments, with the rankings summarized in Table 15.14 in Section 15.6.7 so as to aide in the selection of preferred options, with the expectation that an environmental assessment will follow.

15.6.1 Surface Water Quality

Based on examination of monitoring data for the Reach 1 Emergency Outlet Channel in 2011 through 2012, (KGS Group, 2013; NSC, 2013), there would likely be varying levels of impact to surface water quality during construction, initial operation and continued operation of the proposed outlet channels. Water quality in direct proximity to the project activities and the downstream receiving waters can be affected through several physical effects pathways associated with the proposed Lake Manitoba and Lake St. Martin channels including:

- Alteration in the rate and seasonality of flow discharged into downstream water bodies within the project study area.
- Effects of flooding along the outlet channel routes.
- Potential for erosion and/or mobilization of sediment due to the diversion.

Many of the details described in the following sub-sections were previously developed as a part of the base of examination for the 2011-2012 monitoring program for the construction and operation of the Reach 1 Emergency Outlet Channel (NSC, 2013).

15.6.1.1 Alterations in Rate and Seasonality of Flow Discharges

The proposed project will result in an increased rate of discharge from Lake Manitoba through Lake St. Martin and into Lake Winnipeg. These water bodies typically drain into one another in this sequence; however, the outlet channel systems will alter the timing and volume of the

discharge. The concern is that these alterations could lead to changes in the water quality within the downstream water bodies. Preliminary review of routine water quality variables indicated that water quality in the Fairford and Dauphin Rivers exhibit seasonal differences, which may in turn affect conditions in Lake Winnipeg (NSC, 2013).

The outlet channel would be operated when needed in response to flood conditions, regardless of which option is selected, so there would be little to no difference between any of the options in relation to changes in timing of discharge. In comparison, the change in rate of discharge is directly related to the increased volume or capacities of the proposed outlet channels and the volume of the downstream receiving water body. As such, each reach of the Lake St. Martin outlet channel options was ranked the same because they all have the same potential increase in capacity, while they ranked better than the Lake Manitoba outlet channels because they will have less effect given the larger volume of Lake Winnipeg. Option E was ranked the best of the Lake Manitoba channels, as it had the lowest increase in capacity followed by Option F; whereas Options A, B, C and D were all equally ranked the worst, as they all have the largest potential capacity increases.

15.6.1.2 Effects of Flooding Along the Outlet Channel Routes

Flooding of terrestrial habitat may affect water quality through leaching and decomposition of organic materials, which may in turn consume dissolved oxygen (DO), decrease pH, and increase the concentration of various nutrients and metals in surface water. Flooding of terrestrial habitat may also mobilize mercury and increase the methylation of mercury leading to increased concentrations downstream and potential bioaccumulation within aquatic biota. This could in turn be affected by factors such as the total area flooded, differences in terrestrial habitat that is inundated (mass of organic material), and changes in water residence times and thermal regimes.

All of the proposed channels will provide a benefit to some extent as they are designed to decrease flooding within the drainage basins and the associated adverse effects. Of the Lake Manitoba channels, Options B and C were ranked the best as they will relieve the flooding on Lake Manitoba with no new flooding along the channel. Option D was next best as only minor flooding would likely occur in the small lakes, creeks and wetland complex along the route.

Whereas Options A, F and E would increase potential flooding on Lake Pineimuta, with Option E being the worst, as it provides the least reduction in flood relief on Lake Manitoba, while potentially flooding Lake Pineimuta.

While each reach of the Lake St. Martin channel options has equal capacities to reduce the flooding on Lake St. Martin, Reach 1 results in the potential for flooding of Buffalo Lake and the surrounding bog, whereas Reach 2 and Reach 3 have containment dikes to prevent flooding of the surrounding land. Both the Johnson Beach and Willow Point Reach 3 options were ranked the same because they both divert the flood waters directly to Lake Winnipeg without flooding the surrounding land.

15.6.1.3 Erosion and Sediment Mobilization

The construction, initial operation, and continued operation of the outlet channels have the potential to increase shoreline, lake bed, and creek bed erosion, which can cause increases in the concentrations of Total Suspended Solids (TSS) / turbidity and related variables in downstream receiving waters. The impacts to water quality parameters such as TSS can have lethal and sub-lethal adverse effects on fish and other aquatic biota.

Construction activities such as excavation of shorelines or in proximity to shorelines, dredging within waterways or water bodies, or dewatering project sites in direct proximity to surface water bodies or watercourses can result in localized plumes of suspended sediment. To mitigate the effects of these activities, use of appropriate sediment control measures such as silt fences, turbidity curtains, and cofferdams (to allow the activities to proceed under dry conditions) would be required. In situations where dewatering is required, dissipation mats and or the use of sumps / settling ponds can reduce the extent of turbidity.

During initial operation of any channel option, it is expected that an immediate increase in TSS concentrations will be observed. This spike in the concentration of TSS could be from a combination of any of the following:

- Mobilization of loose earth material within the outlet channels.
- Eroding of the channel banks.

- Release of materials from the upstream water body being re-suspended as the inlet structure is operated.
- Movement of materials from within water bodies or wetland along the channel route.

While it is probable that the increase in concentrations during initial operation will be well above typical conditions for the downstream receiving waters, these effects are expected to be short-term in duration. The effects from prolonged operation of the emergency channels have the potential to continue to provide increased TSS / turbidity loading into Lake St. Martin and Lake Winnipeg; although, this would be in much lower concentrations, as the channels have been designed so that average flow velocities are below the threshold to cause erosion. Periodic monitoring of the water quality throughout the project study area is advised to ensure that concentrations of routine water quality parameters are not exceeding conditions, which will have detrimental impacts on fish and other aquatic biota.

All the proposed channel options will require construction of an inlet and outlet requiring work in or adjacent water. However, based on the above discussion, the options that require additional construction activities within or immediately adjacent to water would be worse than channels through dry land. Additionally, channel options that are longer, and in particular through till sections, would have more materials exposed for potential erosion during initial operation. As such, Option F was ranked the worst and much lower than the other Lake Manitoba options as it required excavating the river bank and dredging the river bottom to increase the capacity, as well as other in water work to expand the existing FRWCS, construction of a new downstream control structure and the decommissioning of a rail bridge. Additionally, Option F is one of the longest channels and constructed entirely within till that could continue to erode during long-term operation. Option A was ranked second worst as the route crosses the Fairford River twice requiring substantial in water work and was one of the longer routes also constructed entirely in till. Option D was ranked third worst as the channel is in close proximity to several small lakes and wetlands along Birch Creek and is the longest route, constructed entirely within till. In comparison, Options B, C and E would be mostly constructed in the dry, with Option B and C ranked the same, as they have similar lengths of channel in till, and Option E ranked the best as it was the shortest.

All three reaches of the Lake St. Martin outlet channel will likely require work immediately adjacent to water or some in water work. Reach 1 will require construction of the control

structure at the inlet and dredging to improve the existing inlet. Reach 2 will require bank excavation along Buffalo Creek. Both Reach 3 options will require shoreline excavation and construction of the outlet to Lake Winnipeg. The effects associated with initial and long-term operation are directly related to the length of each Reach, with greater effects for longer channels. An additional consideration was that Reach 1 will input sediment into Buffalo Lake and that both Reach 3 options will input sediment into Lake Winnipeg. Based on these considerations, the Reach 3 channel to Johnson Beach was ranked slightly better compared to the channel to Willow Point as it was approximately 1.7 km (1.1 mi) shorter.

15.6.2 Groundwater

Excavation of the proposed channels into the till and in particular bedrock sections will likely result in constant seepage of groundwater into the channel that will lower the water table and may affect groundwater wells in the vicinity of the channel. Likewise flood waters in the channel could lead to potential infiltration, especially in areas where the channel intersects bedrock. When the flood waters are from developed or agricultural land, there is the potential for the infiltration to cause aquifer contamination. Additional construction related impacts include the potential for contamination of groundwater resulting from leaks and accidental spills of fuel or other hazardous substances. These potential groundwater issues are more of a concern for the Lake Manitoba options as there are groundwater wells in the bedrock aquifer that are used as a source for potable water. Mitigation of wells affected by the project along Lake Manitoba routes would include deepening or drilling new wells. Mitigation to prevent leaks, spills, and releases include providing secondary containment for fuel and hazardous material storage, requiring drip trays for equipment, providing spill clean-up equipment and materials, and providing an emergency (spill) response plan.

Construction of the Lake St. Martin outlet channel may adversely affect the groundwater regime or cause potential contamination during construction, as noted above; however, because the groundwater in this area is not used as a potable water supply, each of the reaches would likely not have a significant effect. Options E and F were ranked the best of the Lake Manitoba channels. There are very few wells in the vicinity of Option E; it is the shortest channel and constructed entirely in till; whereas, because Option F is expanding the existing river, it should have minimal changes to the groundwater regime. Options D was ranked the next best

because, again, there are few wells in the vicinity; and the channel will be constructed entirely in till, although it is the longest of the routes. Options A and B were ranked next best, while Option C was ranked the worst. While Option A is constructed entirely in till, there are many domestic wells in the vicinity of the channel that may be affected. In comparison, Option B has very few wells in the vicinity, however, there is a portion excavated through the bedrock so it may have a more direct effect on the groundwater. Option C was ranked the worst, as there are several wells in the vicinity; and a portion of the channel is excavated through bedrock.

15.6.3 Terrestrial Environment

The key components of the terrestrial environment that may potentially be impacted by this project include vegetation associated with clearing for the channel ROW, and the associated impacts to wildlife through physical disturbance and loss of habitat.

15.6.3.1 Vegetation

Vegetation will be lost during the site preparation and construction activities such as clearing, excavation, and blasting. Secondary impacts of construction activities include introduction of weedy species and the production of fugitive dust, which can settle on vegetation surrounding the project site and impair growth and development. For most channel options, the extent of impacts to vegetation is a function of alignment length, as the entire right-of-way will be cleared to develop the channel. A qualifier for determining the extent of impact is the quality of vegetation being cleared; agriculture fields or heavily disturbed areas may be viewed as lower quality compared to relatively undisturbed vegetation. Mitigation to prevent these effects may include ensuring equipment is clean to avoid transfer of weedy species, as well as taking measures to control dust such as dust suppressants, covering loads when hauling materials and restricting activities during high wind events, minimizing clearings to the extent absolutely required to complete the tasks, whenever possible using previously disturbed areas, and revegetate disturbed and reclaimed areas after construction.

Operation of several outlet channels (Options A, F, D and Reach 1) will result in higher flows along natural waterways and could result in losses in vegetation either from inundation, or from

high velocity of the water eroding and flushing the substrate material downstream. No mitigation measures are proposed for this effect.

Option E was ranked the best of the Lake Manitoba channel options because it would require the smallest area of disturbance, and most of the vegetation is lower quality consisting of already disturbed agricultural land. Option C was ranked the next best because it also consists of mostly disturbed agricultural land and would require a similar area of disturbance to many of the other options. Options A, B and F were ranked the next best with Option A and B requiring a similar area of disturbance as Option C; however, the route would require clearing more undisturbed forested land. While Option F would require less area of disturbance than Options A and B, it was ranked similarly because there was less previously disturbed vegetation, with a large portion of the vegetation being riparian wetland vegetation considered important for wildlife. Option D was ranked the worst as it would disturb at least twice as much area as most of the other Options, and the vegetation that would be impacted was a mix of previously disturbed agricultural land and wetland vegetation.

Reach 1 of the Lake St. Martin outlet channel was previously cleared when the channel was originally constructed in 2011 and will, therefore, have minimal further impact. In comparison, the vegetation around Reach 2 is largely undisturbed and will need to be cleared; however, this area of disturbance is relatively small. The Johnson Beach option for Reach 3 is ranked better than the Willow Point option because it requires a smaller area of disturbance and more of this route has been cleared previously.

15.6.3.2 Wildlife/Habitat

The vegetation clearing during site preparation and construction activities will result in a direct loss of wildlife habitat used for breeding and foraging and may have an indirect effect through construction noise that could disturb wildlife during nesting and rearing. Habitat may also be disturbed or lost through modification of natural conditions through inundation or drainage associated with construction and operation of the outlet channels. Mitigation to reduce the adversity of these effects may include cataloging and avoiding, when feasible, sensitive areas or areas that contain species of interest, in particular, during critical nesting and rearing periods (typically spring and early summer).

Construction activities such as transporting materials and increased vehicle traffic along highways and access roads will increase the risk of vehicle-wildlife interactions and the potential for disturbance, injury or mortality. Mitigation may include operating transport trucks during daylight hours, providing wildlife awareness information to drivers and adhering to set speed limits.

Project effects to wildlife and wildlife habitat, as noted, are directly related to the amount and quality of vegetation disturbed. As such, the ranks assigned to each of the outlet channels reflect the ranks previously assigned under vegetation. The rank assigned to each reach of the Lake St. Martin outlet channels was lowered relative to the Lake Manitoba outlet channels because the area has less human disturbance and, therefore, provides better habitat and more wildlife are likely present that could be affected by these channels.

15.6.4 Fish Habitat

The proposed project has the potential to result in the alteration and loss of fish biota habitat. Habitat loss would occur from the construction footprint of structures within water bodies, while habitat alteration would occur from shoreline excavation, increased or altered flow patterns, flooding, and water quality changes from erosion and sedimentation. The habitat alteration and loss on its own is not likely to result in serious harm to the fishery, although some effort should be made to assess the quality of the habitat that will be affected for the preferred option.

Option F will require the construction of a control structure in the Fairford River downstream of Lake Pineimuta, which will result in the loss of fish habitat. Construction of the inlet and outlet for each of the Lake Manitoba and Lake St. Martin channels, as well as other in water work such as shoreline excavation and dredging, will result in alteration of fish habitat. However, this alteration of fish habitat may be offset by the creation of fish habitat in the channels. Upstream of the control structures the channels will typically contain water and provide fish habitat, except during low water years when the upstream lake levels are lower than the inlet. Downstream of the control structure the channels will only provide temporary fish habitat during operation until the flows are ended.

Several options for the Lake Manitoba outlet channels (Options A, E, and F) and the Lake St. Martin outlet channel will involve increased flows or altered flow patterns along existing water ways including the Fairford River and upper Buffalo Creek. Within the Fairford River, habitat changes could occur from increasing water levels and velocities altering the existing erosion and sedimentation patterns. Similar to what was observed in 2011-2012, the operation of Reach 1 will result in increased water levels in Big Buffalo Lake and flooding of the surrounding bog and wetland areas. This will provide additional useable habitat for fish during operation and for some time after closure as the water recedes. However, the inclusion of flow from Reach 1 and subsequent increased flow out of Big Buffalo Lake have the potential to alter the habitat type towards a more riverine environment, which could reduce the residency time for fish within the lake. As water continues along the upper portion of Buffalo Creek, the increase in flow and potential for scour could alter channel geometry, and certainly impact the riparian and in-stream vegetation and fine-grained sediment.

The downstream end of all channel options could be used as temporary fish habitat, if fish were able to access the downstream end, except Option F and Reach 2, which are part of a natural water body. After closure of the channel control structures, the water would recede; and, for most options, the channel will be inaccessible to fish. For channel options that connect directly to existing waterways (Fairford River and Buffalo Creek), the reduction in flow could result in exposure of spawning habitat and a reduced littoral zone in the natural waterways. This could result in exposure of egg incubation and early larval rearing habitat in these waterways.

Option F was ranked the worst of the Lake Manitoba options because it would result in habitat loss and likely the greatest amount of and most destructive habitat alterations associated with the downstream control structure, excavating the river bank and dredging the river bottom, as noted above. Option A and E were ranked the next worst. Option A will result in substantial habitat alterations at the two locations where the channel crosses the Fairford River and will result in flow alterations due to the twinning of the river. While Option E will only have minor habitat alterations at the inlet and outlet because it is the shortest channel, there will be relatively little fish habitat created to offset the alterations made. Option D was ranked next worst as the channel is in close proximity to Birch Creek and may require in water work to tie the channel into the creek near the downstream end, although route optimization may eliminate the in water work. In comparison, Options B and C were ranked the best as they would be

constructed mostly in the dry, except the habitat alterations at the inlet and outlet, which would be compensated for by the similar lengths of channel providing new fish habitat.

All three reaches of the Lake St. Martin outlet channel will require alteration of fish habitat. Reach 1 will require dredging to improve the existing inlet; Reach 2 will require bank excavation along Buffalo Creek; and both Reach 3 options will require shoreline excavation and construction of the outlet to Lake Winnipeg. While Reach 2 will result in the alteration of the largest area of fish habitat, this will be offset by the creation of a substantial area of new habitat within the widened portion of Buffalo Creek. Both the Johnson Beach and Willow Point Reach 3 options were ranked the same because they are mostly in the dry, except the minor area of habitat alterations at the outlet.

15.6.5 Fish Resources

Three aspects of the project that could impact fish resources in the study area include habitat change, disruption of access to habitat and re-distribution of fish species from changes in flow patterns.

15.6.5.1 Habitat Change from Alterations in Flow Patterns

Within the Fairford River, Buffalo Lake, upper Buffalo Creek and Birch Creek, increase in flows could result in flooding and erosion of aquatic and terrestrial vegetation. This could affect the availability and distribution of spawning habitat and potentially affect spawning behavior and timing. Erosion of substrate composition could also affect both egg distribution and incubation success rates. Similarly, increased depth, sedimentation, and turbidity from increased flow could impact the littoral zone habitat used by many species of fish for rearing and feeding and could result in a temporary redistribution of fish to other areas within the watershed or to adjacent, unaffected watersheds. Other indirect effects include transportation and redistribution of invertebrates, which serve as prey items for the fish and may result in movement of fish to locations further downstream.

Option B and C were ranked best of the Lake Manitoba channels because they were not connected to any other natural water body along the route in which flow patterns or water levels

would be altered. Option D was ranked next best as the proposed route could be optimized so that it would not have any connection to Birch Creek, although currently the channel is shown connected at the downstream end, which could change fish habitat. Option A, E and F were ranked much lower because these options are directly connected to the Fairford River and Lake Pineimuta with the potential to change water levels and flow patterns causing the associated effects described above. Option E was ranked the worst because it would result in an increased flow capacity without increasing the capacity of the Fairford River further downstream to prevent water level increases on the river or Lake Pineimuta.

Reach 1 of the Lake St. Martin outlet channel would result in an increased flow to Buffalo Lake and the associated effects described above. Similarly, Reach 2 will have increased flows to upper Buffalo Creek; however, widening Buffalo Creek will mitigate these effects. Reach 3 to both Willow Point and Johnson Beach were ranked the same because both options would result in an increased flow to Lake Winnipeg and the associated effects related to sedimentation.

15.6.5.2 Change in Species Composition and Distribution

Increased flows and changes in flow pattern may result in velocity barriers to certain species of fish that are adapted to lower flow conditions (e.g., yellow perch, northern pike, etc.), which may limit the amount of available habitat for these species during operation. Conversely, fish that are more adapted to higher water velocity conditions may become more prevalent during operation of the outlet channels. Depending on the timing, higher flows may negatively impact egg and larval fish distribution. Eggs may be flushed downstream into habitat unsuitable for incubation and drifting larval fish may be transported to areas further downstream that are unsuitable or have increased numbers of potential predators. Both scenarios can lead to increased mortality rates.

Alternative flows and/or increased flows may act as attractants to certain fish species during fall and spring spawning runs, affecting the timing and distribution of spawning activity. This could occur at the downstream end of all the Lake Manitoba channel options (excluding Option F) and at the downstream end of either alignment option for Reach 3. Fish that would normally migrate up the Fairford or Dauphin Rivers may alternatively ascend the corresponding channel. Closure of the channels following operation may result in stranding of fish as the water recedes.

Consideration should be given to the operation and closure of the channels so as to reduce the effect on spring and fall spawning runs. Further, consideration should be given to installing drop structures at the downstream end of all Lake Manitoba options (except Option F) and for Reach 3, to prevent fish from ascending the channels during migration. Alternative options will have to be considered for Reach 1, as a drop structure will not be feasible at the downstream end of that reach.

Option B and C were ranked best of the Lake Manitoba channels because they were not connected to any other natural water body along the route in which flow patterns or water levels would be altered and, therefore, only have the potential concern of fish stranding. Option D was ranked next best as the proposed route could be optimized so that it would not have any connection to Birch Creek, although currently the channel is shown connected at the downstream end, which could change flow patterns and velocity in addition to the concern of fish stranding. Options A, E and F were ranked lower because these options were directly connected to the Fairford River and Lake Pineimuta with the potential to change water levels and flow patterns affecting fish composition to a greater extent. Option F was ranked better than Option A because there was no concern for fish stranding in the Fairford River expansion. Option E was ranked the worst because it would result in an increased flow capacity without increasing the capacity of the Fairford River further downstream resulting in the greatest potential for water level increases and flow pattern changes.

Reach 1 of the Lake St. Martin outlet channel has a large potential to change flow patterns and water levels on Buffalo Lake and is a demonstrated concern for fish stranding. Reach 2 consists of widening Buffalo Creek to increase the capacity to ensure water levels on Buffalo Lake are contained, and there is no concern for fish stranding in Buffalo Creek. Reach 3 to both Willow Point and Johnson Beach were ranked the same because both options will act as an attractant to certain species with the increased flows at the outlet and may be a concern for fish stranding.

15.6.5.3 Fish Passage

The Lake Manitoba and Lake St. Martin Regulation Review Committee recommended that *"if the Province builds a new outlet to Lake Manitoba, it should take that opportunity to design the new outlet in such a way as to provide unrestricted fish passage between Lake Manitoba and*

Lake St. Martin". However, with the exception of Option F, none of the channel options will be operated year round; therefore, none of the Lake Manitoba Channel options are appropriate for providing unrestricted fish passage between the two lakes. As indicated in the previous section, fish access to the channels during operation could hinder rather than help the fish populations by altering migration patterns during spawning. An option that may be more valuable in accomplishing the goal of unrestricted fish passage would be including the improvement for fish passage at the current FRWCS as part of the proposed project, regardless of the Lake Manitoba outlet channel that is selected.

Because none of the Lake Manitoba outlet channel options are appropriate for providing unrestricted fish passage between the two lakes, as noted above, and because the review committee did not require fish passage for the Lake St. Martin outlet channels, the options were not ranked based on this criteria.

15.6.6 Social Environment

Certain options may be viewed as more or less favorable by First Nation communities and other stakeholders. While the project as a whole provides a direct benefit to the communities on the drainage basin by mitigating future flooding issues and subsequent environmental and social impacts, the individual components may not be viewed as such. The most likely First Nation and public concerns associated with this project include land use and ownership, access, and unresolved First Nation litigation claims. This is based on our knowledge of the area and experiences associated with this project to date. The following sections contain a brief overview of these issues and the public engagement process that is proposed to advance the project.

15.6.6.1 Land Use / Ownership

The Lake St. Martin outlet channel is located entirely within crown land while the Lake Manitoba outlet channel options are routed through a mix of private and crown land with Options A and F also located within the Pinaymootang First Nation Land. The land consists of a mix of undeveloped natural forest and wetland vegetation, agricultural fields and residential development. Acquiring the required land for the channels will take these areas away from the current land use while some of the channels may also affect the adjacent land use.

Lake Manitoba channel options A, E and F directly associated with the Fairford River will likely be faced with strong resistance. There are long-term unresolved issues over the operation of the FRWCS and how changes in flow conditions on the Fairford River have caused flooding along the river and in Lake Pineimuta and Lake St. Martin. This flooding has impacted land use and activities including fishing, muskrat trapping, waterfowl hunting, agriculture hay fields, and a multitude of infrastructure (roads, houses, etc.).

Because of the issue of flooding on First Nation land and Lake Pineimuta with the associated land use impacts noted above, Option A and F were ranked the worst of the Lake Manitoba channels, with Option E only slightly better because it required the least amount of land acquisition. In comparison, the remaining options were ranked much higher as they are located within private and crown land and will not have the associated effects of the surrounding land use. Options B and C ranked the best with Option D ranked next best as it required a greater amount of land acquisition.

Reach 1 of the Lake St. Martin outlet channel will result in increased water levels in Big Buffalo Lake and flooding of the surrounding bog. Reach 2 may result in localized flooding along the upper portion of Buffalo Creek but will not affect land use in the area. Both the Johnson Beach and Willow Point Reach 3 options will affect the shoreline of Lake Winnipeg. However, the Dauphin River First Nation community noted a preference to have the Reach 3 outlet located on the east side of Willow Point due to concern that the Johnson Beach outlet would have an adverse effect on the Lake Winnipeg fishery.

15.6.6.2 Access

The Lake Manitoba outlet channel options will disturb access along PTH 6 and potentially several municipal roads during construction. Bridge crossings have been proposed to minimize interruption to existing public access along municipal roads. However, Option A and Option D will each result in the blockage of one existing municipal road requiring rerouting of the current access. In comparison, the Lake St. Martin channel options require construction of an all-season road that will provide access to an area previously inaccessible. The Peguis First Nation may have a concern with the project as the proposed all-season road would be constructed

along the periphery of the Area of Interest for the Peguis First Nation. A likely concern is that the road could provide access to hunters that would otherwise not attempt to hunt in this area.

Option E was ranked the best of the Lake Manitoba channels because it will only cause disruption of the PTH 6 Bridge during construction with no permanent change in access. Options B and F were ranked the next best followed by Option C, as they will also cause temporary disturbance to one and two municipal crossings, respectively, in addition to the PTH 6 Bridge. Whereas, Option A and D were ranked the worst because, in addition to the temporary construction disturbance, they will result in permanent changes to the current access patterns, which will likely be faced by public opposition.

The Lake St. Martin options are all similarly ranked because they all require construction of the all-season road and associated potential change to land use. Reach 1 was ranked slightly higher, as all of the other options required a slightly longer access road providing more access for potential hunters.

15.6.6.3 First Nation Litigation

Overall, the project and the public engagement process may be hindered by an overtone of distrust that was apparent during previous engagement and consultation events within the communities in 2011-2012 associated with the Reach 1 Emergency Outlet Channel. As well, the Pinaymootang, Little Saskatchewan, Lake St. Martin and Dauphin River First Nation communities are currently involved in a class action lawsuit pertaining to the flooding that occurred in the region in 2011. The suit against the Province includes, but is not limited to the following: general non-pecuniary damages for negligence, nuisance, breach of fiduciary duty and breach of or interference with treaty rights, damages for expenses and loss of income, and diminution and loss of property value (Manitoba Justice, 2012). Further to the issues over the 2011 flooding, the Pinaymootang First Nation has an on-going lawsuit with Canada (1961 to the present day) over the fiduciary duties, which the Band suggested was owed to them relating to flooding from operation of the FRWCS. The allegations associated with the lawsuit included the loss of hay lands negatively impacting farming, destruction of the natural habitat negatively impacting the opportunity to engage in hunting, fishing and trapping both on and off the reserve, disease such as dysentery and contamination of the water. The Band further alleged that a

byproduct of the breach of fiduciary responsibility has led to a loss of self-sufficiency by Band members including an increase in alcoholism, drug addiction, mental illness or distress, community strife and an increase in unemployment and dependency on government welfare (Office of the Commissioner for Federal Judicial Affairs Canada, 1998).

Based on the unresolved litigation, the Lake Manitoba channel options A, E and F will be faced with strong resistance as they are directly associated with the Fairford River; and options A and F are located within the Pinaymootang First Nations land, as previously discussed. As such, they were ranked the worst and much lower than the remaining options. While the remaining options could still affect flooding on Lake St. Martin, this issue can be resolved by the Lake St. Martin channel; and the options will not require acquisition of land from the Pinaymootang First Nation. As such, Options A, B and C were all similarly ranked the best.

The Lake St. Martin outlet channel will alleviate flooding on Lake St. Martin without contributing to additional flooding along the Dauphin River and affecting the Dauphin River First Nation. As such, each reach was similarly ranked, although Reach 3 to Johnson Beach was ranked slightly lower because the Dauphin River First Nation preference to have the Reach 3 outlet located on the east side of Willow point may lead to opposition of the Johnson Beach outlet.

15.6.7 Summary of Environmental Concerns

The biophysical and social environmental concerns for each of the Lake Manitoba and Lake St. Martin channel options, as described in the preceding sections has been summarized in Table 15.14. This summary of environmental concerns, indicates the overall ranking of each alternative from a biophysical and social environmental perspective and will be used along with other criteria in the selection of a preferred option.

Table 15.14
Summary of Environmental Concerns

Criteria	Lake Manitoba Outlet Channels						Lake St. Martin Outlet Channels			
	A	B	C	D	E	F	R1	R2	R3JB	R3WP
Biophysical Environment										
Surface Water Quality										
- Flow Rate	6	6	6	6	8	7	9	9	9	9
- Flooding	7	9	9	8	6	7	8	9	9	9
- Erosion/Sedimentation	4	7	7	5	8	1	6	8	5	4
Groundwater	6	6	5	7	8	8	8	8	8	8
Terrestrial Environment										
- Vegetation	7	7	8	6	9	7	9	8	8	7
- Wildlife/Habitat	7	7	8	6	9	7	8	7	7	6
Fish Habitat	6	8	8	7	6	2	6	8	7	7
Fish Resources										
- Habitat Change	5	8	8	7	4	6	8	9	7	7
- Species Composition	5	8	8	7	4	6	5	8	6	6
Subtotal	53	66	67	59	62	51	67	74	66	63
Social Environment										
Land Use/Ownership	2	9	9	8	3	2	8	9	5	7
Access	5	8	7	5	9	8	7	6	6	6
First Nation Litigation	1	8	8	8	1	1	8	8	7	8
Subtotal	8	25	24	21	13	11	23	23	18	21
TOTAL	61	91	91	80	75	62	90	97	84	84

Based on the above ranking summary for the Lake Manitoba channels, Options C and B were the best with A and F clearly the worst from a biophysical environment perspective. Similarly, Options B and C were the best, closely followed by Option D, with options A and F again clearly the worst from a social environment perspective.

For the Lake Manitoba outlet channels, there was no difference between the total ranking for the Johnson Beach and Willow Point options for Reach 3. Johnson Beach was slightly better because it was a shorter option that would require less vegetation clearing with associated

impact to wildlife habitat and would result in potentially less erosion and sedimentation. However, this was balanced by the social concerns related to land use and litigation.

15.6.8 Public Engagement Process

The Provincial licensing process, which considers the extent of public engagement and consultation that occurred prior to submission of an Environment Act Proposal, does not stipulate mandatory consultation at specific points throughout the advancement of a project. Therefore, stakeholder interests are not always effectively included in the early stages of a project. While the current scope of work did not involve developing an Environment Act Proposal, nor did it include a public engagement process, a public open house event specific to the outlet channels was held in September 2014 to present the results of the study and the preferred alternatives. Nevertheless, it is suggested that a public engagement process be included at the early stages of the next phase of study to address social environment concerns. The engagement process should specifically involve the Pinaymootang, Little Saskatchewan, Lake St. Martin, and Dauphin River First Nation communities located along the Fairford River, Lake St. Martin and the Dauphin River, as they comprise the majority of stakeholders likely to be directly impacted by any decisions made pertaining to the proposed project. Consideration should also be given to the potential concerns of the Peguis First Nation related to access (Section 15.6.6.2).

The proposed public engagement process includes three sessions, which will be held at each of the four First Nation communities listed above and will comprise the following:

1. Session 1: Introduction to the Project and initial feedback from the communities.
2. Session 2: Feedback on the preferred alternatives.
3. Session 3: Presentation of the selected design for the Project.

The meeting attendees should include, at a minimum, the Chief and Council, elders, and any interested community members. The Manitoba Aboriginal and Northern Affairs Consultation Group should be involved in the process so the communities can be reassured that their interests are being considered. MIT representatives should also be included to allow the communities to pose questions and concerns directly to the Province.

The Public Engagement process should achieve the following:

- **Opportunities for early involvement** – this includes providing early notice and information about the Project and the engagement process so that all parties can assess their interests and provide early comments.
- **Opportunities for ongoing involvement** – this includes providing ongoing opportunities to learn about the Project, to provide input with respect to any concerns or opinions, to resolve issues raised, to have views and input recorded, and to learn about actions or results that occur as a result of studies and planning activities.
- **Opportunities at various stages** – this includes opportunities to provide input during the three rounds of engagement: a) Introduction to Project; b) Feedback on preferred alternatives; and c) Presentation of selected design.
- **Variety of mechanisms** – this includes a variety of mechanisms (appropriate for different segments of the public) to communicate, to receive feedback and to engage in ongoing meaningful dialogue.
- **Adaptive approach** – this includes adjusting the engagement process, as required and feasible, throughout the course of the environmental review and planning process, in response to issues, concerns and challenges.

15.7 PREFERRED ALTERNATIVE SELECTION

To aide in the selection of the preferred alternative for the Lake Manitoba and Lake St. Martin outlet channels, an estimate of costs required to develop each channel option was prepared as described in Section 15.5; and a preliminary screening of potential biophysical and social environmental concerns for each option was completed as described in Section 15.6.

The recommended capacity for the Lake Manitoba outlet channels was in part based on the results of the economic analyses and the basin-wide evaluation of the options later discussed in Sections 22.0 and 23.0 of the report. Outlet channel capacities from zero (do nothing) to 425 m^3/s (15,000 cfs) have been considered for Lake Manitoba.

As a minimum, the Lake St. Martin Emergency Outlet Channel should be made permanent with a base capacity of 113 m^3/s (4000 cfs). Additionally, if a Lake Manitoba outlet channel is constructed, a similarly-sized Lake St. Martin outlet channel would be required to ensure that flooding would not be aggravated on Lake St. Martin (i.e., base capacity plus Lake Manitoba channel capacity). Therefore, outlet channel capacities from 113 m^3/s (4000 cfs) to 540 m^3/s (19,000 cfs) have been considered for Lake St. Martin.

With the exception of Option E, which has a maximum capacity of 106 m^3/s (3750 cfs), all outlet channel options can provide the complete range of the hydraulic capacity considered. Therefore, selection of a preferred alternative was a function of the route associated with each option but independent of the various channel capacities and the corresponding averted damages. The selection of a channel capacity was based on the results of the economic analyses and the basin-wide evaluation of the options, with consideration of the overall funding commitments for flood mitigation in the Province.

15.7.1 Lake Manitoba Outlet Channel

The following is a summary of the key selection criteria for the Lake Manitoba outlet channels.

Estimated Cost – Option E has the lowest estimated cost by a substantial margin, as shown in Table 15.13 in Section 15.5, but it is the only option that has a limited hydraulic capacity. For the outlet channels that considered greater hydraulic capacities, Option A was nearly half the cost compared to the other options. The remaining options from least expensive to most were Options D, C, B and F.

Biophysical Environment – As summarized in Table 15.14 in Section 15.6.7, Options C and B have the highest ranking, followed by E and D. Options A and F are clearly ranked the worst from a biophysical environment perspective.

Social Environment – As summarized in Table 15.14 in Section 15.6.7, Options B and C have the highest ranking, followed closely by Option D. Options E, F and A are clearly ranked the lowest from a social environment perspective.

Hydraulic Capacity – With the exception of Option E, which has a maximum capacity of 106 m^3/s (3,750 cfs), all options have the same ranking because they meet all the various capacity requirements.

Considering the above, the Lake Manitoba outlet channel options have been listed in order of preference, as follows:

Option C – Channel slightly south of Pinaymootang First Nation has a moderate cost ($168 million for 142 m^3/s (5,000 cfs)) relative to other options that could meet all the various capacity requirements but has the highest biophysical and social environment ranking.

Option B – Channel south of Pinaymootang First Nation has a moderate cost ($187 million for 142 m^3/s (5,000 cfs)) relative to other options that could meet all the various capacity requirements and a very high biophysical and social environment ranking. However, Option C essentially supersedes Option B, as it has very similar characteristics; but it has been optimized with respect to cost.

Option D – Channel following Birch Creek has a lower cost ($158 million for 142 m^3/s (5,000 cfs)) relative to other options that could meet all the various capacity requirements but also has a lower biophysical and social environment ranking than Options B and C.

Option E – Bypass channel north of the FRWCS has the lowest cost ($30 million for 106 m^3/s (3,750 cfs)) by a substantial margin but is not preferred because the hydraulic capacity is limited; it increases the downstream water levels; and it has a low social environment ranking. However, Option E should still be considered in an emergency situation to supplement the capacity of one of the other Lake Manitoba outlet options (i.e., only used during extreme flood events).

Option A – Twinning the Fairford River has the lowest cost ($101 million for 142 m^3/s (5,000 cfs)) relative to other options that could meet all the various capacity requirements. However, it is not considered viable due to the very low biophysical and social environment ranking.

Option F – Expansion of the Fairford River and the FRWCS (including dredging of the inlet channel) has the highest cost ($268 million for 142 m^3/s (5,000 cfs)) out of all the options by a substantial margin. It also has a very low biophysical and social environment ranking and, therefore, was not considered viable.

15.7.2 Lake St. Martin Outlet Channel

Since only one route was considered for both Reach 1 and Reach 2, the two possible Lake St. Martin outlet channel options being evaluated include:

- Option JB – Reach 1, Reach 2 and Reach 3 to Johnson Beach.
- Option WP – Reach 1, Reach 2 and Reach 3 east of Willow Point.

The following is a summary of the key selection criteria for the Lake St. Martin outlet channels.

Estimated Cost – The outlet channel option with Reach 3 to Johnson Beach was approximately 8% lower in cost compared to the option with Reach 3 to Willow Point, as shown in Table 15.13 in Section 15.5.

Biophysical Environment – As summarized in Table 15.14 in Section 15.6.7, the outlet channel option for Reach 3 to Johnson Beach was ranked slightly higher than the option for Reach 3 to Willow Point from a biophysical environment perspective.

Social Environment – As summarized in Table 15.14 in Section 15.6.7, the outlet channel option for Reach 3 to Johnson Beach was ranked slightly lower than the option for Reach 3 to Willow Point from a social environment perspective.

Hydraulic Capacity – Both options have the same ranking because they both meet all the various capacity requirements.

Considering the above, the Lake St. Martin outlet channel options have been listed in order of preference as follows:

Option-WP – Reach 1, Reach 2 and Reach 3 east of Willow Point have higher social environment rankings and, therefore, are preferred. However, Option WP also has a lower biophysical environment ranking and a slightly higher estimated cost ($168 million for 255 m^3/s (9,000 cfs)).

Option JB – Reach 1, Reach 2 and Reach 3 to Johnson Beach has a lower estimated cost ($180 M for 255 m^3/s (9,000 cfs)). It also has a higher biophysical environment ranking, but it has a lower social environment ranking.

Note that the differences in the selection criteria for the Lake St. Martin outlet channel options are subtle. Therefore, the order of preference could be altered if the estimated costs change during the next phase of design or if more detailed studies suggest changes to the environmental rankings for the options.

15.8 CONCLUSIONS

Six Lake Manitoba outlet channel options and two Lake St. Martin outlet channel options were developed at this conceptual design stage. Outlet channel capacities from zero (do nothing) to 425 m^3/s (15,000 cfs) have been considered for Lake Manitoba. Outlet channel capacities from 113 m^3/s (4000 cfs) to 540 m^3/s (19,000 cfs) have been considered for Lake St. Martin. The following items can be concluded based on the current study results.

15.8.1 Lake Manitoba

- A Monte Carlo analysis was determined to be an acceptable method to generate combined flood volumes for the Assiniboine River and local Lake Manitoba inflow given the limited record of flows and the limited information on the correlation.
- For the 200-year event, the peak still water level on Lake Manitoba without an outlet channel is approximately 248.9 m (816.6 ft).
- For the 200-year event, a 142 m^3/s (5,000 cfs) and a 283 m^3/s (10,000 cfs) outlet channel would reduce the peak Lake Manitoba level by 0.24 m (0.8 ft) and 0.43 m (1.4 ft), respectively.
- Option E has the lowest cost ($30 million for 3,750 cfs) by a substantial margin; but the hydraulic capacity is limited; and it increases downstream water levels locally.
- Costs for LMB outlet channels that considered greater hydraulic capacities ranged from $101 million (Option A for 5,000 cfs) to $462 million (Option B for 15,000 cfs).
- An outlet channel capacity sufficient to avoid flooding due to a 2011-sized event would be greater than 425 m^3/s (15,000 cfs) and, therefore, would not be justifiable due to the high cost.

- Option B should not be considered further, as it has very similar characteristics to Option C, but has a higher cost (Option C is an optimization of Option B).
- The combined biophysical and social environment rankings of the options were the lowest for Option A (61 out of 120) and the highest for Option C (91 out of 120).
- Options A and F should not be considered further due to the significant environmental and social concerns.
- Each LMB outlet channel option is viable from a constructability perspective.
- Each LMB outlet channel option should incorporate a control structure to regulate flows.
- Fish passage is not feasible within the outlet channels because they would not be continuously operated; therefore, all options should consider improvement of fish passage directly at the FRWCS.

15.8.2 Lake St. Martin

- Costs for LSM outlet channels ranged from $107 million (Option R-JB for 4,000 cfs) to $330 million (Option R-WP for 19,000 cfs).
- Each LSM outlet channel option is viable from a constructability perspective.
- Each LSM option should incorporate a control structure at the inlet to Reach 1 to regulate the outlet channel flows.
- All LSM outlet channel options should incorporate a permanent access road to facilitate construction, maintenance and operation.
- Utilizing the lower portion of Buffalo Creek on a permanent basis is not recommended due to environmental impacts, increased potential for flooding of the Dauphin River communities and opposition from stakeholders.
- The combined biophysical and social environment rankings of the options were equivalent for both Option R-JB and Option R-WP (average of 90 out of 120). However, Option R-WP is preferred due to its higher social environment concerns.

15.9 RECOMMENDATIONS

Based on the concept designs, the estimated costs and environmental assessment, the following recommendations have been developed:

15.9.1 Lake Manitoba

- Options C and D should be considered for further review as part of Conceptual Design Stage 2 – Detailed Review of Preferred Alternatives.

- Channel capacities in the range of 142 to 283 m^3/s (5,000 to 10,000 cfs) should be considered for further review since lesser capacities will not adequately reduce lake levels and greater capacities are not justifiable due to the high cost. Based on the results of the study and available funds, the Province committed to a 212 m^3/s (7,500 cfs) capacity in the fall of 2014.
- Option E should be considered for further review in an emergency situation to supplement the capacity of one of the other Lake Manitoba outlet options (i.e., only used during extreme flood events). However, this option would take additional effort as it would require its own environmental approval, and has environmental and social impacts on Lake Pineimuta and First Nations due to increased flows.

15.9.2 Lake St. Martin

- The Lake St. Martin Emergency Outlet Channel should be made permanent with a base capacity of 113 m^3/s (4,000 cfs) to control Lake St. Martin levels.
- Additional capacity in the range of 142 to 283 m^3/s (5,000 to 10,000 cfs) should be considered for further review to match a Lake Manitoba Outlet Channel to ensure that flooding will not be aggravated on Lake St. Martin. Based on the results of the study and available funds, the Province committed to a 326 m^3/s (11,500 cfs) capacity in the fall of 2014.
- Options for the Reach 3 outlet at Johnson Beach and east of Willow Point have similar rankings and, therefore, both should proceed to Stage 2 for further review.

15.9.3 Conceptual Design - Stage 2

The scope of the Conceptual Design Stage 2 – Detailed Review of Preferred Alternatives should include the following:

- Optimization of the specific basis of design and the hydraulic characteristics for the preferred options identified in Stage 1.
- Geotechnical field investigations (peat probes, test pits, boreholes, monitoring wells, laboratory testing and geo-seismic) will be required at the proposed structure locations, along the Reach 3 route east of Willow Point and permanent access road routes.
- Survey investigation will be required at the proposed structure locations, along the Reach 3 route east of Willow Point and permanent access road routes.
- Hydrodynamic modeling and analysis of ice conditions will be developed further for each of the preferred options.
- Costs and quantities (land acquisition, excavation, bridges, municipal roads, control structures, etc.) developed in Stage 1 will be re-evaluated and updated for the preferred options. Detailed descriptions will be developed for each of the required infrastructure components.

- Environmental impacts (water quality, terrestrial vegetation and wildlife, groundwater, etc.) of the preferred options will be assessed in greater detail for the preferred options and the environmental assessment requirements for future licensing will be identified.
- Methods for improved fish passage between Lake Manitoba and Lake St. Martin at the FRWCS will be identified.
- Social impacts of the preferred options will be assessed in greater detail based on feedback received during the Stage 1 public consultation process. An additional public engagement / consultation is anticipated in the Stage 2 scope.

The focus of Stage 2 will be to complete the conceptual design of the preferred alternatives. This would enable the next phase of design to proceed to the detailed design stage.

TABLES

Design Criteria For Outlet Channels On Lake Manitoba and Lake St. Martin													
Design Alternatives	**Design Flow (cfs)**	**Material**	**Base Width (m)**	**Channel Side Slopes (H:V)**	**Channel Roughness (n)**	**Channel Length (m)**	**Channel Slope (%)**	**Average Flow Velocity (m/s)**	**Average Depth of Flow (m)**	**Average Depth of Excavation (m)**	**Average Top Width (m)**	**Design Lake Manitoba Level (ft)**	**Design Lake St Martin Level (ft)**
A1	5000	Till	26	4:1	0.028	12300	0.032	1.1	3.5	4.7	64	814	-
A2	10000	Till	64					1.1	3.5		83		
A3	15000	Till	112					1.2	3.5		131		
B1	5000	Till	31	4:1	0.028	5760	0.013	1.0	3.4	8.5	82	814	-
		Rock	47	1:4	0.032	5740	0.013	0.9	3.4				
B2	10000	Till	72	4:1	0.028	5760	0.013	1.0	3.4		125		
		Rock	92	1:4	0.032	5740	0.013	0.9	3.4				
B3	15000	Till	113	4:1	0.028	5760	0.013	1.0	3.4		167		
		Rock	136	1:4	0.032	5740	0.013	0.9	3.4				
C1	5000	Till	31	4:1	0.028	5470	0.013	1.0	3.4	7.9	79	814	-
		Rock	47	1:4	0.032	6090	0.013	0.9	3.4				
C2	10000	Till	72	4:1	0.028	5470	0.013	1.0	3.4		122		
		Rock	92	1:4	0.032	6090	0.013	0.9	3.4				
C3	15000	Till	113	4:1	0.028	5470	0.013	1.0	3.4		165		
		Rock	136	1:4	0.032	6090	0.013	0.9	3.4				
D1	5000	Till	27	4:1	0.028	22800	0.013	0.5	4.0	5.5	72	814	-
D2	10000		64					0.6			108		
D3	15000		100					0.7			144		
E1	2500	Till	9	4:1	0.028	2150	0.030	0.9	3.5	5.4	54	814	-
E2	3750		21					0.9	3.5		65		
F1	18500	Till	40 to 150	4:1	0.025 to 0.031	18700	n/a	0.8	2.5	3.0	n/a	814	-
R1 (Base)	4000	Till	45	3:1	0.028	6000	0.020	0.9	2.8	4.2	70	-	801
R1+5000	9000		81								106		
R1+10000	14000		128								153		
R1+15000	19000		176								201		
R2 (Base)	4000	Till	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	-	n/a
R2+5000	9000		10	3:1	0.028	2900	0.013	0.5	7.0	5.0	35		
R2+10000	14000		80	3:1	0.028	2900	0.013	0.5	7.0	5.0	105		
R2+15000	19000		140	3:1	0.028	2900	0.013	0.5	7.0	5.0	165		
R3C (Base)	4000	Till	21	3:1	0.028	8040	0.034	1.1	3.5	3.0	39	-	n/a
		Rock		1:4	0.032	1660	0.074	1.6	2.0				
R3C+5000	9000	Till	52	3:1	0.028	8040	0.034	1.1	3.5	3.0	70		
		Rock		1:4	0.032	1660	0.074	1.6	2.0				
R3C+10000	14000	Till	82	3:1	0.028	8040	0.034	1.1	3.5	3.0	101		
		Rock		1:4	0.032	1660	0.074	1.6	2.0				
R3C+15000	19000	Till	113	3:1	0.028	8040	0.034	1.1	3.5	3.0	131		
		Rock		1:4	0.032	1660	0.074	1.6	2.0				
R3E (Base)	4000	Till	21	3:1	0.028	9760	0.034	1.1	3.5	3.4	41	-	n/a
		Rock		1:4	0.032	1660	0.074	1.6	2.0				
R3E+5000	9000	Till	52	3:1	0.028	9760	0.034	1.1	3.5	3.4	72		
		Rock		1:4	0.032	1660	0.074	1.6	2.0				
R3E+10000	14000	Till	82	3:1	0.028	9760	0.034	1.1	3.5	3.4	103		
		Rock		1:4	0.032	1660	0.074	1.6	2.0				
R3E+15000	19000	Till	113	3:1	0.028	9760	0.034	1.1	3.5	3.4	133		
		Rock		1:4	0.032	1660	0.074	1.6	2.0				

1) Design Alternative

Lake Manitoba Options:

Option A - Twinning the Fairford River

Option B - Channel South of Pinaymootang First Nation

Option C - Channel Slightly less south of Pinaymootang First Nation

Option D - Channel Following Birch Creek

Option E - Bypass channel north of the FRWCS

Option F - Expansion of the Fairford River & FRWCS (including dredging of the inlet channel)

Lake St. Martin Reaches

Reach 1 (R1) - Re-opening and/or expansion of Reach 1

Reach 2 (R2) - Expansion of Buffalo Creek (Reach 2) downstream of Buffalo Lake

Reach 3 (R3-JB) - Completion and/of expansion of Reach 3 to Lake Winnipeg at Johnson Beach

Reach 3 (R3-WP) - Completion and/or expansion of Reach 3 to lake Winnipeg east of Willow Point

Lake St. Martin Combined Options:

Option R123-JB - Reach 1, Reach 2, and Reach 3 to Johnson Beach

Option R123-WP - Reach 1, Reach 2, and Reach 3 east of Willow Point

TABLE 15.1: DESIGN CRITERIA FOR OUTLET CHANNELS ON LAKE MANITOBA LAKE ST. MARTIN (JAN. 23, 2014)
KGS: 12-0300-011
ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

Design Alternative[*1]	Quantities										Costs																	
	Existing Capacity (cfs)	Proposed Capacity (cfs)	Channel Width {till} (m)	Channel Width {rock} (m)	Earth Excavation (Mil Cu. m)	Rock Excavation (Mil.Cu.m)	Riprap/ Drop Struct (Mil.Cu.m)	Diking (Mil.Cu.m)	Land Acquisition (ha)	Permanent Access Rd (Km)	Bridge/Control Structure (Mil CAD)	DS Control Structure (Mil CAD)	Municipal Bridges (Mil CAD)	FRWCS Fish Passage (Mil CAD)	Earth Excavation (Mil CAD)	Rock Excavation (Mil CAD)	RIPRAP/ Drop Struct (Mil CAD)	Diversion Structure (Mil CAD)	Diking (Mil CAD)	Land Acquisition (Mil CAD)	Permanent Access Rd (Mil CAD)	Rail Bridge Removal (Mil CAD)	Construction Camp (Mil CAD)	Revegetate Channel (Mil CAD)	Mitigation Costs (Mil CAD)	5% Misc. Allowance (Mil CAD)	Subtotal Cost (Mil CAD)	Total + 20%Eng/ Approv Fees (Mil CAD)	Total + 30% Contingency (Mil CAD)
A1	NA	5000	26.00	--	2.63	--	--	--	204	--	$7.3	$2.7	$6.9	$2.0	$31.6	--	--		--	$1.0	--	--	$2.2	$2.3	$5.6	$3.1	$64.7	$77.6	$100.9
A2	NA	10000	64.00	--	4.83	--	--	--	251	--	$11.6	$5.4	$9.0	$2.0	$58.0	--	--		--	$1.3	--	--	$3.6	$2.9	$9.4	$5.2	$108.2	$129.8	$168.8
A3	NA	15000	112.00	--	7.61	--	--	--	310	--	$14.7	$7.3	$14.1	$2.0	$91.3	--	--		--	$1.5	--	--	$5.4	$3.6	$14.0	$7.7	$161.7	$194.0	$252.2
B1	NA	5000	31.00	47.30	4.13	1.980	--	--	197	--	$6.0	--	$2.2	$2.0	$41.3	$49.5	--		--	$1.0	--	--	$4.2	$2.3	$5.4	$5.7	$119.6	$143.5	$186.5
B2	NA	10000	72.10	91.80	6.85	3.770	--	--	244	--	$9.0	--	$3.4	$2.0	$68.5	$94.3	--		--	$1.2	--	--	$7.2	$2.8	$9.4	$9.9	$207.8	$249.3	$324.1
B3	NA	15000	113.20	136.30	9.58	5.560	--	--	291	--	$12.1	--	$4.5	$2.0	$95.8	$139.0	--		--	$1.5	--	--	$10.3	$3.3	$13.4	$14.1	$296.0	$355.2	$461.8
C1	NA	5000	31.00	47.30	3.47	1.740	--	--	198	--	$6.0	--	$4.3	$2.0	$34.7	$43.5	--		--	$1.0	--	--	$3.8	$2.3	$4.9	$5.1	$107.5	$129.0	$167.7
C2	NA	10000	72.10	91.80	5.84	3.320	--	--	246	--	$9.0	--	$6.6	$2.0	$58.4	$83.0	--		--	$1.2	--	--	$6.5	$2.8	$8.5	$8.9	$187.0	$224.4	$291.7
C3	NA	15000	113.20	136.30	8.18	4.900	--	--	294	--	$12.1	--	$8.9	$2.0	$81.8	$122.5	--		--	$1.5	--	--	$9.3	$3.4	$12.1	$12.7	$266.2	$319.4	$415.2
D1	NA	5000	27.30	--	6.29	--	--	--	381	--	$7.7	--	$7.8	$2.0	$62.9	--	--		--	$1.9	--	--	$3.5	$4.4	$6.3	$4.8	$101.3	$121.5	$158.0
D2	NA	10000	63.70	--	10.86	--	--	--	464	--	$12.2	--	$11.7	$2.0	$108.6	--	--		--	$2.3	--	--	$5.7	$5.3	$10.3	$7.9	$166.0	$199.3	$259.0
D3	NA	15000	99.50	--	15.37	--	--	--	546	--	$17.4	--	$15.6	$2.0	$153.7	--	--		--	$2.7	--	--	$7.9	$6.3	$14.4	$11.0	$230.9	$277.1	$360.3
E1	NA	2500	9.10	--	0.37	--	--	--	32	--	$5.9	--	--	$2.0	$3.7	--	--		--	$0.2	--	--	$0.5	$0.4	$1.0	$0.7	$14.2	$17.0	$22.2
E2	NA	3750	20.60	--	0.50	--	--	--	35	--	$8.7	--	--	$2.0	$5.0	--	--		--	$0.2	--	--	$0.7	$0.4	$1.4	$0.9	$19.2	$23.1	$30.0
F	13500	18500	95.00	--	4.88	--	--	--	336	--	$3.3	$11.2	$3.5	$2.0	$111.8	--	--		--	$1.7	--	$1.0	$5.5	$2.2	$21.3	$8.2	$171.6	$205.9	$267.6
R1 Base	4000	4000	45.00	--	0.55	--	--	--	--	80	$6.3	--	--		$5.5	--	--		--	--	$20.4	--	$2.7	$1.3	$1.8	$1.9	$39.8	$47.8	$62.1
R1 +5000	4000	9000	80.55	--	1.46	--	--	--	--	80	$11.2	--	--		$14.6	--	--		--	--	$20.4	--	$3.8	$1.5	$2.6	$2.7	$56.8	$68.1	$88.6
R1 +10000	4000	14000	128.40	--	2.68	--	--	--	--	80	$17.5	--	--		$26.8	--	--		--	--	$20.4	--	$5.3	$1.9	$3.6	$3.8	$79.2	$95.0	$123.5
R1 +15000	4000	19000	176.20	--	3.90	--	--	--	--	80	$21.7	--	--		$39.0	--	--		--	--	$20.4	--	$6.7	$2.2	$4.5	$4.7	$99.1	$118.9	$154.6
R2 Base	NA	4000	0.00	--	0.01	--	--	0.03	--	--		--	--		$0.1	--	$2.0		$1.0	--	$0.0	--	$0.3	$0.3	$0.1	$0.2	$3.8	$4.6	$6.0
R2 +5000	NA	9000	10.00	--	0.02	--	--	0.07	--	3.7		--	--		$0.2	--	$2.0		$2.0	--	$2.8	--	$0.6	$0.4	$0.3	$0.4	$8.6	$10.4	$13.5
R2 +10000	NA	14000	80.00	--	0.76	--	--	0.11	--	3.7		--	--		$7.6	--	$2.0		$3.0	--	$2.8	--	$1.3	$0.5	$0.7	$0.9	$18.6	$22.4	$29.1
R2 +15000	NA	19000	140.00	--	1.31	--	--	0.16	--	3.7		--	--		$13.1	--	$2.0		$4.0	--	$2.8	--	$1.8	$0.6	$1.0	$1.3	$26.4	$31.7	$41.2
R3-JB Base	4000	4000	21.00	21.00	0.36	0.001	0.01	--	--	16		--	--		$3.6	$0.0	$0.6	$2.5	--	--	$12.0	--	$1.6	$1.8	$1.3	$1.2	$24.6	$29.5	$38.4
R3-JB +5000	4000	9000	51.90	51.90	1.47	0.032	0.03	--	--	16		--	--		$14.7	$0.8	$2.8	$2.5	--	--	$12.0	--	$2.8	$2.1	$2.3	$2.0	$42.1	$50.5	$65.6
R3-JB +10000	4000	14000	82.40	82.40	2.35	0.059	0.06	--	--	16		--	--		$23.5	$1.5	$5.1	$2.5	--	--	$12.0	--	$3.8	$2.5	$3.1	$2.7	$56.6	$67.9	$88.3
R3-JB +15000	4000	19000	112.70	112.70	3.22	0.086	0.08	--	--	16		--	--		$32.2	$2.1	$7.5	$2.5	--	--	$12.0	--	$4.7	$2.8	$3.8	$3.4	$71.0	$85.3	$110.8
R3-WP Base	4000	4000	21.00	21.00	0.69	0.001	0.01	--	--	16		--	--		$6.9	$0.0	$0.6	$2.5	--	--	$12.0	--	$1.9	$2.1	$1.6	$1.4	$29.0	$34.8	$45.2
R3-WP +5000	4000	9000	51.90	51.90	2.11	0.029	0.03	--	--	16		--	--		$21.1	$0.7	$2.8	$2.5	--	--	$12.0	--	$3.3	$2.5	$2.7	$2.4	$50.0	$60.1	$78.1
R3-WP +10000	4000	14000	82.40	82.40	3.27	0.072	0.06	--	--	16		--	--		$32.7	$1.8	$5.1	$2.5	--	--	$12.0	--	$4.6	$2.9	$3.7	$3.3	$68.6	$82.3	$107.0
R3-WP +15000	4000	19000	112.70	112.70	4.43	0.083	0.08	--	--	16		--	--		$44.3	$2.1	$7.5	$2.5	--	--	$12.0	--	$5.7	$3.3	$4.6	$4.1	$86.1	$103.3	$134.3

Estimated Construction Quantities and Costs of Outlet Channel Options

Design Alternative	Subtotal Cost (Mil CAD)	Total + 20%Eng/ Approv Fees (Mil CAD)	Total + 30% Contingency (Mil CAD)
Option JB Base	$68.2	$81.9	$106.5
Option JB +5000	$107.5	$129.0	$167.6
Option JB +10000	$154.4	$185.3	$240.9
Option JB +15000	$196.5	$235.8	$306.6
Option WP Base	$72.6	$87.2	$113.3
Option WP +5000	$115.4	$138.5	$180.1
Option WP +10000	$166.4	$199.7	$259.6
Option WP +15000	$211.6	$253.9	$330.1

Lake St. Martin Combined Options

1) Design Alternative

Lake Manitoba Options:

Option A - Twinning the Fairford River
Option B - Channel South of Pinaymootang First Nation
Option C - Channel Slightly less south of Pinaymootang First Nation
Option D - Channel Following Birch Creek
Option E - Bypass channel north of the FRWCS
Option F - Expansion of the Fairford River & FRWCS (including dredging of the inlet channel)

Lake St. Martin Reaches

Reach 1 (R1) - Re-opening and/or expansion of Reach 1
Reach 2 (R2) - Expansion of Buffalo Creek (Reach 2) downstream of Buffalo Lake
Reach 3 (R3-JB) - Completion and/of expansion of Reach 3 to Lake Winnipeg at Johnson Beach
Reach 3 (R3-WP) - Completion and/or expansion of Reach 3 to lake Winnipeg east of Willow Point

Lake St. Martin Combined Options:

Option JB - Reach 1, Reach 2, and Reach 3 to Johnson Beach
Option WP - Reach 1, Reach 2, and Reach 3 east of Willow Point

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TABLE 15.11: ESTIMATED CONSTRUCTION QUANTITIES AND COSTS REV.2 K
KGS: 12-0300-011
ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

DRAWINGS

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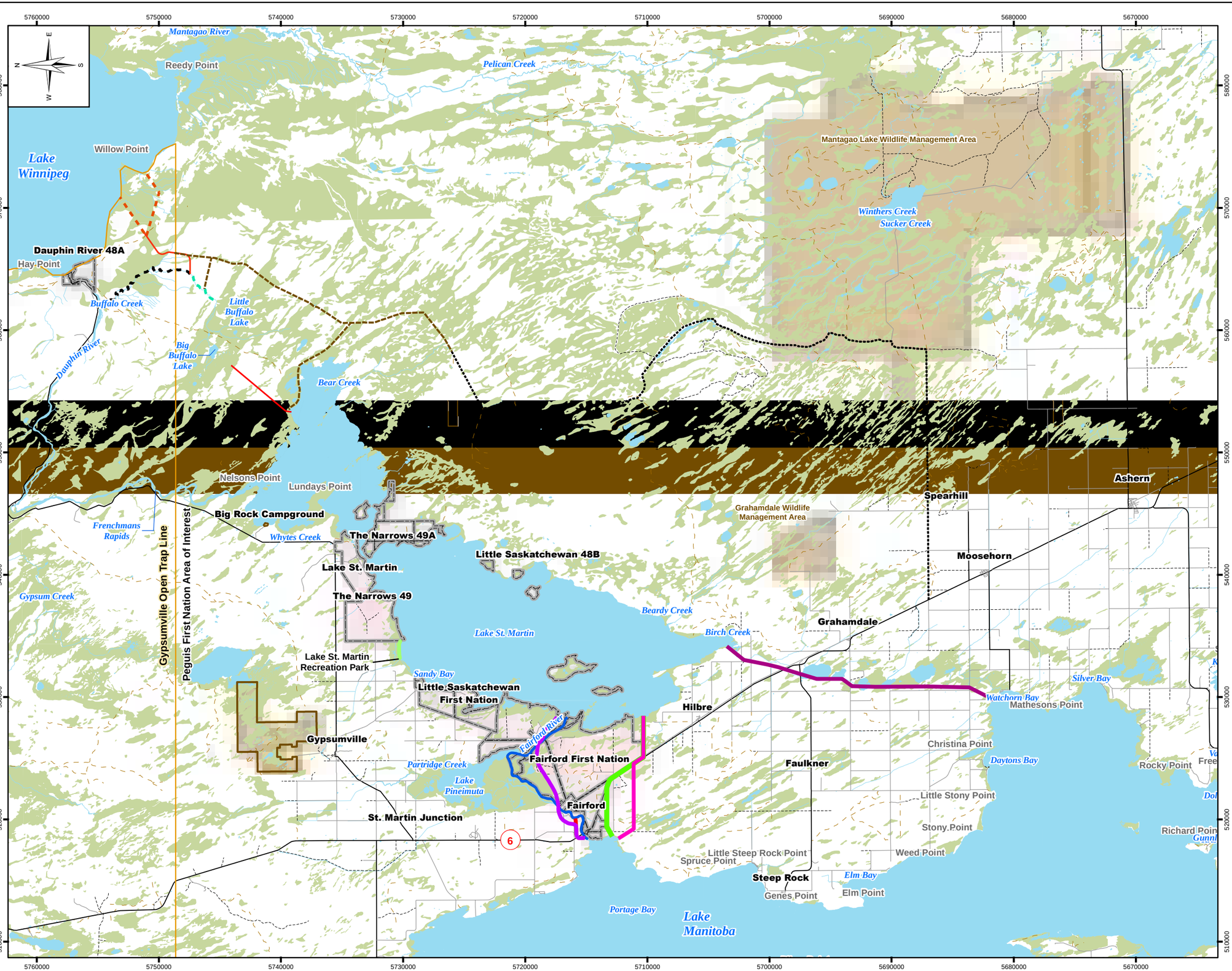

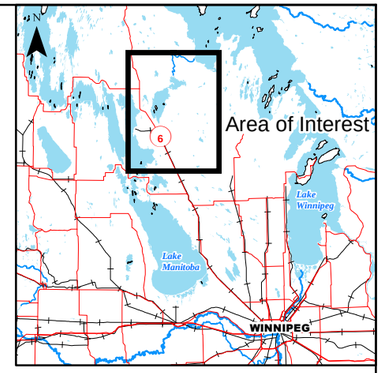

LEGEND:

- Forestry Road to be Upgraded
- Winter Road to be Upgraded
- Channel Option A
- Channel Option B
- Channel Option C
- Channel Option D
- Fairford Bypass Option E
- Fairford River Option F
- Buffalo Creek Drainage System
- Reach 1 (As Constructed)
- Reach 2 (Proposed)
- Reach 3 Emergency Channel (As Constructed)
- Reach 3 Johnson Beach Outlet (Proposed)
- Reach 3 Willow Point Outlet (Proposed)
- Municipal Road
- Highway
- Limited Use Road
- Trail
- Watercourse
- Wetlands
- Waterbody
- First Nation
- Wildlife Management Area
- Gypsumville Open Trap Line
- Campsite

2 0 2 4 6 8 10
Kms

SCALE: 1:300,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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5720000 5715000 5710000

525000

520000

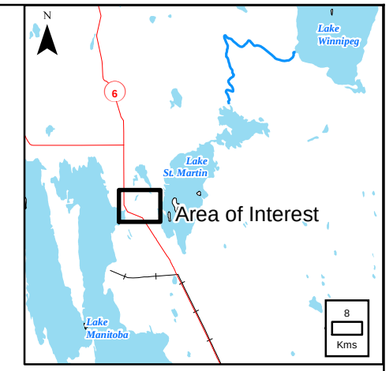
Lake St. Martin

Lake Pineimuta

Fairford River

Lake Manitoba

6



LEGEND:

- Combined Control Structure and PTH Bridge
- Municipal Road & Control Structure
- Municipal Road Bridge
- Channel Option A
- Channel Option B
- Channel Option C
- Fairford Bypass Option E
- Fairford River Option F
- Road Paved 2 or more lanes
- All Season Gravel Road
- Paved Street or Road
- Gravel Road
- Unclassified Road
- Accessway or Backlane
- Trail
- Railway
- Rail Bridge (To Be Removed)
- First Nation

500 0 500 1,000 1,500 2,000

Meters

SCALE: 1:50,000 METRIC 11"x17"

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Transverse Mercator Projection, NAD 1983, Zone 14
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Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

LAKE MANITOBA CHANNEL OPTIONS A, B, C, E, F

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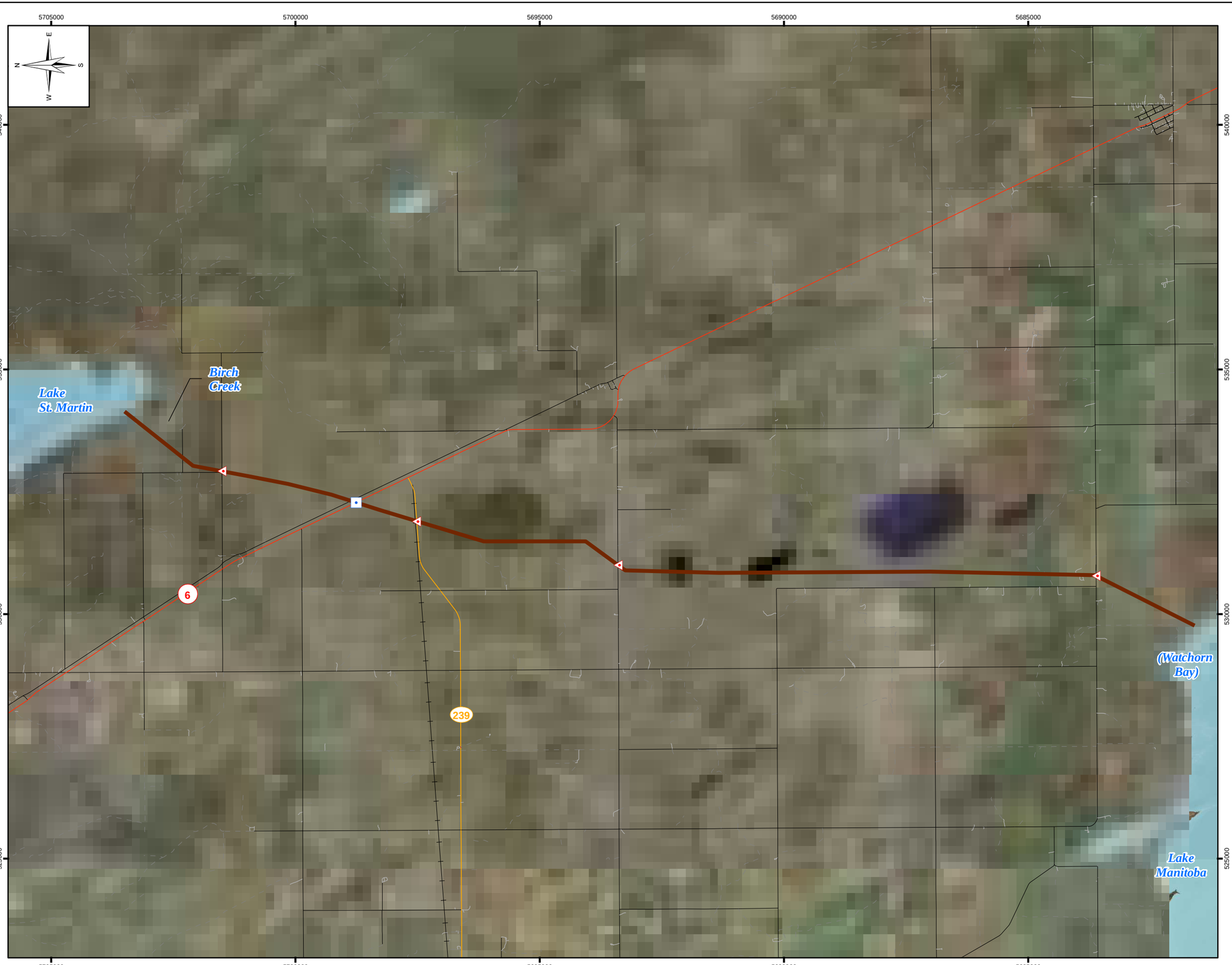

LEGEND:

- Combined Control Structure and PTH Bridge
- Municipal Road Bridge
- Channel Option D
- Road Paved 2 or more lanes
- All Season Gravel Road
- Paved Street or Road
- Gravel Road
- Unclassified Road
- Accessway or Backlane
- Trail
- Railway
- First Nation

1,000 0 1,000 2,000 3,000

Meters

SCALE: 1:75,000 METRIC 11"x17"

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Manitoba INFRASTRUCTURE

LAKE MANITOBA CHANNEL

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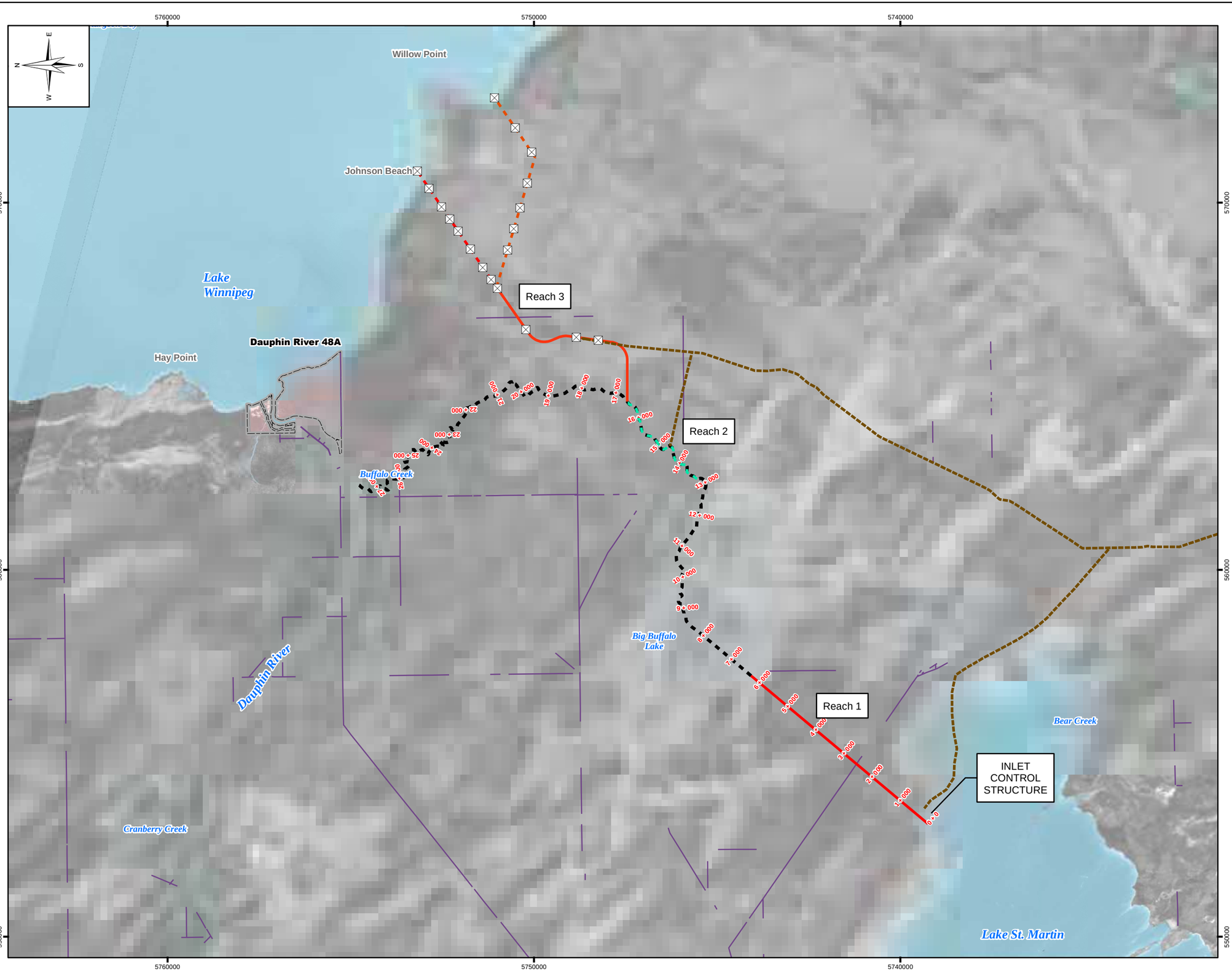

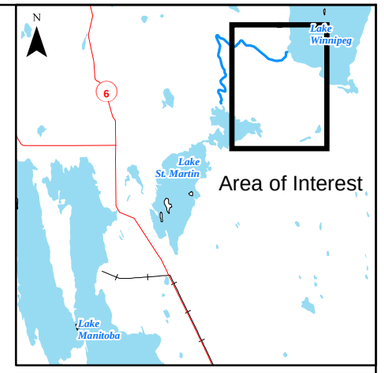

LEGEND:

- Drop Structure
- Buffalo Creek Drainage System
- Reach 1 Emergency Channel (As Constructed)
- Reach 2 (Proposed)
- Reach 3 Emergency Channel (As Constructed)
- Reach 3 Johnson Beach Outlet (Proposed)
- Reach 3 Willow Point Outlet (Proposed)
- Winter Road to be Upgraded
- Cutline (Established 1996)
- Treeline
- 5m Index Contour
- 1m Contour
- Municipal Road
- Highway
- Limited Use Road
- Trail
- Watercourse
- Waterbody
- First Nation

1 0 1 2 3 4
Kms

SCALE: 1:100,000 METRIC 11"x17"

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Elevations are in metres above sea level (MSL)

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16.0 SHOAL LAKES

The Shoal Lakes are located within the southwestern portion of Manitoba's Interlake Region approximately 15 km (9 mi) east of Lake Manitoba. Due to the topographic relief surrounding the Shoal Lakes, the system is generally a land-locked drainage basin with no natural outlet for the lakes except when water levels are extremely high. As a result, during periods with higher than average rainfall, the water levels in the lakes rise. Conversely, in times of drought, the levels in the lakes recede. A location plan showing the Shoal Lakes watershed is provided on Drawing 16.1 (located at the end of Section 16.0).

In recent years, the water levels on the Shoal Lakes have risen to record levels, resulting in flooding of the adjacent lands. This led to KGS Group being retained by Manitoba Water Stewardship (MWS) to undertake a watershed study of options for alleviating flooding issues within the Shoal Lakes Watershed. It included the development and assessment of flood mitigation measures for the lands surrounding the Shoal Lakes. A summary of the alternatives investigated and of the results of the economic analysis from the October 2010 Final Report (KGS Group, 2010) is included in Section 16.1.

In the context of the Assiniboine River & Lake Manitoba Basins Flood Mitigation Study, the potential effects on Lake Manitoba water levels from implementing the flood mitigation alternatives investigated in the Shoal Lakes Watershed Study have been quantified and are summarized in Section 16.2.

16.1 SHOAL LAKES WATERSHED STUDY

The Shoal Lakes Watershed Study conducted by KGS Group investigated three alternative measures to mitigate flooding issues within the lakes. These alternatives have been described below.

1. Construction of a diversion / outlet channel from the Shoal Lakes to Lake Manitoba.

   The purpose of the diversion/outlet channels would be to divert excess flood waters from the Shoal Lakes to Lake Manitoba when the water levels in the lakes exceeded the maximum desirable lake level (target level).

Two alternate alignments of the diversion channel were defined by MWS and have been shown on Drawing 16.1. They include:

- A diversion channel along the alignment of Wagon Creek connecting to the north end of West Shoal Lake (there are two alternative configurations based on the connection of the channel to West Shoal Lake).
- A diversion channel along the alignment of both Roy's Drains and Boundary Drain connecting to the west end of West Shoal Lake.

An assessment of the flood diversion channels with a numerical model indicated that the lake levels would be significantly reduced from the natural levels and would as a result be quite shallow in the drought periods. However, the channels would be moderately effective in controlling the maximum lake levels near the target elevations.

2. Construction of upland storage within the Shoal Lakes watershed

   Due to the high topographic relief in the uplands of the Shoal Lakes watershed, few areas lend themselves to the creation of upland storage areas. Five storage areas were identified and considered. The effectiveness of the storage reservoirs is limited due to the fact that the storage volume can only be used once during an extended flood period that can persist over a number of years.

3. Purchase of flood prone lands

   The purchase of flood prone lands was considered as a means of reducing flood damages by buying out any property that could be affected by flooding. The hypothesis adopted was that a property would be purchased if it would be affected by the highest recorded flood level (for the period from 1977 to 2010). It was assumed that the land could be leased back to local farmers in periods of low water. This criterion would lead to the purchase of approximately 16,900 ha (41,761 ac) of land from 117 separate owners.

The alternative, which had the greatest reduction of water levels on the Shoal Lakes, was the flood diversion channels. The upland storage option had a minor reduction of water levels on the lakes. The purchase of flood prone land had no effect on water levels on the lake, but would remove all potential flood damages associated with buildings and agricultural activities.

Flood damages were estimated for the existing conditions and the conditions with each of the flood mitigation alternatives in place. The reduction of flood damages from those of existing conditions to those for each of the flood mitigation alternatives was considered to be the benefits associated with each of the alternatives. An economic analysis was then completed, which compared the present values of the costs for each alternative to the present value of the benefits of each alternative to determine the benefit / cost ratio. A summary of the estimated costs, benefits, and B/C ratio (2009 dollars) is presented in Table 16.1.

Table 16.1
Benefit / Cost Ratio of Flood Mitigation Alternatives

Flood Mitigation Alternative	Present Value of Costs	Present Value of Benefits	Benefit / cost
Flood Diversion Channels			
Wagon Creek – Option A	$23,773,000	$8,018,268	0.34
Wagon Creek – Option B	$26,610,000	$8,018,268	0.30
Roy's / Boundary Drain	$31,546,000	$8,018,268	0.25
Upland Storage Areas	$ 4,550,000	$1,580,487	0.35
Purchase of Flood Prone Land	$11,361,000	$7,580,698	0.67

Through this study, it was determined that the most attractive flood mitigation alternative from an economic perspective was the purchase flood prone land surrounding the Shoal Lakes. However, it was noted that the construction of a diversion channel could become more economical if soil conditions were favourable and the corresponding excavation costs were less than had been assumed. Further geotechnical investigation of the soils would be required to verify this.

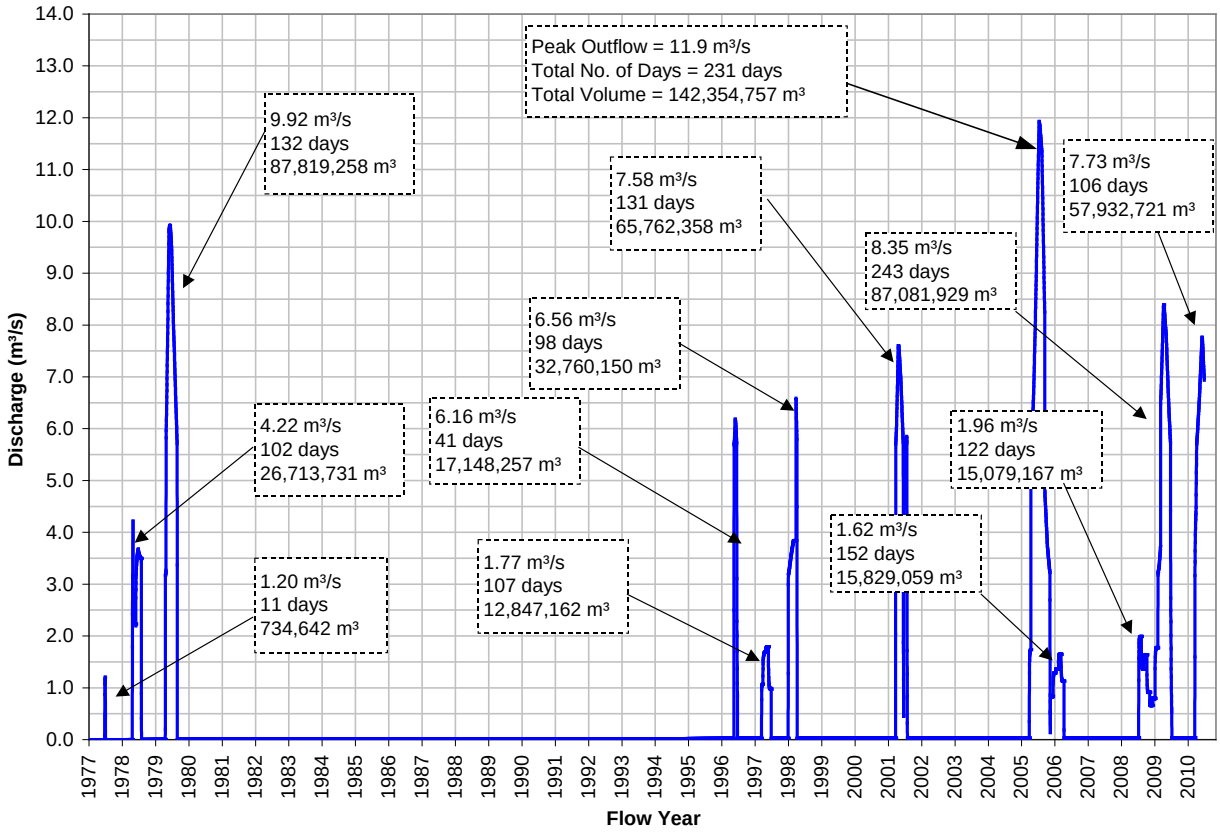
16.2 EFFECT OF SHOAL LAKE DIVERSION ON LAKE MANITOBA

In the context of the Assiniboine River & Lake Manitoba Basins Flood Mitigation Study, the results of the Shoal Lakes Watershed Study were analysed to determine the potential effects of the alternatives investigated on Lake Manitoba water levels. This analysis was conducted to understand what would be the impacts, if any, of the proposed Shoal Lakes flood mitigation alternatives to the flood mitigation alternatives being considered for the Assiniboine River & Lake Manitoba Basins Flood Mitigation Study.

Since the Shoal Lakes are part of a generally land-locked drainage basin with no natural outlet from the lakes, except when water levels are extremely high, only the schemes of constructing diversion / outlet channels from the Shoal Lakes to Lake Manitoba have a potential direct impact on Lake Manitoba inflows and water levels. Other flood mitigation alternatives considered in the Shoal Lakes Watershed Study have no direct impact, as they do not change or add inflow to the existing Lake Manitoba drainage area.

In the event that a diversion channel is constructed, water would be allowed to flow from the Shoal Lakes to Lake Manitoba. This extra inflow into Lake Manitoba would result in an increased water level on the lake. Results from the Shoal Lakes Study indicated that, with a diversion channel in place, the largest outflow from the Shoal Lakes over the study period of 1977 to 2010 occurred in 2005 when a total volume of 142,400 dam^3 (115,000 ac-ft) of water was transferred to Lake Manitoba. A summary of the Shoal Lakes outflow diverted to Lake Manitoba over the study period is shown on Figure 16.1.

Figure 16.1
Shoal Lake Outflows Diverted to Lake Manitoba through Proposed Diversion Channels



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

To determine the maximum increase in water level on Lake Manitoba due to the proposed diversion channel, the entire 2005 volume of 142,400 dam^3 (115,000 ac-ft) was hypothetically assumed to be added instantaneously to Lake Manitoba. Assuming a Lake Manitoba level of El 247.65 m (El 812.5 ft) and a storage capacity of approximately 4,700,000,000 dam^3/m

(1,160,000 ac ft/ft), the added volume of water would cause the lake to rise by 3 cm (1.2 in). This estimate is considered to be conservatively high, considering that the entire volume of water would not enter Lake Manitoba instantaneously, and the increased stage on Lake Manitoba would cause an increase in outflow, thereby partially offsetting the additional inflow. A more detailed analysis would clearly show an even lesser impact to Lake Manitoba water levels. It was concluded that no further examination of this issue would be warranted at this level of conceptual study of Lake Manitoba.

16.3 CONCLUSIONS

1. The results of the KGS Group 2008-2010 study (KGS Group, 2010) showed that the most attractive flood mitigation alternative from an economic perspective for alleviating flooding issues within the Shoal Lakes Watershed was to purchase the flood prone land surrounding the Shoal Lakes. This conclusion applied even if the alternative, which would have the greatest reduction of water levels on the Shoal Lakes would be the flood diversion channels.

2. Of the various options considered, only the alternatives involving flood diversion channels from Shoal Lakes to Lake Manitoba could potentially have a direct impact on Lake Manitoba water levels.

3. The maximum increase in water level on Lake Manitoba due to a diversion channel from Shoal Lakes was calculated to be 3 cm (1.2 in), as a conservatively high estimate. Therefore, potential flood mitigation alternatives considered for the Assiniboine River & Lake Manitoba Basins Flood Mitigation Study were, for practical purposes, deemed to be independent of the mitigation alternatives considered in the Shoal Lakes Watershed Study.

DRAWINGS

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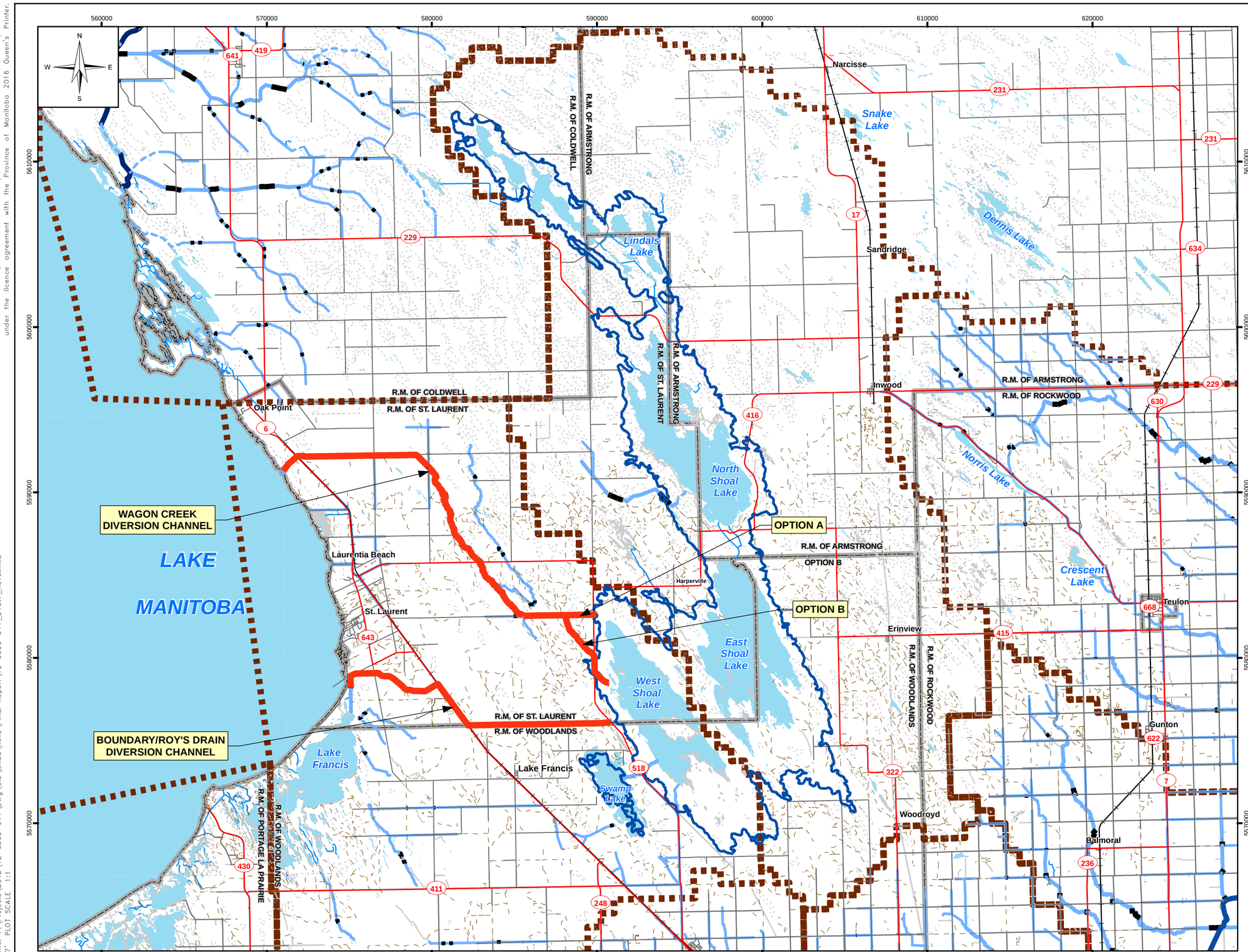

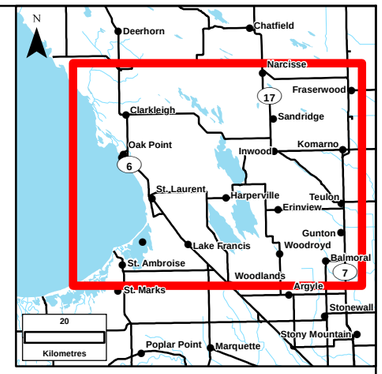

LEGEND:

- Approximate High Water Level (2010)
- Drain Route
- Provincial Road
- Secondary Road
- Trails
- Railway

Drain Lines by Order Number

- 0
- 1
- 2
- 3
- 4
- 5

- River
- Water Feature Non Indexed
- Wetland
- Designated Watershed Drainage Basin
- Rural Municipality
- Lake

1.5 0 1.5 3 4.5 6
Kilometres

SCALE: 1:220,000 METRIC 11"x17"

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Transverse Mercator Projection, NAD 1983, Zone 14
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Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

LOCATION PLAN OF SHOAL LAKES WATERSHED

JANUARY 2016 | DWG 16.1 | REV: 0

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17.0 FLOOD MITIGATION WITH A COMBINATION OF SMALL DAMS

17.1 INTRODUCTION

Flood mitigation with a combination of small dams on tributaries of the Assiniboine and Souris rivers in Manitoba was considered as part of this study. These dams would be operated strategically to reduce peak flood flows in the Assiniboine and Souris rivers. This section describes the impact of 21 small dams, which could be constructed and operated for flood control. The effect of the dams on discharge hydrographs at key locations along these major rivers is presented for a range of possible design floods with return periods of 20, 50, 100, 200 and 500 years.

17.2 SELECTION OF LOCATIONS FOR POSSIBLE SMALL DAMS

Small dams were defined as having storage volumes of 35,000 dam^3 (28,400 ac-ft) or less. This demarcation was based largely on a review of proposed small water supply dams in the Assiniboine and Souris watersheds during previous studies by PFRA and Manitoba Water Stewardship. Many of these 'small dams' were identified during the review of background reports discussed in Section 3.0 such as: PFRA, 1989, or were identified in KGS Group, 2005. Further references to previous studies are provided in Table 1 of KGS Group, 2005.

Since the small dams would be operated to reduce tributary flows at specific points in time, they would all require a means of controlling outflow. Gates or stop logs were selected to achieve this. Fixed-crest dams (without gates) on tributaries would be of virtually no use for flood control on the mainstem, since their reservoirs would usually fill long before the point at which withholdings of flood inflow would be needed to reduce flood peaks on the mainstem river. Having the ability to store and control the water is clearly necessary to have the desired effect on flood crests downstream.

Dams were planned to be located mostly on large gauged tributaries, which would contribute significant flows near the time of the flood crest on the mainstem river.

The location for dams was based in large part to take advantage of existing topography. Locations were preferred where stream valleys were of significant size to provide substantial storage without severe environmental impacts such as flooding of farmland, towns and upstream road crossings. Many of the proposed dams were planned to have small storage volumes due to concerns about the environmental impacts. Detailed topographic information, in addition to the provincial designation of drain maps, was not available for this study. If small dams were proven to be advantageous, the final suitability of the locations of the dams would require further review with improved topographic information.

Storage volumes at the small dams were approximated at this stage of the feasibility study. They were based on information from previous water supply studies by PFRA (1989) and other Manitoba Water Stewardship studies. For tributaries where small dams had not been previously studied, the storage volumes were estimated based on approximations of channel width, slope and average depth at the full supply level of the proposed reservoir. Such approximations were based mostly on information from provincial drainage maps. Consideration was given to minimizing backwater effects and impacts on upstream road crossings and towns.

Of the proposed dams, 10 would be located on tributaries of the Assiniboine River upstream of Brandon, 4 between Brandon and Portage la Prairie, and 7 on tributaries of the Souris River. Together, the 21 small dams would have a total storage volume of approximately 200,000 dam^3 (162,000 ac-ft). Information on proposed locations, drainage areas and estimated storage volumes has been shown in Tables 17.1 and 17.2. The conceptual location of the proposed dams has been shown on Drawings 17.1 and 17.2 (located at the end of Section 17.0).

Table 17.1
Proposed Dams on Assiniboine River Tributaries

Dam	Conceptual Location	Drainage Area (km^2)	Storage at FSL (dam^3)
1. Shell River	Duck Mtn Prov. Forest	840	28,000
2. Thunder Creek	5-22-29W	170	8,000
3. Deerhorn Creek	16-21-29W	550	2,500
4. Silver Creek	u/s PTH 83	457	15,300
5. Birdtail River	u/s PTH 16	1100	32,500
6. Minnewasta Creek	u/s PR 355	452	6,000
7. Arrow River	23-13-26W	606	9,300
8. Gopher Creek	1 mile se Virden	300	3,200
9. Oak River	18-11-22W	1860	6,400
10. Little Saskatchewan	u/s PTH 10	2610	35,000
11. Little Souris River	18-9-17W	690	4,500
12. Willow Creek	u/s PR 468	665	4,500
13. Oak Creek	u/s PTH 2	1030	4,500
14. Cypress River	u/s PTH 2	630	5,500

Note: Multiply dam^3 by 0.8107 to convert to ac-ft.

Table 17.2
Proposed Dams on Souris River Tributaries

Dam	Conceptual Location	Drainage Area (km^2)	Storage at FSL (dam^3)
15. Antler River	u/s PTH 83	3220	4,500
16. Gainsborough Creek	E11-2-28W	1150	5,000
17. Graham Creek	u/s PR 445 east	740	5,000
18. Jackson	u/s PTH 83	450	4,000
19. Pipestone Creek	u/s PTH 83	4240	6,000
20. Medora Creek	u/s PTH 3W	980	5,000
21. Elgin Creek	u/s PR 22	530	6,000

Note: Multiply dam^3 by 0.8107 to convert to ac-ft.

17.3 STREAMFLOW HYDROGRAPHS FOR DAM AND MAINSTEM LOCATIONS

Typical flood inflow hydrographs at the proposed dams are needed to determine how much water can be stored and withheld from contributing to the mainstem river at the appropriate time. Design flood hydrographs on the mainstem rivers are needed for comparison of flows with and without dams. It was decided during the course of the study that the effects of small dams would be evaluated at five locations along the Assiniboine River and three locations along the Souris River, as follows:

- Assiniboine River at Shellmouth, St. Lazare, Virden, Brandon and Holland.
- Souris River at Melita, Souris and Wawanesa.

Design flood hydrographs for the mainstem locations listed above were based on the flood hydrographs of the Assiniboine River at Brandon and the Souris River at Wawanesa. These hydrographs at Brandon and Wawanesa were previously developed for this study and documented in Section 5.4. The ratios of drainage areas (area at the selected mainstem location divided by the area at Brandon or Wawanesa, whichever was appropriate) were multiplied by the flows for the previously estimated hydrographs. The estimated flood flows at Brandon were used for all Assiniboine River locations, and the flows at Wawanesa were used for all Souris River locations.

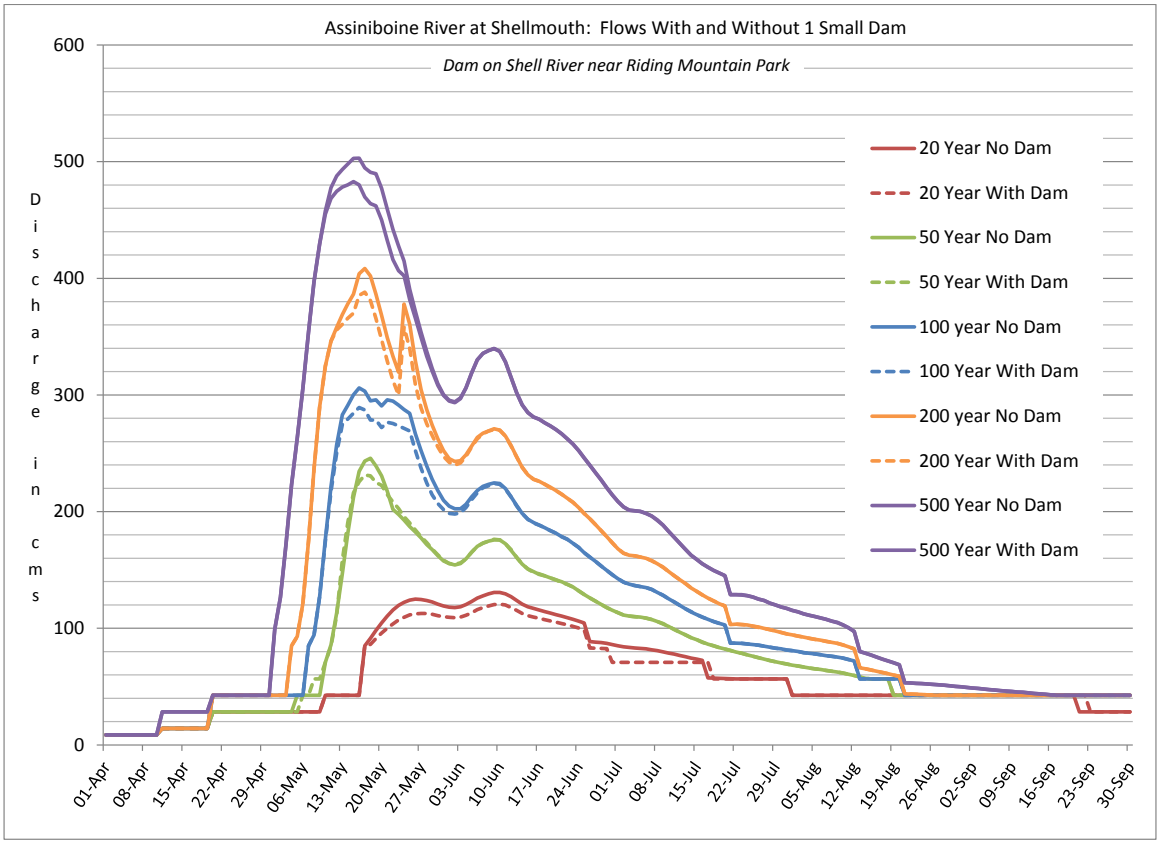
The method outlined above to generate hydrographs on the tributaries of the Assiniboine and Souris Rivers was selected for its simplicity and relative accuracy for the purposes of this conceptual study. It was recognised that flows generated using this method could yield slightly larger or smaller hydrographs on the tributaries than would have been generated based on a more detailed method of analysing the hydrology of each tributary individually. However, since the method was based on hydrographs developed on the Assiniboine River at Brandon and on the Souris River at Wawanesa, it had the benefit of yielding accurate hydrographs on the Assiniboine and Souris Rivers once the inflows from the tributaries were routed downstream. Any anomalies in the method become negligible downstream on the Souris and Assiniboine Rivers where the impacts of the small dams are of the greatest importance. Nevertheless, to minimize the anomalies in the method, several of the hydrographs were adjusted by a factor of 0.75 based largely on the data from the 2011 Flood. A detailed analysis of the hydrology of each

tributary of the Assiniboine and Souris Rivers was not considered warranted for this conceptual study.

17.4 EFFECT OF SHELL RIVER DAM ON ASSINIBOINE RIVER AT SHELLMOUTH

Evaluation of the effects of a dam on the Shell River was complicated by the fact that the river flows into the Shellmouth Reservoir. Those inflows are subject to attenuation by the Shellmouth Reservoir, which is controlled by operation of the Shellmouth Dam. There appeared to be no other way to properly evaluate this effect than to undertake flood routing calculations of inflows to the Shellmouth Reservoir. Two conditions were addressed: one without the Shell River Dam, and one with the modified inflows resulting from holdouts at the Shell River Dam. Design flood inflows to the Shellmouth Reservoir without the Shell River Dam in place were computed based on flows at Shellmouth (just downstream of Shellmouth Dam) computed from the method of prorating drainage areas incremented by a factor of 2.5. This factor was necessary as the drainage area method to develop the design hydrographs resulted in an underestimation of flows on the Shell River. This made the computed inflows approximate the observed inflows reasonably well for the past 40 years. Flood routing calculations were then performed for the five return periods based on typical Shellmouth Dam operation rules that had been previously selected by MIT to minimize outflows. The Shell River Dam was operated so that the holdouts would reach Shellmouth Dam a few days prior to the peak outflows for the routing without the Shell River Dam. The effect of this procedure was to optimize the timing of the holdouts from the upper Shell River Dam to minimize peak outflows from the Shellmouth Dam. Since the peak Shellmouth outflows in the model reach Brandon after the Assiniboine River crest in Brandon has occurred, the Shell River holdouts had little benefit (i.e., reduction in peak flows) for Assiniboine River locations downstream of St. Lazare. This was consistent with conditions from past floods where peak Shellmouth outflows almost always reached Brandon after the Assiniboine River crest in Brandon has occurred. Figure 17.1 shows the simulated flows just below the Shellmouth Dam for each of the design floods with and without the dam on the Shell River.

Figure 17.1
Assiniboine River at Shellmouth



Note: Flows can be converted to cfs by multiplying metric values by 35.3.

17.5 OPERATION OF SMALL DAMS AND COMPUTATION OF CONTROLLED FLOWS

For the simulations in this study, it was assumed that all dams, with the exception of the dam on the Shell River discussed in Section 17.4, would remain fully open until such time as the crest of the flood hydrograph on the mainstem river reached the area of the dam. At that point in time, the gates or stoplogs of the dams on the tributaries would be closed and water stored until full supply level was reached in the reservoir. The design flood hydrographs on the mainstem river nearby were dictated by the appropriate time to close the gates on each tributary. The total volume of flow stored behind each dam would result in a reduction of the flood flow in the mainstem river by that same amount. These daily holdouts were then routed down the mainstem river and combined with holdouts from the other small dams to compute the total daily flow reduction at each study point on the mainstem river. For example for the Assiniboine River, holdouts from dams upstream of St. Lazare were added together and the cumulative daily

holdout values subtracted from the downstream mainstem flow of the Assiniboine River at Virden, along with holdouts from dams between St. Lazare and Virden. The same procedure was applied to other river reaches. The cumulative holdouts at mainstem locations were subtracted from the uncontrolled mainstem flows to estimate the controlled daily design flows. It was assumed that storage in the dams would eventually be released during low flow periods in the summer or autumn. These releases were considered of little interest to a flood mitigation study and were not applied to the computed design hydrographs for mainstem locations.

Hydrographs showing mean daily flows with and without the small dams, for the 8 locations on the Assiniboine and Souris Rivers indicated in Section 17.3, are shown on Figures 17.2 to 17.8 with each graph showing both conditions (with and without small dams) for a range of floods with 20 year to 500 year return periods. As shown on the figures, locations that were the furthest downstream on the two rivers were impacted by the greatest number of small dams, and showed the greatest reductions in flood flows.

Figure 17.2
Assiniboine River at St. Lazare

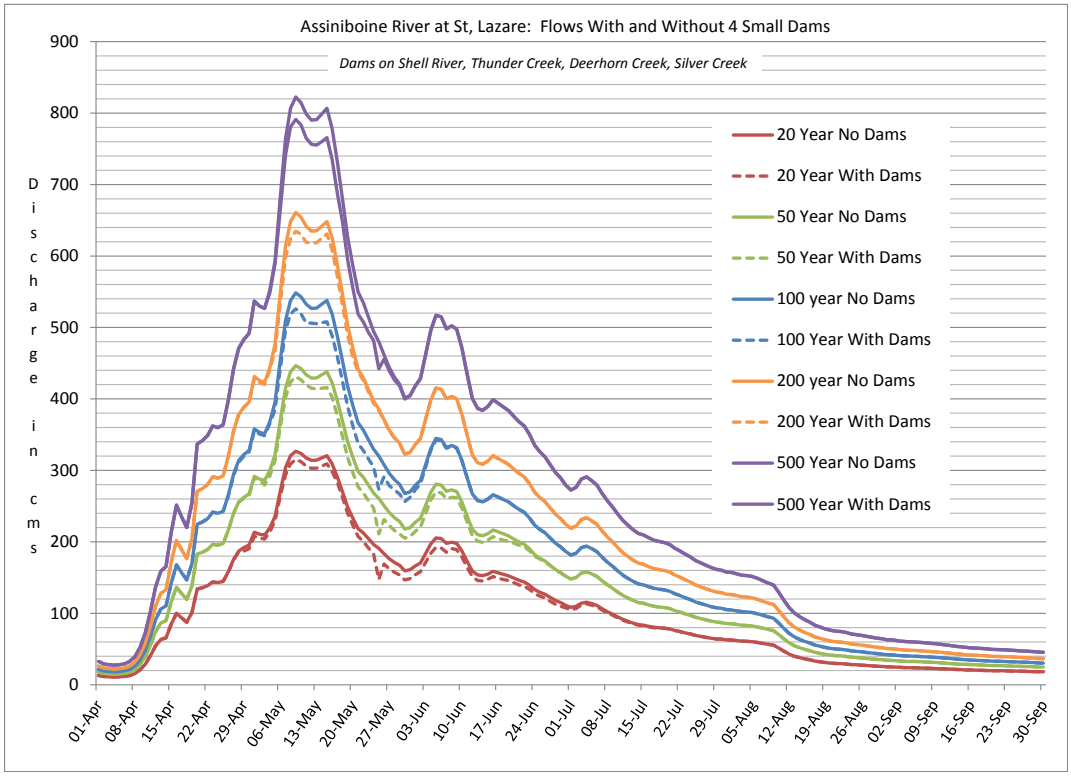


Figure 17.3
Assiniboine River at Virden

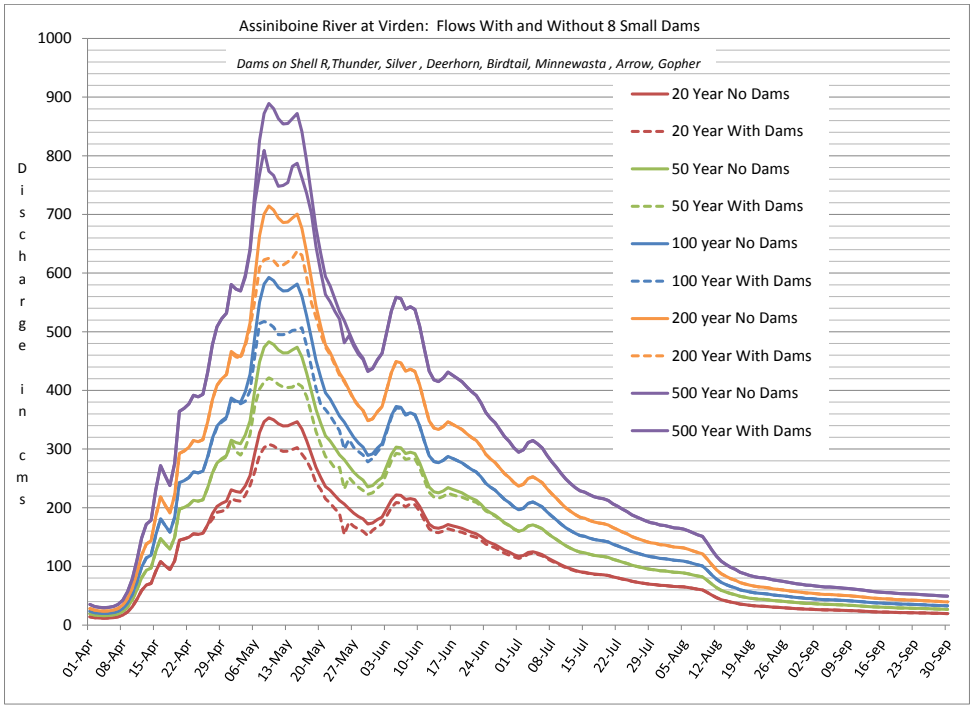


Figure 17.4
Assiniboine River at Brandon

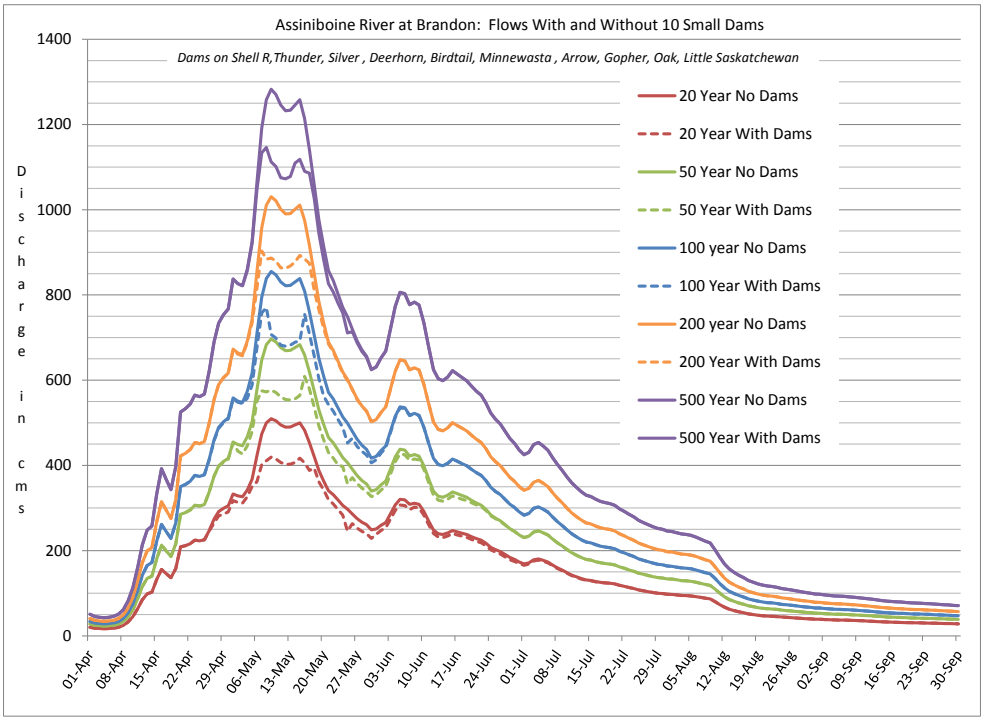


Figure 17.5
Souris River at Melita

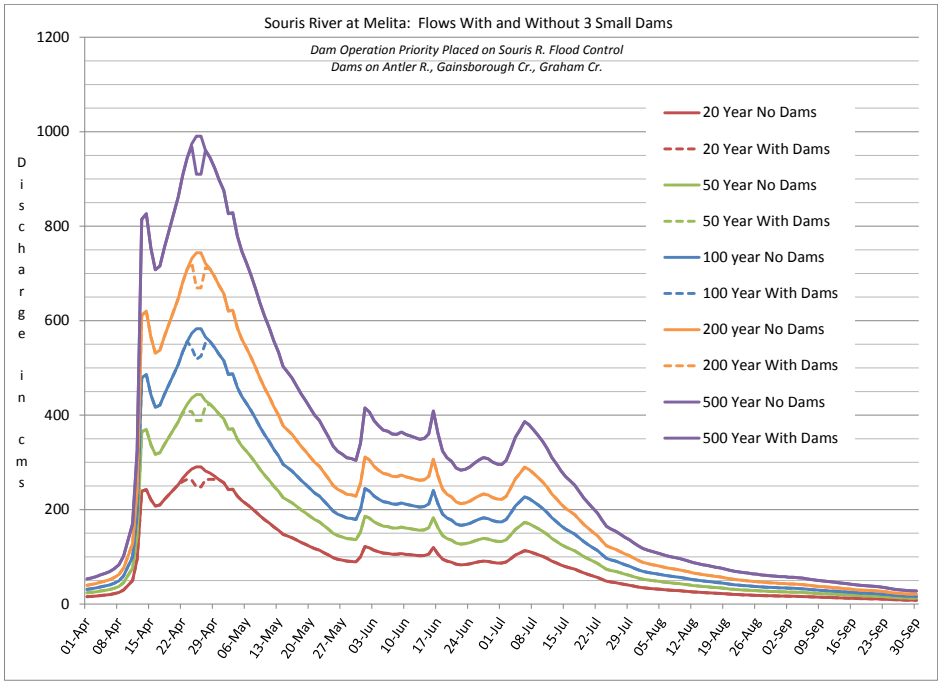


Figure 17.6
Souris River at Souris

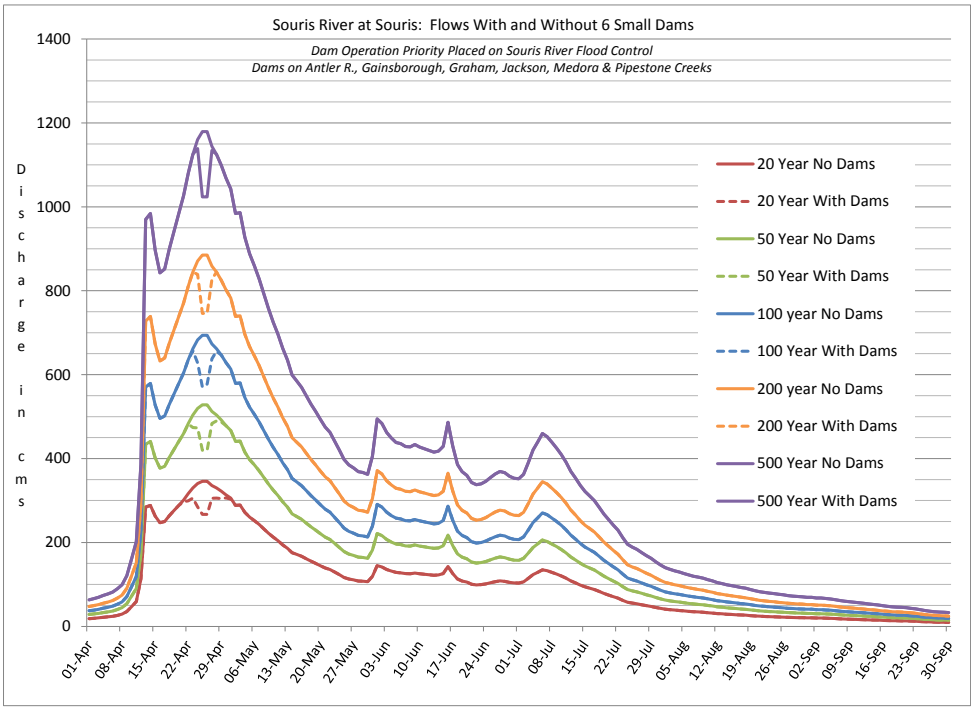


Figure 17.7
Souris River at Wawanesa

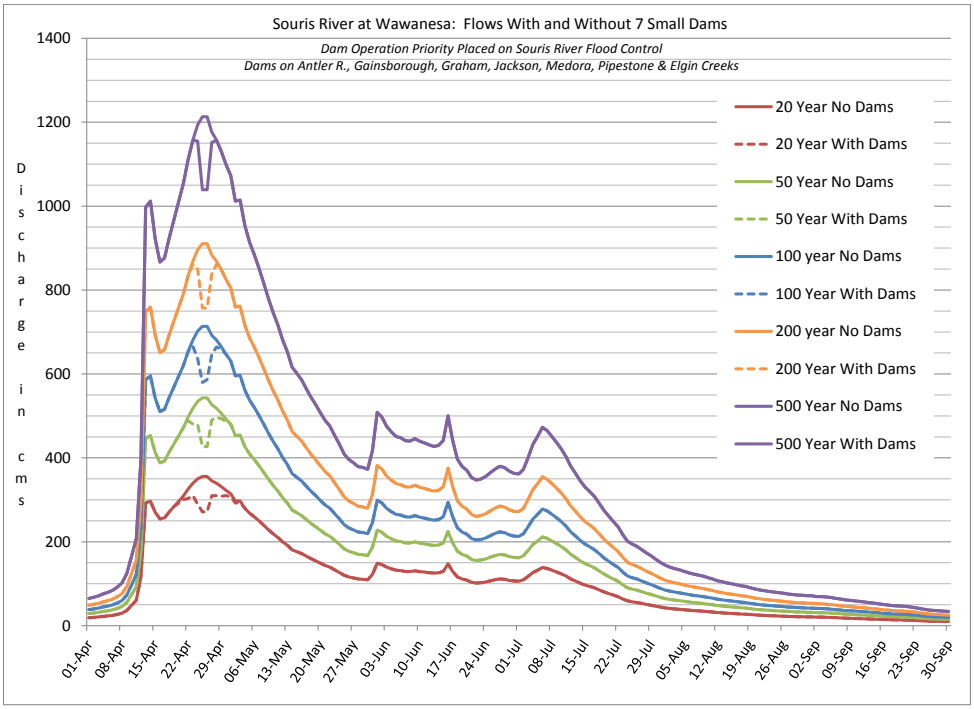
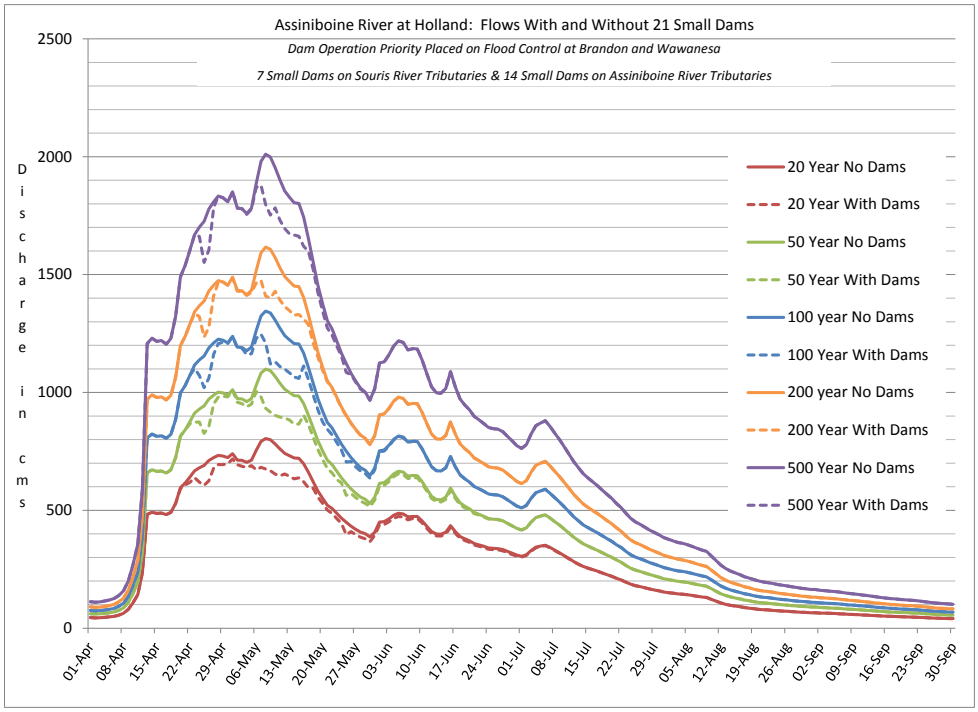


Figure 17.8
Souris River at Holland



17.6 OPERATION OF SOURIS RIVER SMALL DAMS TO REDUCE FLOOD CRESTS AT HOLLAND

The seven small dams identified for the Souris River would first be operated to reduce the flood crest along the Souris River, as discussed in Section 17.5. However, the design hydrographs of the Souris River at Wawanesa reach a peak about two weeks prior to those in the Assiniboine River at Holland. As a result, operations of the small dams to reduce the flood crest along the Souris River were shown to have little or no effect on flood crests at Holland. Since crests of major floods at Holland are of great significance to operation of the Portage Diversion and to the crest of the flood in the Lower Assiniboine River, it was decided to investigate the effect of operating the proposed small dams on the Souris River in a manner which would place priority on reduction of the flood peaks of the Assiniboine River at Holland rather than on reduction of flood peaks strictly in the Souris River. Accordingly, a second scenario of operation of the small dams on the Souris River was developed to assume cutback of the outflow from the dams to be delayed to have maximum impact at Holland. The resultant hydrographs, showing less benefit for the Souris River, have been shown on Figures 17.9 to 17.11. The hydrographs showing the additional reduction in peak flood flows at Holland have been shown on Figure 17.12 (note that Figure 17.12 also includes the operation of 14 small dams on Assiniboine River tributaries as discussed in Section 17.5).

Figure 17.9
Souris River at Melita with Dam Operation Priority Placed on Flood Control at Holland

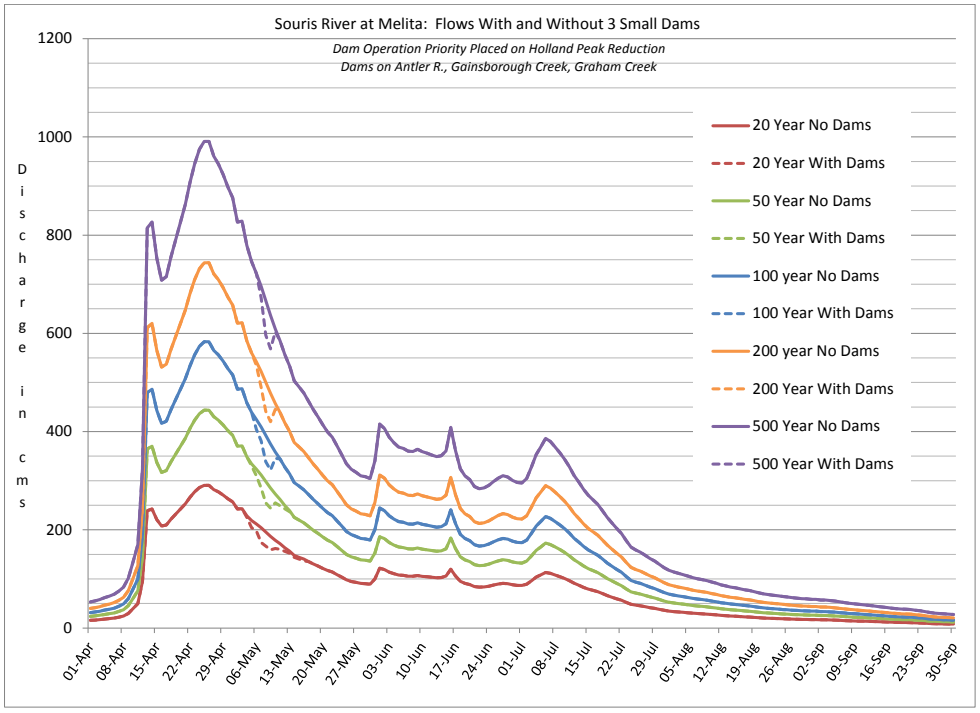


Figure 17.10
Souris River at Souris with Dam Operation Priority Placed on Flood Control at Holland

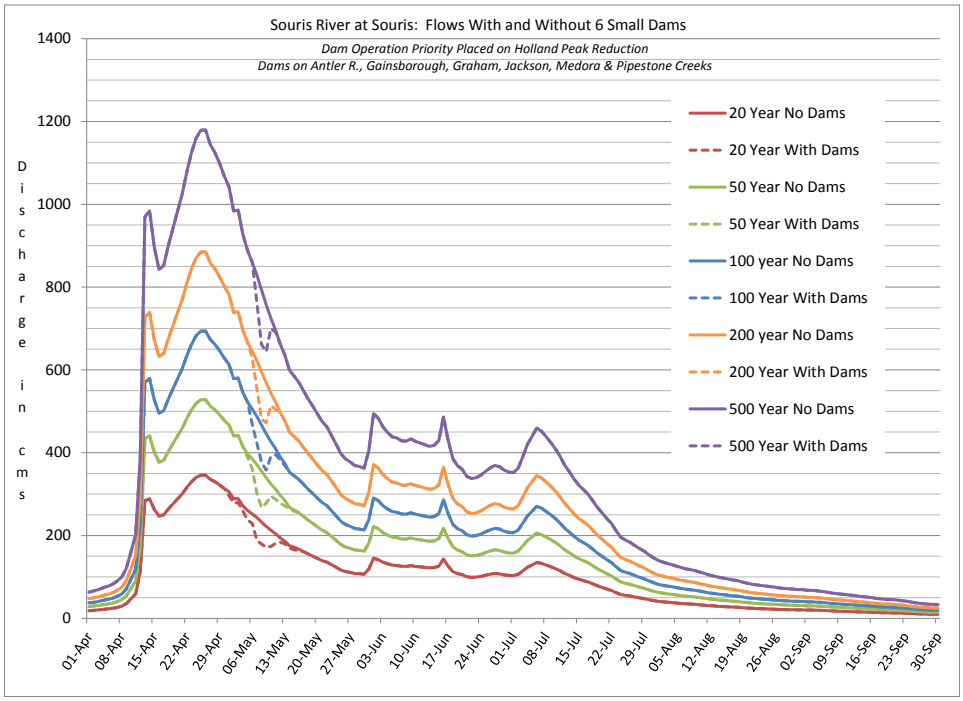


Figure 17.11
Souris River at Wawanesa with Dam Operation Priority Placed on Flood Control at Holland

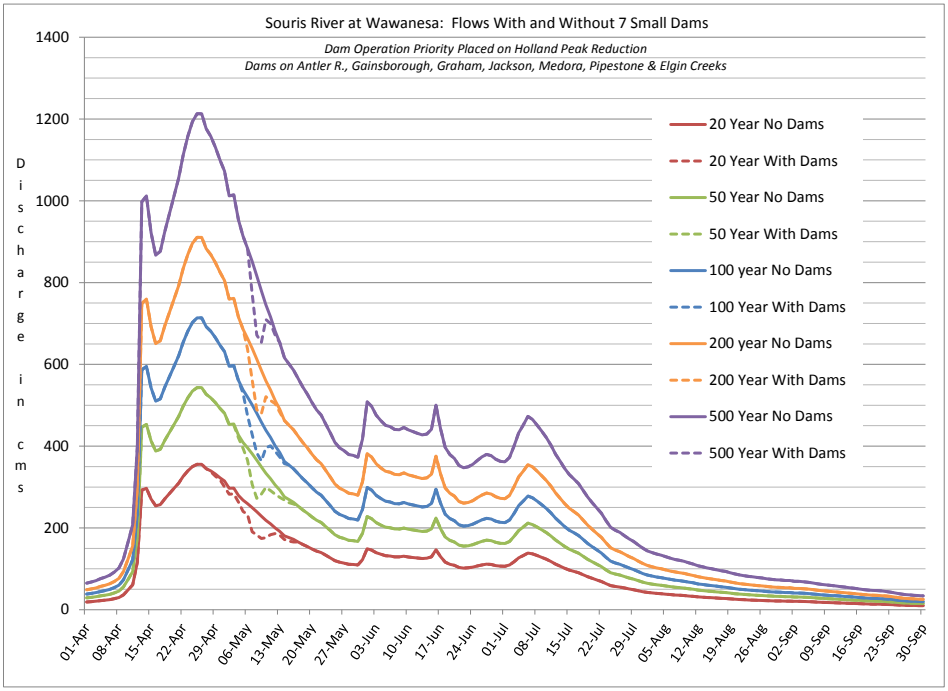
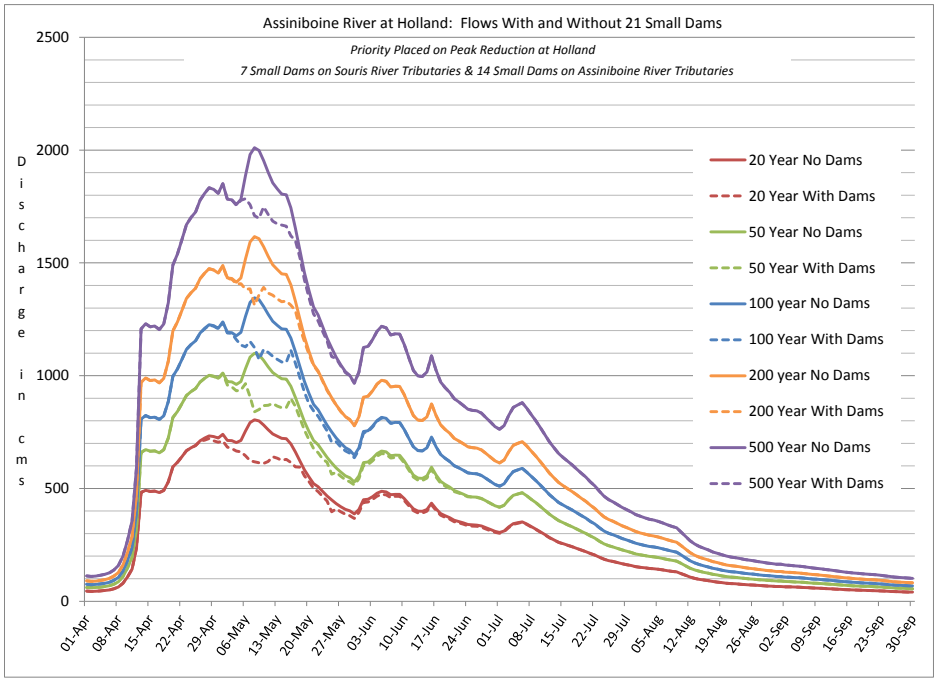


Figure 17.12
Assiniboine River at Holland with Dam Operation Priority Placed on Flood Control at Holland



The computed hydrographs based on the simulation of operation of the small dams were erratic. This was in large part due to the manner in which the cumulative holdouts at the dams developed. Some reservoirs of dams would fill earlier than others due to relatively high inflows or due to lesser storage volumes. Also the operations at the dams, which determine the holdouts and the resultant effect on mainstem flows, were not timed perfectly with respect to mainstem peak flows. This caused further randomness. In addition, the simulated operations were believed to be very efficient in reducing mainstem flow when perfect flood forecasts were assumed. However, in reality, the reductions would be much less dramatic in a real-time setting where unknowns in the dates of impending floods would adversely affect the operations and their effects on flood peaks downstream.

A summary of peak flow reductions due to the operation of small dams is shown on Table 17.3 for two of the return periods. These are based on the hypothetical analyses using perfect foreknowledge of flood flows.

Table 17.3
Summary of Peak Flow Reductions

	Reduction in Peak Flow (m^3/s)	Reduction in Peak Flow (m^3/s)	Reduction in Peak Flow (%)	Reduction in Peak Flow (%)
	50 Year	200 Year	50 Year	200 Year
Assiniboine at Shellmouth	14.4	20.3	5.9%	5.0%
Assiniboine at St. Lazare	15.7	26.2	3.5%	4.0%
Assiniboine at Virden	61.3	77.1	12.7%	10.8%
Assiniboine at Brandon	87.6	127.5	12.6%	12.4%
Souris at Melita	20.4	21.4	4.6%	2.9%
Souris at Souris	38.5	40.6	7.3%	4.6%
Souris at Wawanesa	47.0	49.0	8.7%	5.4%
Assiniboine at Holland	89.1	131.2	8.1%	8.1%

Note: Flows can be converted to cfs by multiplying metric values by 35.3.

17.7 COMMENTS ON RESULTS AND ISSUES OF CONCERN

As discussed in Section 17.3, the locations and characteristics of the small dams identified in this report are approximate and conceptual only. The exact locations and quantities of the dams would have to be refined in a more detailed analysis should the proposed concept of flood mitigation with a combination of small dams for flood control proceed to a subsequent stage of analysis. One concern would be that the locations, volumes and the environmental impacts caused by the dams could make some proposed dams appear to be unfeasible. If so, the locations of those dams could be shifted to other more ideal locations. On the other hand, if some dams must be eliminated, the cumulative effect of the remaining dams would be less than suggested in this report.

An issue of importance is the operating efficiency of the dams. During this study, operations were simulated with the benefit of perfect foreknowledge of river behaviour during floods; thus, the calculations could approach the maximum possible reduction in flood crests. In reality, it could be very difficult to achieve an optimum operation given the many uncertainties which could affect decision making. For example, it could appear that the mainstem crest was near; and, therefore, dams were closed to shave off the flood peak. Then, if the date of the mainstem crest turned out to be different than predicted or was delayed due to rainstorm(s), the dams would be near full and would be of little or no further use in reducing the delayed crest. It would take unusual foresight to draw down the dams again in preparation for a second crest, which might or might not occur. Releasing water from the dams while rivers are still in a serious flood condition would face much opposition, making such operations unlikely to be socially acceptable. In addition, there would need to be sufficient staff available to operate 21 dams on short notice, which could strain staff resources that were already devoted to fighting a major flood. This has both practical and cost implications that have not been included in the analyses described herein.

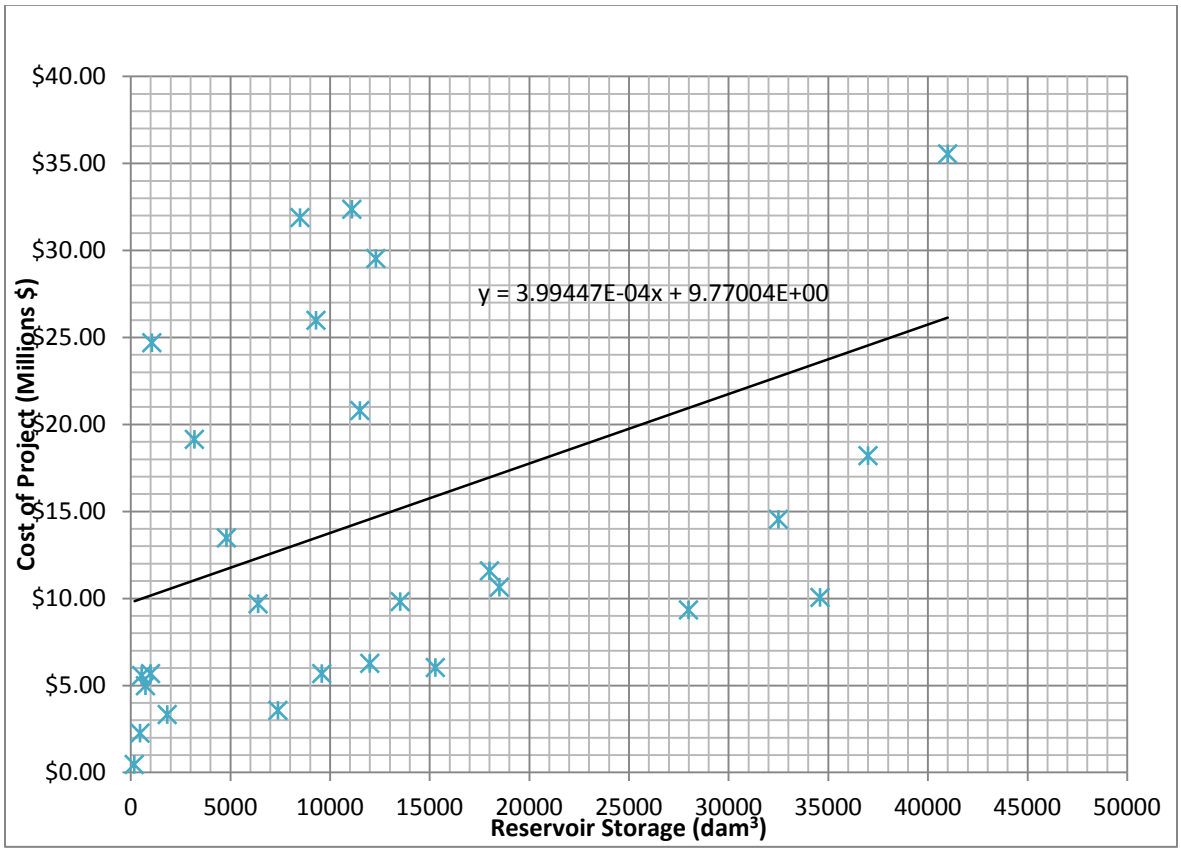
On the tributaries, each small dam could have local flood reduction benefits. However, for the operating scenarios presented in this report, the benefits would likely be minimal or negligible since each dam would be operated to store water that coincided with the peak on the Souris and Assiniboine River, and not of the tributary. Nevertheless, an analysis was conducted on Pipestone Creek to obtain a representative estimate of the potential benefits of a dam, if it was

operated to reduce the peak only on the tributary and not on the mainstem rivers. Assuming optimal forecasting and operating conditions, that dam could reduce the peak flows immediately downstream by 15% to 20% depending on the magnitude of the flood. However, the dam would no longer provide the optimal benefits downstream on the mainstem rivers that were reported in Sections 17.5 and 17.6 and, therefore, would not be a preferred flood mitigation strategy for the Souris and Assiniboine rivers. The local benefits of each individual dam have not been evaluated in this study and would require further review should flood mitigation with a combination of small dams be selected to proceed to the next phase of study.

17.8 PRELIMINARY COST ESTIMATE

A preliminary cost estimate was developed for each of the 21 small dams included in the analysis. At this conceptual level study, a relationship of direct cost per unit of available storage was used to determine the total project costs. The relationship developed was based on the estimated direct costs of 28 similar dam sites, which had reservoir storage volumes of 41,000 dam^3 (33,200 ac-ft) or less, and were previously studied for increasing water supply in the Assiniboine River Basin. The capital costs of these dams (in 2004$) were summarized in the June 2005 memorandum entitled, *"Water Storage and Supply Options for the Assiniboine*" (KGS Group, 2005). To estimate the direct costs in 2014$, allowances for contingencies and indirect costs such as engineering, approvals, administration and owner's costs, were subtracted from the capital cost estimates of 2004. The direct construction costs were then escalated to present day using a 5% yearly increase. The 5% rate was based on Consumer Price Index data from Statistics Canada, as well as experience with other known similar, recent projects. The relationship developed between cost and reservoir storage volume is provided in Figure 17.13.

Figure 17.13
Relationship of Cost as a Function of Storage for Small Dams



Though the relationship shown on Figure 17.13 is not ideal to estimate with precision the cost of a single dam site, it does provide a good approximation of the total cost of numerous dams when added together, such as what was being considered in the current study. The relationship shown on Figure 17.13 was adopted at this conceptual level of study. Table 17.4 summarizes the total direct and capital costs of the 21 small dams that were investigated. As per the economic guidelines set out for this project and described in Section 7.0, an allowance of 20% of the total direct costs was added to account for indirect costs such as engineering, approvals, administration and owner's costs. An allowance of 30% of direct and indirect costs was added for contingency. The estimated capital cost of the 21 dam sites would be approximately $482 million.

Table 17.4
Estimated Cost of Small Dams

Dam	Storage at FSL (dam^3)	Total Direct Cost (Million $)	Enviro. Mitigation Cost (Million $)	Indirect Costs (Million $)	Contingency (Million $)	Total Capital Cost (Million $)
1. Shell River	28,000	21.0	2.1	4.2	8.2	35.4
2. Thunder Creek	8,000	13.0	1.3	2.6	5.1	21.9
3. Deerhorn Creek	2,500	10.8	1.1	2.2	4.2	18.2
4. Silver Creek	15,300	15.9	1.6	3.2	6.2	26.8
5. Birdtail River	32,500	22.8	2.3	4.6	8.9	38.5
6. Minnewasta Creek	6,000	12.2	1.2	2.4	4.7	20.6
7. Arrow River	9,300	13.5	1.3	2.7	5.3	22.8
8. Gopher Creek	3,200	11.0	1.1	2.2	4.3	18.7
9. Oak River	6,400	12.3	1.2	2.5	4.8	20.8
10. Little Saskatchewan	35,000	23.8	2.4	4.8	9.3	40.1
11. Little Souris River	4,500	11.6	1.2	2.3	4.5	19.5
12. Willow Creek	4,500	11.6	1.2	2.3	4.5	19.5
13. Oak Creek	4,500	11.6	1.2	2.3	4.5	19.5
14. Cypress River	5,500	12.0	1.2	2.4	4.7	20.2
15. Antler River	4,500	11.6	1.2	2.3	4.5	19.5
16. Gainsborough Creek	5,000	11.8	1.2	2.4	4.6	19.9
17. Graham Creek	5,000	11.8	1.2	2.4	4.6	19.9
18. Jackson	4,000	11.4	1.1	2.3	4.4	19.2
19. Pipestone Creek	6,000	12.2	1.2	2.4	4.7	20.6
20. Medora Creek	5,000	11.8	1.2	2.4	4.6	19.9
21. Elgin Creek	6,000	12.2	1.2	2.4	4.7	20.6
					Total Cost	**482.2**

Note: Multiply dam^3 by 0.8107 to convert to ac-ft.

17.9 CONCLUSIONS

The impact of operating 21 small dams on tributaries of the Assiniboine and Souris Rivers for flood control showed that the reduction in peak flows on the Assiniboine and Souris Rivers ranged between approximately 3% and 13% depending on the location and size of flood. The reduction in flows on the Assiniboine River near Holland was estimated to be approximately 8%. This value was considered conservative as a lesser peak reduction could be expected without

perfect foreknowledge of the river peak, and without perfect timing of dam operation and closures.

Operation of the dams to reduce peak flows on the tributaries is not a preferred flood mitigation strategy for the Souris and Assiniboine Rivers, as it would not provide optimal benefits downstream on the mainstem rivers. However, the dams could have relatively small local benefits that could justify further review in the future.

The estimated capital cost of the 21 dam sites was approximately $482 million, based on a relationship of dam cost vs storage volume developed from the results of previous studies. With such a high cost, potential environmental impacts, relatively small reductions in flood peak flows, and difficulties in effective operation to achieve the maximum benefits, flood mitigation with a combination of small dams was not considered to be a feasible solution when compared to other options presented in this report.

DRAWINGS

Portions of data presented are owned by the Province of Manitoba and are produced under the licence agreement with the Province of Manitoba 2016 Queen's Printer.

Note: The locations & quantities of the small dams shown are approximate and conceptual only. The exact locations & quantities would have to be refined under a more detailed analysis should the proposed concept of constructing a multitude of small dams proceed forward to a subsequent stage of analysis.

LEGEND:

- Proposed Small Dams
- Rail Line
- Road
- Major River
- Stream with Small Dam
- Other Streams
- Waterbody
- Provincial Forest
- Province or State

10 0 10 20 30 40
Kilometers

SCALE: 1:1,000,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
0	16/01/04	ISSUED WITH FINAL REPORT	PAL

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

CONCEPTUAL LOCATION OF SMALL DAMS ON ASSINIBOINE RIVER TRIBUTARIES

JANUARY 2016 | DWG 17.1 | REV: 0

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11"x17" PLOT SCALE 1:1

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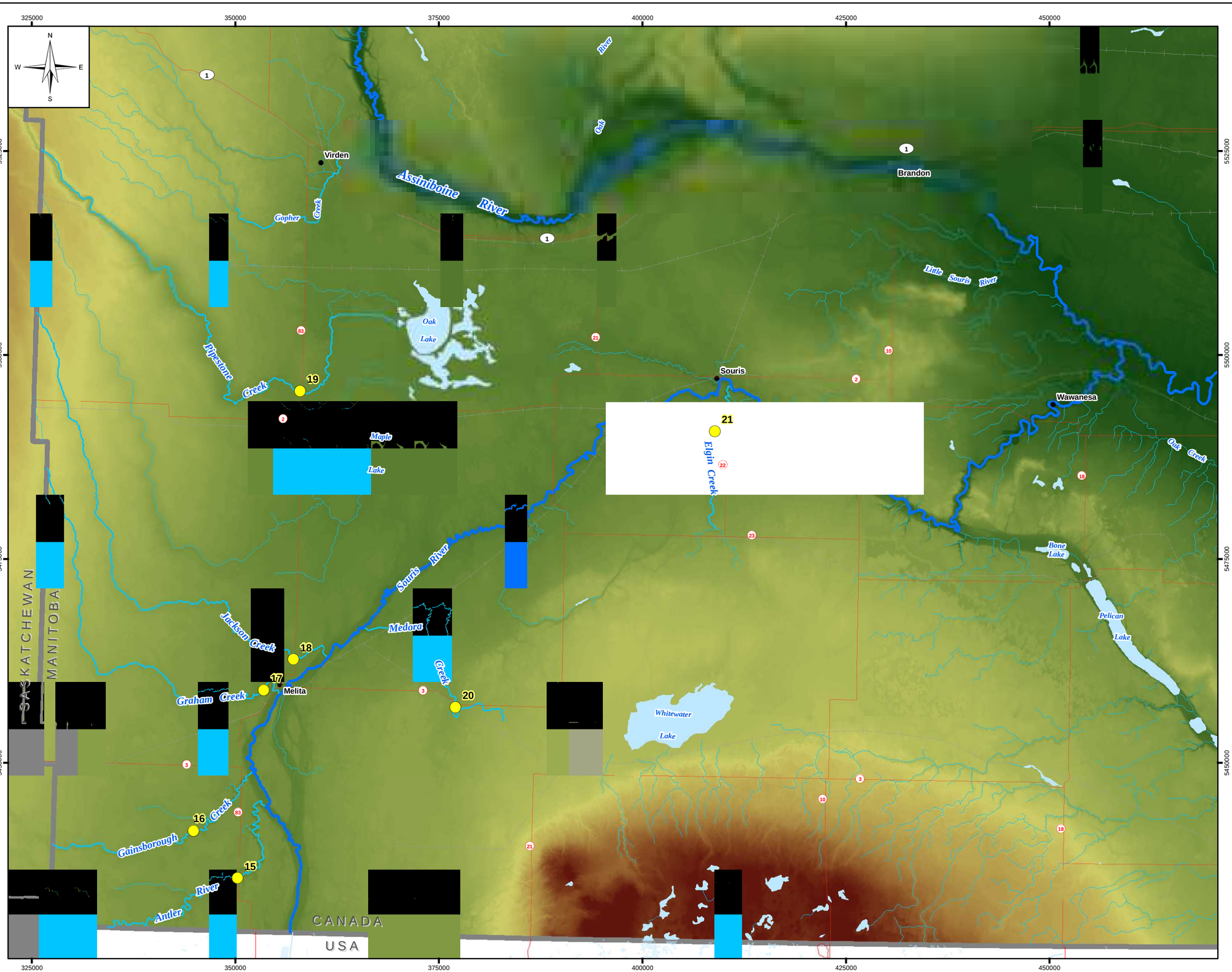

LEGEND:

- Proposed Small Dams
- Rail Line
- Road
- Major River
- Stream with Small Dam
- Other Streams
- Waterbody
- Province or State

5 0 5 10 15 20

Kilometers

SCALE: 1:450,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

MMM INFRASTRUCTURE

ASSINIBOINE RIVER AND LAKE
MANITOBA BASINS FLOOD
MITIGATION STUDY

CONCEPTUAL LOCATION OF SMALL
DAMS ON SOURIS RIVER TRIBUTARIES

JANUARY 2016 | DWG 17.2 | REV: 0

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18.0 DEVELOPMENT CONTROLS AND LAND USE PLANNING

18.1 BACKGROUND

The project terms of reference included an assessment of how development could be restricted or ways in which development could be modified to reduce flood damages, flood fighting and other costs. Floodplain management must rely on both structural measures and non-structural measures including public policy and other strategies that help prevent and reduce flood damage.

A number of vulnerable areas within the study area where development controls could be considered were identified. In addition to identification of these vulnerable areas, options were generated through a review of existing development restrictions and development modifications, public policies and strategies in place in Manitoba and other jurisdictions, and from TAC, stakeholder and government departments input. The findings and recommendations presented in the 2011 Manitoba Flood Review Task Force Report (FRTF, 2013) and the Lake Manitoba, Lake St. Martin Regulation Review Report (Manitoba, 2013a) were also considered.

18.2 LAKE MANITOBA / LAKE ST. MARTIN REGULATION REVIEW COMMITTEE

The Lake Manitoba/Lake St. Martin Regulation Review Committee commissioned a Land Use Planning Study in cooperation with the 2011 Flood Task Force. The findings of that study by McKay Finnigan and Associates (MF&A, 2012) were structured to provide a high level understanding of approaches to land use policies / regulations, to develop general principles, and to arrive at conclusions that would assist the Committee in their reporting.

The land use study concluded that communities in general employ five strategies for managing growth and development in flood-prone areas:

1. Designating hazard lands.
2. Dedicating shoreline reserves.
3. Maintaining / enhancing shoreline vegetation.
4. Defining flood protection levels.
5. Establishing setbacks from water bodies.

Further, it was found that all municipalities in the Red River Valley / Lake Winnipeg areas and the Lake Manitoba / Lake St. Martin areas addressed each of these five strategies to varying degrees in their respective development plans and zoning bylaws. A key issue identified through the Lake Manitoba / Lake St. Martin Regulatory Review and the 2011 Flood Task Force Review reports is that there are insufficient initiatives or planning structures to better ensure an effective coordination of efforts between municipalities, and between municipalities and First Nations communities. For example, it is not uncommon for large drains to terminate at one municipality's boundary with the additional water simply spilling onto the neighboring jurisdiction's lands. Municipalities are also often challenged in trying to administer and enforce adequate land use policies and/or regulations, due to constraints such as the lack of having adequate staff to administer the approval process and to perform inspections. Additionally, some want to avoid placing too many restrictions in the way of new development. Similar issues are also faced by First Nations when it comes to trying to limit potential damage due to floods.

18.3 REVIEW OF MANITOBA PLANNING AND POLICIES

18.3.1 Manitoba's Surface Water Management Strategy

The goal of Manitoba's Surface Water Management Strategy was to develop sustainable, watershed-based planning across the entire Province to ensure that future management of specific water issues is done carefully (Manitoba PowerPoint, 2016).

A number of specific actions have been identified to address issues within each policy area and many of these are presently underway. The strategy endorses the principals associated flood mitigation and management. Applicable flood management strategies are:

- **Implement Basin Level Flood Mitigation Measures** – Implement the recommendations of the Assiniboine Basin and Lake Manitoba Flood Mitigation Study, which is conducting basin-level assessments to identify optimal areas for additional water retention and storage as well as structural (such as large dams) and non-structural (such as land use change) flood mitigation measures.
- **Greater Restriction of Development on Flood Prone Lands** – Apply existing controls on development more stringently in flood prone lands and update building codes to minimize flood damage. For example: require that buildings be built to an appropriate flood protection level by defining flood protection levels in local bylaws; adopt and enforce minimum safe setback distances for structures to reduce susceptibility to

flooding and erosion; reduce the vulnerability of basements to flooding through restrictions on the size and placement of basement windows; and prohibit the location in basements of electrical and mechanical works such as electrical panels and furnaces. The roles and responsibilities of the province and municipalities in mapping flood prone lands and adopting supportive policies in development plans need to be clearly identified, with greater focus on implementation and enforcement

- **Greater Implementation of Existing Designated Flood Area Legislation** – Identify additional designated flood areas that are at high risk for flooding. Under current legislation, new development in a designated flood area must have flood protection levels that comply with the 100-year flood level or the flood of record with an additional 0.6 m (2 ft) freeboard. For example, this generally involves the construction of ring dykes or the raising of structures on pads.
- **Evaluate and Improve Flood Protection Works** – Investigate the optimal use of existing flood protection works including the Red River Floodway, the Shellmouth Dam, the Portage Diversion and the Fairford River Water Control Structure. Evaluate options for additional water control works for flood protection using a full cost / benefit analysis to determine the viability of each.

18.3.2 Provincial Land Use Policies

The Planning Act requires that all municipalities in Manitoba adopt a development plan bylaw that is generally consistent with the Provincial Land Use Policies (PLUPs) identified in the *Provincial Planning Regulation*. Development plan bylaws must be approved by the Province before they can come into effect locally. The Provincial Land Use Policies include a comprehensive planning policy framework for flood prone lands. The PLUPs state that Land Subject to Flooding is land that:

- Is inundated by floods up to and including the design flood (i.e. 1:100 year flood).
- Has a known history of flooding.
- Experiences flooding during a flood event of a magnitude specified by the Province in areas protected by flood control works.
- Is identified under the *Designated Flood Area Regulation*.

The PLUPs further state that development of this land may be permitted only if the risks are eliminated or ways are identified to ensure that:

- No additional risk to life, health or safety is created as a result of the development.
- Buildings and other things constructed, such as septic fields are protected from the risks related to flooding, erosion and bank stability.

- Water flow, velocities and flood levels will not be adversely altered, obstructed or increased as a result of the development.

To incorporate this Provincial Land Use Policy direction in their local development plan bylaws, municipalities rely on the Province to determine what the Land Subject to Flooding (noted above) means in practical terms and the local land use context. This means identification of the Flood Protection Level (FPL) expressed as minimum building elevations in feet / meters above sea level and elevations that can be used by home owners and contractors. It may also involve flood risk mapping of flood-prone lands.

The target FPL varies by location across the Province and is based on the following criteria:

- The 1:200 year event, as recommended by the 2011 Flood Review Task Force, which is greater than the previous 1:100 year standard, as discussed further below.
- The highest flood on record if greater than the 1:200 year event.
- A higher flood event that can be justified in an economic analysis that considers costs and benefits.

The interim Flood Protection Level (FPL) for Lake Manitoba was computed based on the May 31, 2011, wind event (248.5 m (815.3 ft) wind-eliminated water level), plus wind setup (0.6 m to 1.5 m (2 to 5 ft)), plus wave uprush (0.3 m to 0.6 m (1 to 2 ft)), resulting in flood protection levels that could potentially be as high as 250.6 m (822.3 ft). Such lake levels on Lake Manitoba have uncertain probabilities of recurring, but would be very rare (estimated by KGS Group to be in the order of 1:300 years to 1:2,000 years); and communities around Lake Manitoba would have one of the highest standards of flood protection in the Province based on this single event. On the other hand, if structures were constructed (or existing structure raised or diked) based on this Flood Protection Level (FPL), there would be little structural flood damage in future.

As outlined above and discussed in Section 4.0 of this report, the Manitoba 2011 Flood Review Task Force recommended that the FPL for flood affected areas should be the 1:200 year flood. KGS Group concurs with the opinion of the Flood Review Task Force that the previous Provincial standard of the 1:100 year flood was likely too low. With a 1:100 year flood, there is a 1% chance that it will occur in any given year; but the chance that the 1% flood will be exceeded

sometime in that 10-year period is 9.6%; and the chance of flooding sometime in the next 50-year period is 40%.

KGS Group recommends that the government long-term flood protection strategy should be based on the 1:200 year level of protection or the Flood of Record if it is higher, as a minimum. A higher level of protection should be justified on a risk based assessment.

18.3.3 Municipal Planning

The Planning Act requires that all municipalities in Manitoba adopt a development plan and zoning bylaw (either individually or jointly as part of a planning district). The development plan is a policy and mapping document that reflects the essence of the Provincial Land Use Policies as they pertain to the land use and topography of the local planning area. The zoning bylaw then serves as a regulatory document used to implement the policy and mapping direction of the development plan.

The subdivision of land must conform to development plan policies, mapping and zoning bylaw standards. *The Planning Act* requires that all subdivisions be approved by the local municipal council and also by the subdivision Approving Authority (the Province or the local planning district if it has been delegated the authority by the Province). Ideally, flood prone lands should generally be designated for uses that do not allow the construction of permanent structures and left in their natural state or used for low intensity uses compatible with the flood hazard including agricultural, open space and outdoor recreational uses. When local planning documents provide allowance for subdivision and development of permanent structures on flood prone lands, such development should be approved conditional upon the provision of flood protection for all permanent structures. The flood hazard protection level is based on the 200-year flood level, or the flood of record of the adjoining water body or watercourse, whichever is greater (LM&LSMRRC, 2014).

A concern is that some areas of Manitoba and consequently some development plans do not contain sufficient technical information on flooding. This illustrates the need for additional flood risk mapping of communities. The last mapping of Manitoba communities was in the early 1980s under the Canada / Manitoba Flood Damage Reduction Program.

The development controls manage and regulate future property development to ensure it conforms to the set of policies in the plans. A potential gap in the development controls is ensuring that development(s) complies(y) with the Flood Protection Level (FPL). In some cases, compliance is not enforced due to limited technical information and professional capacity at the local level. Furthermore, without well-defined limits or potential impacts to proposed development, it can be difficult for municipalities and the subdivision Approving Authority to reject proposals outright.

To address the issue, all new development(s) should require that the lot grade for the property be compliant with the FPL to be eligible for future disaster financial assistance. Options could be for the Provincial Government, municipalities and planning districts to provide more resources for floodplain management or to have a requirement that the developer have an appropriate professional certify the final lot grade. In this case, the developer(s) or the owner would be required to provide a financial deposit until the FPL grade is verified by a qualified professional, such as a legal surveyor as compliant. The municipalities', planning districts' or the Province's only cost then would be the FPL lot grade certificate.

18.3.4 Integrated Water Management Planning

The Manitoba Conservation District Program was established in 1972 under the authority of the *Conservation District Act*. Today, the Program has expanded to include eighteen conservation districts with a mandate to "*create healthy and sustainable watersheds through focused, priority-based programs that provide definite improvements to watershed health*". Conservation Districts foster watershed cooperation and communication between upstream and downstream municipalities and landowners to address local land and water management issues. To date, all eighteen (18) have been designated as Water Planning Authorities (under *The Water Protection Act*) and have successfully completed or begun the development of an Integrated Watershed Management Plan (IWMP) for their respective areas. To date, 24 planning processes have been initiated, of which 20 plans have been completed and 4 plans are currently being developed. Not all Conservation Districts were established on watershed boundaries. For example, the Turtle Mountain Conservation District has two IWMPs (East Souris River and Pembina River Watersheds) that guide the programming and activities of the district, which covers portions of more than one watershed. IWMPs provide direction to the local community on resource

management including recommendations for development planning within the IWMPs. They currently do not provide all of the technical information such as flood protection levels or flood risk maps required to implement these recommendations. Additional resources to develop flood risk maps would assist with the development of IWMPs.

While Manitoba has experienced significant progress over the past decade in establishing Conservation Districts and developing IWMPs, linkages between these plans and land use planning implemented "on the ground" could be strengthened. IWMPs have evolved to include specific recommendations for land use planning in recent years, however, they are only recommendations without legislated authority. Municipalities or Planning Districts may or may not include these recommendations in their development or secondary plans or local bylaws, and it is often left up to the planner or consultant to seek out and include recommendations from these IWMPs. Stronger linkages would ensure that the IWMP recommendations related to riparian and shoreline protection and surface water management are adequately considered and adopted into land use plans, thereby strengthening the adoption of recommended practices.

The Planning Act only requires RMs and planning districts to consider recommendations from within IWMPs when drafting a development plan or from regulations within the *Water Protection Act*. To strengthen this linkage between municipal planning and the IWMP, consideration could be given to adding a regulation under the *Water Protection Act* requiring RMs and planning districts to incorporate recommendations from IWMPs into their development plans.

18.3.5 Designated Flood Areas

Following the 1979 Red River Valley Flood, the Province introduced a regulation under *The Water Resources Administration Act* designating a specific area south of the Red River Floodway and the West Dike to the Canadian-American border encompassing lands primarily along the Red River and the Morris River but including tributaries, as the Red River Valley Designated Flood Area. The original boundary of the Red River Valley Designated Flood Area was based upon hydraulically modelled 1:100 year flood levels. Following the 1997 "Flood of the Century", the regulation was modified and the Designated Flood Area (DFA) boundary broadened to reflect the actual 1:100 year flooded area that occurred during the 1997 Flood.

By means of *Manitoba Regulation 59/2002*, the Province assumed the responsibility and authority for determining building elevations and/or other flood protection requirements to be provided by an individual developer or landowner wishing to build within these designated areas. The *Water Resources Administration Act* states that all structures (excluding fences) within Designated Flood Areas require permits from the Province, whose staff administer and enforce the flood proofing requirements. Should a structure be found to be in contravention of these requirements, the owner could be ordered to remove the structure and/or a caveat may be registered on the title of the property (which, among other things, would advise any future owner that the property will not be eligible for disaster assistance funding in the event of a flood). This regulatory system has apparently resulted in a compliance rate in excess of 90%, a significant improvement over past experiences along the Red River.

The Lower Red River Designated Flood Area, which encompasses lands along the Red River downstream of the City of Winnipeg to Netley Creek, as well as lands upstream from the Red River along Netley Creek and tributaries to PTH 9 at Petersfield, was designated in September 2011, as the result of open water and ice influenced flooding along the Lower Red River and Netley Creek in 1996, 2007, and 2009.

Based on the success of the Red River DFAs as noted above, DFAs are being viewed as the most effective way of ensuring that development controls in flood prone areas are regulated. As part of the Public Open House events, discussed in Section 25.0 of the report, feedback was received on the concept of DFAs. In general, individuals commenting on the implementation of development controls encouraged limiting urban development within DFAs to reduce the likelihood of future flood damages and the associated effects. Based on the feedback received, it was apparent that implementing DFAs should be considered in areas most vulnerable to flooding, particularly areas subject to increased development such as the Lower Assiniboine River, Lake Manitoba and Lake St. Martin, as well as Dauphin Lake.

It is clear the intent for better flood and water management exists within the programs implemented by the Province, the municipalities and the planning districts. These include Provincial Land Use Policies, municipal and planning district development plans and zoning bylaws and IWMPs. Effective implementation of these initiatives appears to be limited by financial and human resources. Given the success of the DFA programs noted above, the

Province should establish DFAs in areas where development conflicts with the objective of provincial flood control systems such as along Lower Assiniboine River. These would best be implemented in collaboration with the existing IWMPs and goals of the municipalities and planning districts. It is recognized that DFAs in areas such as the Lower Assiniboine will be complex, as they will need to consider the interaction between the flood protection level, existing developments, the operating rules for the Portage Diversion and future Lake Manitoba and Lake St. Martin Outlet Channels.

18.4 REVIEW OF OTHER JURISDICTIONS

18.4.1 Saskatchewan

The Government of Saskatchewan has a similar philosophy for development near watercourses or waterbodies, however, a significantly higher recommended level of protection against flooding. The *Planning and Development Act*, 2007 and the *Statements of Provincial Interest Regulations* (SPI) require that all new, official community plans and zoning bylaws must contain policies to address the management of lands subject to natural hazards including flooding, slumping and slope instability. This includes the requirement that development be prohibited in the floodway of the 1:500 year flood elevation of any watercourse or water body and that new development in the flood fringe of a 1:500 year flood be flood proofed to an elevation 0.5 m (1.6 ft) above the 1:500 year flood elevation. Saskatchewan's policy is different than Manitoba's in that the level of protection is much higher, and there are different development restrictions in the floodway and floodway fringe. Manitoba does not restrict development in the floodway as long as it is adequately flood proofed. To reduce future flood costs, Manitoba should consider adopting a policy similar to Saskatchewan's where development of new buildings and additions to buildings in the floodway should be prohibited.

The Government of Saskatchewan currently requires that all planning documents and decisions, insofar as is practical:

- Identify potential hazard lands and address their management.
- Limit development on hazard lands to minimize the risk to public or private infrastructure.
- Prohibit the development of new buildings and additions to buildings in the floodway (i.e., portion of the floodplain where the waters are projected to meet or exceed a depth of

1.0 m (3.28 ft) or a velocity of 1.0 m per second (3.28 ft/sec)) of the 1:500 year flood of any watercourse or water body.

- Require flood proofing of new buildings and additions to buildings to an elevation of 0.5 m (1.6 ft) above the 1:500 year flood elevation of any watercourse or water in the flood fringe (i.e., portion of the floodplain where the waters are projected to be less than a depth of 1.0 m (3.28 ft) or a velocity of 1.0 m per second (3.28 ft/sec)).

Saskatchewan planning and development policies are applied universally, but their application is directed towards the high risk and pressure areas such as the Qu'Appelle River where recreational properties were flooded in 2011 and 2014.

18.4.2 Alberta

Flooding in southern Alberta in June 2013 resulted in four fatalities and unprecedented damage to property. A review indicated that flood mapping efforts over the last four decades have identified 48 cities and towns across Alberta with an inundation hazard, but fewer than half of those were designated under Alberta's Water Act as hazardous. It appeared that towns could reject the designation of flood prone if they did not see any clear benefits. This led to inconsistent bylaws across the province; some municipalities restricted development in the floodway and others did not.

In the wake of the 2013 Flood, the following policies are being implemented by the provincial government when it comes to rebuilding:

- ***Buildings in Flood Risk Areas*** – For homes located within a floodway, there will be funding available through the Disaster Recovery Program for homeowners to rebuild or relocate to a new location outside flood-risk areas. There will also be funding available through the same program for specific mitigation infrastructure that will protect buildings within a flood fringe area.
- ***Extra money for Flood Mitigation*** – For heavily damaged homes requiring major repairs or complete reconstruction, homeowners in flood fringe areas will be eligible for additional funding through the Disaster Recovery Program, on top of the disaster recovery assistance for eligible expenses. This additional money must be spent on approved flood mitigation measures. Mitigation measures will be approved if they are sufficient to protect against a 1:100 year flood event.
- ***Future Coverage*** – Homeowners in a flood fringe who do not implement mitigation measures to protect against a 1:100 year flood event will not be eligible for Disaster Recovery Program assistance in the event of future flooding. Residents who undertake approved flood mitigation measures will be eligible to receive Disaster Recovery

Program assistance for any future flood that exceeds the 1:100 year flood event criteria. Homes and businesses currently in floodways and those who utilize provincial Disaster Recovery funds will have a notation on their land title to ensure that future owners of the property are also informed.

- ***Future Development in Floodways*** – Municipalities will no longer be allowed to approve future development in floodways, however, unique circumstances which some municipalities face will be accounted for.

Alberta believes that these new policies will align with federal flood assistance programs and bring the province in line with Manitoba, Ontario and Quebec, which have policies on development in flood-prone areas. The Alberta experience illustrates the need to consistently apply development controls across the province as well as the need for flood risk mapping.

18.4.3 British Columbia

In British Columbia, land-use management is provided through information published in the Flood Hazard Area Land Use Management Guidelines (2004). Up until 2003, the provincial government was directly responsible for reviewing and regulating municipal flood management. However, full responsibility shifted to the municipalities due to major legislative changes in 2003 and 2004. Municipalities are not legally required to incorporate standards from the Guidelines into their bylaws. Instead, the relevant provincial legislation requires only that municipalities consider the Guidelines when adopting a flood bylaw. British Columbia faces similar issues as other western jurisdictions:

- Flood protection, particularly non-structural measures is primarily the responsibility of local governments. With the enabling legislation delegating responsibility to local governments, the response has led to considerable variety in approaches to flood protection including a lack of implementation.
- There are particular resource limitations among small communities including available staff time and technical expertise.
- Conflicts or conflicting objectives sometimes arise between different jurisdictions having authority with respect to development in flood hazard areas. This is largely a result of differing priorities.
- There is a need for updated floodplain mapping and other technical flood hazard information. Floodplain mapping is an important area, which has not received adequate attention considering its importance.

BC's flood management has been largely standard-based, with a focus on particular flood magnitudes (the 200-year return period flood in general, and the flood-of-record for the Fraser River). BC appears to be heading towards a risk based approach to flood management because of climate change and land use. Flood risk combines the probability of a hazardous flood occurring and the potential consequences to elements at risk (the population, buildings or engineering works, economic activities, public services, utilities infrastructure and environmental features in the area potentially affected by floods or landslides (APEGBC, 2012).

18.4.4 National Floodplain Mapping Assessment and Ontario Preparation

A recent article published in the Fall 2015 *Canadian Dam Association Bulletin*, entitled "Advancements in Floodplain Mapping and Linkages to Dam Safety" (Boyd et al., 2015), provides an overview of the recent National Floodplain Mapping Assessment (NFMA), the National Disaster Mitigation Program (NDMP) and case studies funded by the Ontario Ministry of Natural Resources and Forestry (OMNRF).

In anticipation of the announcement of the NDMP program, in 2014 OMNRF invested in case studies related to floodplain mapping. The NDMP program established in April 2015 by the Federal Government has the following four funding streams available as stated in Boyd et al. (2015):

1. Risk Assessments.
2. Flood Mapping.
3. Mitigation Planning.
4. Investments in Non-Structural and Small Scale Structural Mitigation Projects.

Regarding flood mapping, it is interesting to note that the National Floodplain Mapping Assessment (NFMA) recently commissioned by Public Safety Canada had its final report published in June 2014. In his article, Boyd et al. (2015) recognized that prior to the NFMA, the last floodplain mapping program was the Federal Damage Reduction Program, which ran for 20 years from 1975 to 1995, and focused on identifying the outer regulatory and the 1:100 year flood lines. Amongst other recommendations, NFMA now recommends the consideration of a 1:350 year flood to define the regulatory floodplain. As stated in Boyd et al. (2015), events more severe than the 1:100 year event should be considered in hazard mapping and flood risk

assessment, particularly when one considers both the degree of uncertainty associated with the estimates and that statistically a 1:100 year event has a 65% chance of occurring in a 100-year period.

18.4.5 California

FloodSAFE California has a long-term strategic initiative developed to reduce flood risk referred to as FloodSAFE (2016). It was designed in recognition that addressing the risks of flood damage statewide would take decades. Integrated flood management is the basis of the program focusing on communities and resources that face a high risk of catastrophic damage. The Integrated Flood Management approach recognizes:

- The interconnection of flood management actions within broader water resources management and land use planning.
- The value of coordinating across geographic and agency boundaries.
- The need to evaluate opportunities and potential impacts from a system perspective.
- The importance of environmental stewardship and sustainability.

Under the program, California has begun the long process of improving flood management systems by investing heavily to complete emergency repairs near several high risk urban areas, evaluating levees that protect urban areas, informing the public about flood risks, enacting significant new laws, and providing funds to lead a sustained effort to improve flood management statewide.

Some short-term actions that do not require substantial resources include:

- Land use planning and decision making must consider flood management. This includes limiting development in floodplains.
- Federal and state agencies must improve planning and permitting processes to allow critical flood management planning, implementation, operations and maintenance action to proceed.
- Flood management projects must be broadened to deliver multiple benefits such as environmental and water supply benefits.
- Ongoing public agency outreach programs must inform policymakers at all levels of government about the risks and consequences of flooding.

Long-term solutions requiring immediate attention are:

- Sufficient and stable funding mechanisms must be developed to invest in public safety.
- Public funding for flood management requires alignment aimed at public agencies to deliver the most effective and economical multiple-benefit projects.

The role of the California Department of Water Resources (DWR) within FloodSAFE includes:

- Leading efforts to establish integrated flood management and emergency response systems throughout California.
- Providing state cost sharing funds to implement new projects.
- Implementing an environmental stewardship approach to develop integrated projects that produce benefits for multiple objectives.
- Developing and administering new grant programs to support regional and local efforts to meet the FloodSAFE foundational objectives.
- Continuing to administer existing state-sponsored flood management programs.
- Promoting legislative reforms that support accomplishing FloodSAFE goals and objectives.
- Evaluating the Central Valley Flood management system.
- Developing funding plans to fully accomplish the FloodSAFE Vision.

Examples of program accomplishments are:

- The best available 1:100 year and 1:200 year floodplain maps and levee Flood Protection Zone maps were developed and publicly disseminated.
- New building code requirements for residential structures and schools in the Central Valley 1:200 year floodplain were developed and submitted to the Building Standards Commission.
- About two million cubic yards of sediment was removed from the Tisdale Bypass and one million from the Fremont Weir.
- Comprehensive flood emergency training and exercises have been developed and are being implemented.
- Over 300,000 Flood Risk Notifications were mailed.
- A Flood Protection Plan (CVFPP) for adoption by the Central Valley Flood Protection Board (CVFPB).
- Sending annual flood risk notifications to property owners protected by levees.

- Preparing project levee status reports.

18.5 INVENTORY OF POTENTIAL DEVELOPMENT CONTROLS

This section contains a list of possible development controls that could assist in reducing flood costs. Floodplain policies are now multi-purpose and result in a mix of many solutions. Development controls manage and regulate property development to ensure it conforms to a set of policies or standard(s) in place. Alternative options to development controls are penalties or disincentives for noncompliance. The list was developed through a review of public policies, strategies and programs in place in Manitoba and other jurisdictions, and from TAC, stakeholder and government departments input. Many may be not be practical in the Manitoba context. Using this list the most relevant to Manitoba were recommended in Section 18.6.

The list of potential development controls is follows:

- Strengthening the linkage between development plans and IWMPs.
- Regulations to prohibit development in the floodway of the 1:200 year flood elevation of any watercourse or water body. Only new development in the flood fringe of a 1:200 year flood, which has been flood proofed to an elevation 0.6 m (2.0 ft) above the 1:200 year flood elevation would be permitted.
- Flood buyouts (homeowners to rebuild or relocate to a new location outside flood-risk areas). Property acquisition may be a cost-effective alternative to structural protection of buildings facing extreme hazards such as in floodways.
- Require flood proofing to current standards for future Disaster Financial Assistance eligibility.
- Funding available for modifying or retrofitting existing buildings within a flood fringe area.
- Require new development(s) to have a lot grade certificate for Disaster Financial Assistance eligibility.
- A notation on their land title to ensure that future owners of the property are also informed that it is in a flood risk area. This may involve a notice placed on Land Titles stating if the property was eligible for past Disaster Financial Assistance funding.
- Deny Disaster Financial Assistance for property damage due to flooding when the property owner has indemnified the local government from any flood loss. This means that if a local government has warned a property that it is in a high-risk zone and the property has accepted the risk, they are no longer eligible for assistance.
- Building Codes requiring flood-resistant construction for new buildings in flood-prone areas.

- Require municipalities or planning districts to periodically send out Flood Risk Notifications to flood prone properties.
- On property tax statements, add a surcharge to flood risk properties to cover expenditures by municipal or planning districts that are not eligible for Disaster Financial Assistance.
- New development should aim to be in harmony with the water environment and not attempt to work against it.
- Planning districts should enforce development controls through stronger penalties. Municipalities currently have the ability to levy fines for non-compliance but this does not appear to be a deterrent, as they are not being issued and/or are not strong enough.

Other initiatives identified through the review were:

- The need for reinstating a provincial floodplain mapping program. A floodplain map is a technical tool that provides information on the area that is expected to be flooded (i.e., a 200-year flood).
- An education program on flood proofing could be developed to disseminate best practices for the mitigation of risks at the property and community levels.
- Improved sharing of information concerning FPLs between government departments (i.e., EMO, MIT, and Municipal Governments).

18.6 CONCLUSIONS AND RECOMMENDATIONS

Manitoba's development controls to minimize the risk of flood damages were reviewed and compared to other jurisdictions. Key components of Manitoba's development controls include:

- The use of the 1:200 year event or the highest flood on record if greater than the 1:200 year event as the provincial flood protection level.
- The designation of Designated Flood Areas (DFAs), where the Province regulates flood protection of new developments.
- Municipal and planning district development plans and zoning bylaws.

The review indicated that to minimize the risk of flood damages, it was recommended the Province's long-term flood protection strategy should be based on the 1:200 year level of protection or the Flood of Record, if it is higher, as a minimum. A higher level of protection could be justified on a risk based assessment. Regulations should be developed to prohibit development of new buildings and additions to buildings in the floodway of the 1:200 year flood elevation of any watercourse or water body. Only new development in the flood fringe of a

1:200 year flood, flood proofed to an elevation 0.6 m (2.0 ft) above the 1:200 year flood elevation would be permitted.

In addition to the recommendation for a 1:200 year level of protection, other supports and mechanisms that should be considered to make the implementation of development controls by municipalities and planning districts easier and more successful were:

- Prohibiting the development of new buildings and additions to buildings in the floodway.
- Strengthening the linkages been IWMP and Land Use Planning would ensure the IWMP recommendations related to riparian and shoreline protection, flooding and surface water management were adequately considered and adopted into land use plans. Consideration should be given to adding a regulation under the *Water Protection Act* that requires RMs and planning districts to incorporate recommendations from IWMPs into their development plans.
- The need to reinstate the 1980s provincial floodplain mapping program. Floodplain mapping identifies flood hazard areas that need to be regulated by development controls by municipalities and planning districts. The mapping would also provide technical information to assist with the development of IWMPs. The floodplain mapping should be readily available to the public.
- New development(s) in flood hazard areas should require a lot grade certificate. Implementation options are to have a qualified professional certify the final lot grade or the developer(s) or owner provide a financial deposit until the FPL grade is verified by a professional as compliant. The municipality or planning district would issue the lot grade certificate.
- The lot grade certificate would be a requirement for Disaster Financial Assistance eligibility for all new development(s).
- A notation on their land title to ensure that future owners of the property are informed that it is in a flood risk area. This may involve a notice placed on the Land Title indicating if the property was eligible for past Disaster Financial Assistance funding.
- The Province should establish Designated Flood Areas in areas where development conflicts with the objective of provincial flood control systems such as along the Lower Assiniboine River. It is recognized that Designated Flood Areas in areas such as the Lower Assiniboine will be complex as they will need to consider the interaction between the flood protection level, existing developments and the operating rules for the Portage Diversion and future Lake Manitoba and Lake St. Martin Channels.
- Ongoing education of the public about flood hazards and flood protection policies and standards.
- Improved sharing of information concerning FPL between government departments (i.e., EMO, MIT and Municipal Government).

Because of the success of implementing Designated Flood Areas on the Red River, Designated Flood Areas are viewed as a more effective way to impose development controls in flood-prone areas. On this basis, as an alternative to delegating the responsibility to local governments, consideration should be given to designating the urban areas along the Souris and Assiniboine Rivers, and properties around Lake Manitoba and Lake St. Martin, as well as Dauphin Lake as Designated Flood Areas.

For these study recommendations to be to successfully implemented, government(s) will require additional resources. As well, all policies recommended will need to be accompanied by regulations that will facilitate their implementation.

19.0 IMPACT OF LAND USE PRACTICES

19.1 INTRODUCTION

This section examines potential land use changes and policies that could assist with preserving, expanding or enhancing areas with low runoff potential. The management of land and water is inextricably linked and has the potential to impact runoff from the area. Agriculture is the dominate land use in the Assiniboine River / Lake Manitoba Basin. Other land uses are urbanization, forestry, park, and other relatively natural land uses.

Land use changes that may impact the basin's runoff potential involve urbanization, deforestation, agricultural intensification, and grassland and habitat conservation. The potential impacts of land use changes are discussed in this section. These changes were considered separately from the impacts of wetland drainage discussed in Section 20.0. The 2011 Flood Task Review Report (FRTF, 2013) made little or no mention of other land use changes, as it focussed on the impacts of wetland drainage. Options to preserve, expand or enhance areas with low runoff potential were generated through a review of public policies, strategies and programs in place in Manitoba and other jurisdictions, and from TAC, stakeholder and government departments input.

19.2 IMPACT OF LAND USE CHANGE

The relative impact of land use may be illustrated through a series of streamflow simulations that examined the sensitivity of spring runoff to the changes in land use in the Smith Creek watershed (Pomeroy et al., 2010). The approximately 435 km^2 (168 mi^2) watershed located in eastern Saskatchewan within the Upper Assiniboine River Basin has undergone rapid changes over the past 50 years, transitioning from areas of extensive forests and wetlands to grain and oilseed cultivation.

Forest and agricultural land use conversion scenarios were simulated. Forest conversion involved converting fallow and stubble fields to tree woodlands (forest). Agricultural conversion involved converting grassland and tree woodlands (forests) to fallow and stubble. The simulations indicated there was a 5% decrease in the watershed's long term spring discharge

volume for the conversion to forest and a moderate 19% increase for the conversation to agricultural.

The impact on peak daily discharge was also examined. The simulations indicated that the forest conversion and agricultural conversion scenarios lead to a higher peak daily discharge. Forest conversion had an increase in the peak discharge by about 3.5 m^3/s (124 cfs). The agricultural conversion increase in peak daily discharge was about half this amount.

19.3 LAND USE IMPACTS ON THE ASSINIBOINE BASINS AND LAKE MANITOBA

The impact of land use change, current policies and regulations to manage them, and potential policies or opportunities to enhance or expand areas with lower runoff potential are discussed in this section.

19.3.1 Urbanization

The effects of urbanization are generally well understood and strongly influenced by local management. Urban development provides a useful illustration of some of the most obvious effects of land use change on water management. Vegetated soils are replaced with impermeable surfaces increasing overland flow and reducing infiltration, bypassing the natural storage and attenuation of the subsurface. In addition, the conveyance of runoff to streams is modified. Overland runoff is conventionally collected by piped storm water drainage systems and conveyed rapidly to the nearest stream. The result is a greater volume of runoff, discharging in a shorter time, potentially leading to dramatically increased flood peaks, but also reduced low flows and less groundwater recharge.

Urbanization represents a very small portion of the Assiniboine and Lake Manitoba Basins. The impact of urbanization may be mitigated by the promotion of storm water retention. Urbanization also may result in floodplain development, which may restrict flow and increase water depths and flood damages from the loss of floodplain storage. Mitigation strategies include:

- Applying existing controls on development more strongly in flood prone lands and updating building codes to minimize flood damage by requiring buildings be built to an appropriate flood level (see Section 18.0, Development Controls for more details).

- Allowing use of the floodplain for sport and outdoor recreational use, amenity open space or for nature conservation purposes where this will not materially increase flood risk.
- Maintaining the natural functions of the floodplain.

19.3.2 Deforestation

The impacts of deforestation are more subtle than urbanization. Deforestation in the Assiniboine and Lake Manitoba Basins is occurring on both private and Crown lands. On private lands, agricultural development has occurred with encroachment and fragmentation of forested areas and wholesale clearing of small woodlands. This has the potential to generate increased runoff. On the escarpment of the Turtle, Riding, Porcupine and Duck mountains, deforestation has the potential to significantly increase erosion, runoff and flooding. Preservation of woodlands may be through habitat retention programs, which would secure habitat for the long-term (i.e., Ducks Unlimited and Manitoba Habitat Heritage Corporation). A policy should be considered for regulating forest clearing and conversion to ensure protection of the landscape along the base of Manitoba escarpment from erosion and flooding. The policy would need to be accompanied by regulations that would result in actual implementation.

To address erosion, Conservation Districts (CDs) have undertaken a number of actions as follows:

- Protect and manage riparian areas to reduce streambank erosion and siltation.
- Rehabilitate streams and alluvial fans to reduce stream gradients and improve infiltration, in order to minimize stream bed and bank erosion.
- Provide access to land protection programs that maintain natural or permanent cover. Focus on at-risk areas characterized by sensitive soils, shelterbelts, woodlands, wetlands, and riparian areas.
- Implement erosion control measures appropriate for steep slopes in this area.
- Installation of erosion control structures, shale removal and drain maintenance.
- Provide incentives to establish permanent cover; install gradient control structures to dissipate hydraulic energy in straightened reaches; re-establish channel meanders; develop a program to purchase land and provide financial incentives to landowners for restoration and maintenance of key alluvial fan and shale trap areas.
- Provide access to land protection programs that maintain natural or permanent cover, focusing on the erodible slopes of this zone.

19.3.3 Agricultural Intensification

This involves the conversion of grasslands to cultivation and/or marginal lands being converted to cultivation. Cultivation practices such as fallow or stubble impact the runoff response. Grasslands slow runoff and increase infiltration into the soil. There are no policies or regulations regarding agricultural intensification; it is the individual agricultural producer's choice. Consideration should be given to establishing a policy to preserve natural grasslands where an individual could receive a property tax credit for the ecological goods and services provided by the grassland. An example is the Rural Municipality of Dufferin's wetland tax credit. The Council of the RM of Dufferin passed a by-law indicating that the land owner will receive $40 per acre for each eligible acre. The minimum to be eligible for the program is 0.4 ha (1 ac). The program is also available to land owners willing to re-establish wetlands that had previously been drained. The RM of Dufferin originally believed they would be able to preserve approximately 100 ha (250 ac) of wetlands over the next three years with a budget of $10,000, however, with a generous contribution from the LaSalle Redboine Conservation District of $15,000 over three years, the number increased to 150 ha (365 ac) that are currently being preserved.

Alternatively, an improvement tax for the loss of the ecological goods and services the natural grassland provides could be charged. Agricultural producers should be encouraged to follow best management practices. Under Growing Forward 2: Growing Assurance, financial assistance is provided to Conservation Districts (CDs) to work with producers to implement beneficial management practices (BMPs) on farms to conserve and enhance ecological goods and services (EG&S) on the agricultural landscape (see Section 19.4.4 for details).

19.4 LAND USE PLANNING – MANITOBA

19.4.1 Manitoba's Surface Water Management Strategy

The goal of Manitoba's Surface Water Management Strategy was to develop sustainable, watershed-based planning across the entire Province to ensure that future management of specific water issues is done carefully (Manitoba PowerPoint, 2016).

A sustainable approach will ensure that all needs are met, while maintaining ecosystem protection. The Strategy identified six inter-related policy areas:

1. Water quality.
2. Conservation.
3. Use and allocation.
4. Water supply.
5. Flooding.
6. Drainage.

A number of specific actions have been identified to address issues within each policy area and many of these are presently underway. The strategy does endorse the principals associated with managing land use for the purpose of reducing runoff, however, it focuses on regulatory approaches for wetlands. Other land use changes associated with cultivation of grasslands and deforestation should be emphasized. Applicable land management strategies to reduce potential runoff from land are:

- **Protected Areas** – Increase the number and extent of small protected areas in Agro-Manitoba to conserve remnant prairie, vulnerable wetlands of significance, and fragmented habitat and habitat corridors for species at risk through the new Protected Areas Strategy currently being developed. The Province will continue to work with partner agencies to secure conservation agreements on privately owned land to protect valuable habitat and natural areas in perpetuity.
- **Crown Lands** – Minimize drainage of Crown land, and where appropriate, use this provincial resource to retain water in the watershed to improve drought resiliency, reduce flood risk, improve water quality and support healthy ecosystems.
- **Basin Water Retention** – Provide support and technical expertise to complete the Assiniboine Basin and Lake Manitoba Flood Mitigation Study, which is conducting basin-level assessments that include identification of potential water storage areas at various scales for multiple uses. Follow up with implementation of recommendations for additional water retention and storage.
- **Small Water Retention** – Work with conservation districts and agricultural producers to target small scale watershed and on-farm water retention and storage aspects of their programming, including options to provide incentives to preserve and restore wetlands on privately owned land.

19.4.2 Forest Management Plan (FMP)

Logging on Crown land is regulated by the Province. To minimize the impact on the environment, a Forest Management Plan (FMP) is required that describes planned timber harvesting, access development, and forest renewal activities. The FMP usually contains best management harvesting practices to minimize the risk to Province's natural resources. The Forest Management Plan, which is a requirement of a company's Forest Management Agreement with the Province of Manitoba, requires approval by Manitoba Conservation and Water Stewardship and a license under *The Manitoba Environment Act*. The FMP is reviewed by various branches and departments of government. Depending on the project, an Environmental Impact Assessment of the Forest Management Plan and a separate Environmental Impact Statement submitted to Manitoba Conservation and Water Stewardship may be required. Land use impacts and opportunities for enhancement can be managed through these processes.

19.4.3 Manitoba Habitat Heritage Corporation

The Manitoba Habitat Heritage Corporation (MHHC) was established as a Crown Corporation by the *Manitoba Habitat Heritage Act* in 1986 to "Conserve, restore and enhance fish and wildlife habitat". MHHC is a non-profit Crown Corporation that works in partnership with public and private agencies to conserve, restore and enhance fish and wildlife habitat in Manitoba.

MHHC's primary tool for long-term habitat conservation is the conservation agreement (CA). Essentially an easement, a CA enables MHHC to enter into an agreement with a landowner for the purpose of protecting the habitat without affecting surrounding land use or the parcel's ownership. The agreement is perpetual and recorded on the land title, remaining in force even if the land changes hands. Most areas that have a CA attached may continue to be used for agricultural activities such as haying and grazing as long as the natural values are protected. Habitat protected by these agreements includes areas that protect water quality, allow wetlands to flourish, support endangered species and provide aquatic habitat and riverbank protection.

MHHC works with local watershed planning authorities and conservation districts to focus its programs on established priority areas within each watershed. Whether it is preventing erosion

and washouts on the slopes of the Turtle Mountains or protecting riparian areas along the Whitemud River, MHHC works with local partners to achieve local results.

MHHC has worked extensively to protect, enhance and restore wetland habitats and their inhabitants while still allowing compatible agricultural production to continue. MHHC protects habitat for specific species at risk through the Habitat Stewardship Program (HSP) for Species at Risk, a voluntary, stewardship-based approach for protecting habitat for threatened species, and support activities specific to individual species. MHHC have focused largely on the protection of existing habitat in agriculturally developed regions, where habitat loss and disturbance has been most significant. MHHC ensures that healthy riparian areas continue to function as filters for nutrient-rich runoff water through the Riparian Conservation Agreements Program in partnership with Manitoba Conservation and Water Stewardship and MHHC.

The corporation disseminates information to the public respecting Manitoba habitat and fish and wildlife populations.

19.4.4 Conservation Districts (CDs) as Watershed Authorities

The Manitoba Conservation District Program was has taken on the role of Watershed Authorities as defined by the *Water Protection Act*. These Authorities were created to provide healthy and sustainable watersheds through focused, priority-based programs that provide definite improvements to watershed health. The program:

- Functions as a partnership between levels of government and the engagement of local citizens.
- Is cost-shared by provincial and municipal government partners who provide long-term core funding.
- Is incentive-based and non-regulatory.
- Relies on local decision-making through conservation district boards and sub-districts. Local people work together with technical and science experts to solve local issues.
- Is watershed-based and uses watershed boundaries to make planning and management decisions and provide program delivery.

Each CD offers a unique suite of programs that are tailored to address local watershed issues. Examples of programs include but are not limited to the implementation of on-farm beneficial

management practices, enhanced coordination and implementation of surface water management activities, the improvement of ground and surface water quality, water conservation, and the protection of natural areas.

CDs may partner with Manitoba Habitat Heritage Corporation, Ducks Unlimited Canada, Nature Conservancy or Municipalities to protect sensitive habitat through Conservation Agreements (easements in perpetuity). Sometimes the land is donated, but at other times, a one-time payment may be made to the landowner or they may be provided with a tax credit for the donated lands. The landowner still holds ownership of the land; but under the agreement, there may be restrictions on the use of the land (e.g., It can only be used for grazing livestock during certain periods of the year, and it cannot be used for annual crop production.).

One CD (Little Saskatchewan River CD) partners with ALUS Canada to continue the ALUS pilot project (alternate land use services), which provides annual payments to landowners to protect or enhance Ecological Goods and Services in their district. Landowners receive funding from the federal government for this program.

Another CD (Turtle Mountain CD) is developing a new pilot project with a couple of their municipalities, which will tax landowners who have drained wetlands on their property. Those tax dollars will go back into securing wetlands and enhancing watershed health. MHHC provided wetland inventory data recently, which will help identify where historical wetlands were located and where drainage improvements have occurred. Details of this pilot project are still being developed.

Under Growing Forward 2: Growing Assurance – Ecological Goods and Services Program, financial assistance is provided to Conservation Districts (CDs) to work with producers to implement beneficial management practices (BMPs) on farms to conserve and enhance ecological goods and services (EG&S) on the agricultural landscape (Manitoba, Growing Assurance, 2016). This program focuses on water quality and provides other important environmental services derived from well-managed stocks of natural capital within the Lake Winnipeg Watershed. Conservation Districts work with farmers and submit applications for on farm projects within their respective CDs (component one) or potentially as joint proposals

(component two). Using a coordinated approach, the CDs implement BMPs to improve critical watershed processes. Eligible practices are:

- Water Retention Structures.
- Wetland Restoration.
- Constructed Wetlands.
- Riparian Area Enhancement.
- Natural Area Maintenance and Enhancement.
- Buffer and Grassed Waterway Establishment.
- Perennial Cover for Sensitive Land.
- Shelterbelt / Tree Establishment.

19.4.5 Provincial Crown Lands

The Crown Lands and Property Agency (CLPA), a special operating agency under MIT, has full authority for all Crown land sales, leases, and permits, as well as land appraisal, acquisition, expropriation and other related real estate services to government. Manitoba Conservation and Water Stewardship is responsible for land use policy and planning, designating Crown land uses including Crown land to be sold or rented, compliance with the legislation and authorizing work to be performed on Crown lands. Manitoba Agriculture, Food and Rural Development is responsible for agricultural land use policy and planning, compliance with the legislation and authorizing work to be performed on agricultural Crown lands.

Crown lands are coded to prescribe limits to land use and development, identify length of commitments and outline required permissions. Decisions on Crown land dispositions under The *Crown Lands Act*, including sales and leases, are based on the suitability of the land for the proposed use and disposition under Crown Land Policy. Crown Land Policy sets out land suitability criteria. Applications to lease or purchase Crown land are reviewed by subject matter experts within various departments, as well as by external governments and agencies. Information from this review is assessed on a case by case basis to determine whether or not the disposition would be consistent with Crown Land Policy. If one or more of the land suitability criteria are not met (for example the land is flood-prone, erosion-prone or required for watershed protection), the parcel must not be sold.

It would appear that Provincial policies are in place to deal with sales, leases and permits. The review process and its comments are a critical component of the process as once the land is sold, the province no longer directly controls its use and development. The circulation process has been reviewed and improvements to the process are anticipated.

During the study, concerns were raised about the use and sale of Crown lands and their impact on flooding. Crown lands which are sold and become private lands become subject to the same requirements to follow all applicable federal, provincial and municipal regulations. For example, if an individual wished to improve their private land through drainage, which may cause increased runoff and associated downstream flooding and erosion, they are required to submit an "Application for Licence to Construct Water Control Works" under The *Water Rights Act*, which is a similar requirement for all private land holdings. Similarly, an individual would be expected to comply with any other legal requirements set out by legislation.

Another issue raised was the sale of flood prone lands within the Shoal Lake Basin. The Shoal Lakes Basin is a closed basin and experienced historic flooding in the 2000s. The sale of Crown Lands was prior to that period, and there was no knowledge of the Crown lands being historically flood prone. The current knowledge of the flooding should prevent this situation reoccurring.

19.5 LAND USE PLANNING – OTHER JURISDICTIONS

19.5.1 United States Department of Agriculture (USDA) Farm Service Agency

19.5.1.1 Grass Land Reserve Program

The Grasslands Reserve Program (GRP) prevents grazing and pasture land from being converted into cropland, used for urban development, or developed for other non-grazing uses. Participants in the program voluntarily limit future development of their grazing and pasture land, while still being able to use the land for livestock grazing and activities related to forage and seed production. The GRP has multiple enrollment options including a rental contract for 10, 15, or 20 years or enrollment of the land in a conservation easement for an indefinite period of time. Applicants must own and be willing to restrict use of the land either in perpetuity or under the

maximum length of time under state law. Those enrolled in GRP receive an annual rental payment for their enrolled acres (USDA, FSA, GRP, 2016).

19.5.1.2 Reserve Enhancement Program

The Conservation Reserve Enhancement Program (CREP) targets high-priority conservation issues identified by local, state, or tribal governments or non-governmental organizations. In exchange for removing environmentally sensitive land from production and introducing conservation practices, farmers, ranchers, and agricultural landowners are paid an annual rental rate. Participation is voluntary, and the contract period is typically 10 to 15 years, along with other federal and state incentives as applicable per each CREP agreement.

CREP programs allow critical conservation issues to be addressed while at the same time meeting regulatory requirements. With the federal and state resources made available through this program, this partnership is in place to address a high priority environmental problem. Those enrolled in CREP receive an annual rental payment for their enrolled acres. FSA also provides cost-sharing and other incentives to help offset the costs associated with putting these practices in place (USDA, FSA, CREP, 2016).

19.5.1.3 Conservation Reserve Program

The Conservation Reserve Program (CRP) is a land conservation program in which, in exchange for a yearly rental payment, farmers enrolled in the program agree to remove environmentally sensitive land from agricultural production and plant species that will improve environmental health and quality. Contracts for land enrolled in CRP are 10 to15 years in length. The long-term goal of the program is to re-establish valuable land cover to help improve water quality, prevent soil erosion, and reduce loss of wildlife habitat (USDA, FSA, CRP, 2016).

19.5.2 USDA Natural Resources Conservation Service

19.5.2.1 Conservation of Private Grazing Land Program (Farm Bill 2002)

The Conservation of Private Grazing Land Program (Farm Bill 2002) is a voluntary program that provides technical and financial assistance to landowners and others to develop upland, wetland, riparian and aquatic areas on their property. Most efforts have concentrated on improving upland habitat such as native prairie.

19.5.2.2 Grassland Reserve Program (Farm Bill 2002)

The Grassland Reserve Program (Farm Bill 2002) is a voluntary program that helps landowners and operators restore and protect grasslands including rangeland, pastureland, shrub land and certain other lands while maintaining them as grazing lands. Participants voluntarily limit future development and crop uses of the land while retaining the right to conduct grazing practices. Enrolment options include: permanent easement, 30-year easement, rental agreements, and restoration agreements. Contracts and easements prohibit the production of crops (other than hay), fruit trees, and vineyards that require breaking the soil surface and any other activities that do not advance conservation goals at the landscape-level.

19.5.2.3 USDA 2014 Farm Bill Agricultural Conservation Easement Program

Under the 2014 Farm Bill, the Agricultural Conservation Easement Program (ACEP) was created to protect critical wetlands and encourage landowners to keep lands in farming and ranching. Financial assistance is provided to eligible partners for purchasing Agricultural Land Easements that protect the agricultural use and conservation values of eligible land. The program protects grazing uses and related conservation values by conserving grassland including rangeland, pastureland and shrub land. Under the Agricultural Land component, up to 50% of the fair market value of the agricultural land easement may be paid. Where grasslands of special environmental significance will be protected, up to 75% of the fair market value of the agricultural land easement will be paid. Land eligible for agricultural easements includes cropland, rangeland, grassland, pastureland and non-industrial private forest land.

Under the bill, 100% of the cost of a permanent easement's value for the purchase of the easement is paid and, additionally, between 75 to 100% of the restoration costs are paid. Under 30-year easements, 50 to 75% of the easement value for the purchase of the easement is paid with, additionally, between 50 to 75% of restoration costs. Each easement is required to have an agricultural land easement plan that promotes the long-term viability of the land (USDA, NR, 2016).

19.5.3 U.S. Fish and Wildlife Service – Grassland Easement Program

The U.S. Fish and Wildlife Service – Grassland Easement Program is a voluntary program that pays landowners to permanently keep land in grass through legal agreements (easements) with the U.S. Fish and Wildlife Service. Mowing, haying and grass seed harvesting must be delayed until after July 15 each year to help grassland nesting species to complete their nesting before the grass is disturbed. Grazing is allowed.

19.5.4 North Dakota Land Protection Plan

The U.S. Fish and Wildlife Service has established the Dakota Grassland Conservation Area and is working with private landowners to accelerate the conservation of native prairie, both wetland and grassland habitats, in the eastern parts of North Dakota and South Dakota (US FWS, 2016).

The conservation area is an easement program to preserve, at a landscape scale, the ecological integrity of the area's mixed-grass prairie, tallgrass prairie, prairie pothole wetlands, and riparian woodlands with the support of the associated ranching culture.

The conservation easement contracts specify perpetual protection of habitat for trust species and limits on residential, industrial, or commercial development. The contracts prohibit alteration of the natural topography, conversion of native grassland to cropland, drainage of wetland, and establishment of game farms. Easement land remains in private ownership with property tax and invasive plant control the responsibility of the landowner, who also retains control of public access to the land. The contracts do not restrict grazing on easement land.

19.5.5 Nature Conservancy Canada

In Manitoba, the Nature Conservancy conserves and protects almost 63,000 ac (25,495 ha) across eight natural areas critical to Manitoba's biodiversity (NCC, 2016). To ensure long-term protection of their properties, landowners may enter into conservation covenants, which are voluntary, legal agreements between landowners and conservation organizations where the landowners promise to protect the land in specific ways. The promises the landowner makes are attached on the title to the land forever, regardless of who owns the land. In return, the conservation organization agrees to monitor the covenant and ensure that the intentions and objectives of the covenant are being maintained. Covenants are very flexible documents that can contain a number of provisions from limiting subdivision and development to committing to the removal of noxious weeds. The covenant applies to every future landowner. The conservation organization works to ensure the terms are enforced (TLC BC, 2016).

19.5.6 Canada-British Columbia Environmental Farm Plan Program

A voluntary program led by the British Columbia Agriculture Council (BCAC) (with Ducks Unlimited Canada as a partner) whereby producers undergo a process to identify environmental strengths and any potential risks on their farms. Once an Environmental Farm Plan (EFP) is approved, producers are eligible to apply for cost-shared incentives under the National Farm Stewardship Program and Greencover Canada. This cost-shared funding will help implement the BMPs (Beneficial Management Practices) that address the environmental risks identified in each plan. There are several categories that apply to grassland and rangeland including:

- Riparian Area Management (GREENCOVER).
- Land Management for Soils at Risk.
- Enhancing Wildlife Habitat and Biodiversity.
- Species at Risk.
- Grazing Management Planning (GREENCOVER).
- Biodiversity Enhancement Planning (Wilson, 2009).

19.5.7 Integrated Watershed Management

More recently, the emphasis has shifted to Integrated Flood Management, a mix of structural and non-structural methods. A review of California's Flood Risk Management (California, 2009) has shown that beneficial land use practices minimize development in flood prone areas (e.g., floodplains, alluvial fans, and low-lying areas) and enhance the ability of undeveloped floodplains and other open spaces to absorb, store, and slowly release floodwaters during small and medium-sized events (California, 2009). The application of Integrated Flood Management extends the range of strategies that may be employed to include those that may be primarily focused on other water resource issues (such as water supply or water quality) but have the potential to provide ancillary flood management benefits. The strategies that may be implemented to manage flood risk within a hydrologic region or watershed will vary depending on the physical attributes of the area, the presence of undeveloped floodplains, the type of flood hazards (e.g., riverine, alluvial fan), and the areal extent of flooding. Integrated resource management strategies that may reduce flood risks are:

- Agricultural Lands Stewardship – which can enhance the ability of lands to absorb rainfall and runoff, limit soil erosion, reduce the magnitude of some flood events, and enhance recovery after flood events.
- Ecosystem Restoration – can enhance the ability of open spaces to absorb rainfall and runoff and reduce the magnitude of flood events, or enhance recovery after flood events.
- Forest Management – including preservation of forest meadows, riparian vegetation along streams, and maintenance of forest canopy may reduce soil erosion and moderate the severity and frequency of some flood events and promote recovery after flood events.
- Watershed Management – can promote the retention of open space and habitat, which may reduce the severity and frequency of flood events and promote recovery after flood events.

19.5.8 Minnesota Prairie Conservation Plan 2010

Minnesota's conservation partners in the Prairie Region of the state collaborated to develop a 25-year strategy for accelerating conservation (MPCP, 2010). This strategy was precipitated by:

- Continuing loss and degradation of prairies, grasslands, wetlands and associated habitats along with the fish and wildlife dependent upon them.

- An acknowledged need to better coordinate between programs and organizations to maximize efficiency.
- Tremendous opportunities provided by the passage of the Clean Water, Land and Legacy Amendment by voters in 2008 that will provide significant conservation funding through 2034.

The plan calls for three approaches to conservation in the Prairie Region of the state. First, core areas with a high concentration of native prairie, other grasslands, wetlands, and shallow lakes were identified. Within these core areas, partners will work to ensure a minimum of 40% grassland and 20% wetland with the remainder in cropland or other uses. Second, habitat corridors connecting core areas were designed that included grassland / wetland complexes (23.3 km^2) (9 mi^2) in size at about (9.7 km) (6 mi) intervals along and within the corridors. Within the corridor complexes, a goal of 40% grassland and 20% wetland was set; and for the remainder of the corridors, 10% of each legal land section was to be maintained in permanent perennial cover. Third, in the remainder of the Prairie Region, a goal to maintain 10% of each Land Type Association in perennial native vegetation was established.

Based on this framework and background, the following was proposed:

- Permanent protection through the acquisition from willing sellers of fee title or easement of native prairies, wetlands and other habitats (including land to be restored).
- Restoration activities of grasslands, wetlands and other habitats.
- Enhancement of prairies and grasslands via prescribed fire, conservation grazing, haying and invasive species control. Enhancement of existing wetlands and shallow lakes through control of invasive species and intensive water level management was also included.
- Incorporation of conservation into “working lands” so that some conservation lands would contribute directly to local economies via “grass-based” agriculture, and agricultural lands in turn provide some natural resources benefits as a result of applying / using the full range of conservation practices.

19.5.9 Summary

Manitoba’s land use strategies for preserving, expanding or enhancing areas with low runoff potential appear to be in line with what other jurisdictions are doing to protect habitat loss. There appears to be a need for additional programing to protect and restore grasslands. Better

coordination between programs and organizations is suggested to maximize efficiency and long term strategies for accelerating conservation of land with low runoff potential.

19.6 INVENTORY OF POTENTIAL LAND USE CHANGES

This sections contains a list of possible land use options and strategies that could assist in preserving, expanding or enhancing areas with low runoff potential. This list, which follows, was developed through a review of public policies, strategies and programs in place in Manitoba and other jurisdictions, and from TAC, stakeholder and government departments input. Many may be not be practical in the Manitoba context. Using this list, the most relevant to Manitoba were recommended in Section 19.7. Possible land use options and strategies that could assist in preserving, expanding or enhancing areas with low runoff potential are:

- Retain Crown land as a provincial resource to improve drought resiliency, reduce flood risk, improve water quality and support healthy ecosystems.
- Use of caveats on Crown land sales to maintain the natural ecological goods and services of the land.
- Conservation agreements to protect the EG&S of the land in perpetuity. A one-time payment to the landowner or provide them with a tax credit for donated lands. The landowner still holds ownership of the land; but under the agreement, there may be restrictions on the use of the land (e.g., It can only be used for grazing livestock during certain periods of the year, and it cannot be used for annual crop production.).
- Conservation easements, which are permanent or for 30 years to protect the EG&S of the land. Under conservation easement for an indefinite period of time, applicants must own and be willing to restrict use of the land either in perpetuity or under the maximum length of time.
- Annual rental payments to maintain the EG&S of the land. For an annual payment, the use of the land is restricted in perpetuity or for the maximum length of lease.
- Land Donation – This is relatively simple and has financial benefits to the landowner. By execution of a deed, the owner gives his or her land (or a specified part of it) for conservation purposes to a qualified non-profit organization or governmental agency.
- Enhancing programs such as Growing Forward 2: Growing Assurance that provides financial assistance to agriculture producers for adoption of targeted beneficial management practices (BMPs) that improve the environmental sustainability of their operations.
- Pay for loss of Ecological Goods and Services (EG&S) through property taxes. If the purpose of the land changes, its value changes.

- Annual payments to landowners to protect or enhance EG&S on their lands.
- Property tax credit on lands that are planted to trees to store carbon or water. Similar to the Rural Municipality of Dufferin's wetland tax credit.
- Tax landowners who have improved property resulting in a loss of EG&S. Those tax dollars will go back into securing, protecting and enhancing lands to mitigate for the loss.
- Require a permit for the development of the land for agriculture, which results in a loss of EG&S of the land.
- Financial assistance to landowners to implement beneficial management practices (BMPs) on farms to conserve and enhance ecological goods and services (EG&S) on the agricultural landscape.
- CDs identify lands for removal and protection based on their contribution to the watershed in agriculturally developed regions, where habitat loss and disturbance has been most significant.
- Designating Development: For those landowners who desire to develop their property for residential and commercial use, land use protection areas that provide EG&S could be integrated into the development plan.
- Greater responsibility of property owners and developers to do the planning and verifying of land development.
- A voluntary program that provides technical and financial assistance to landowners and others to develop the upland, wetland, riparian and aquatic areas on their property.

Other initiatives identified through the review were:

- Increased research identifying fiscal and non-fiscal values of ecological goods and services provided by grasslands, woodlands and forests.
- Implementation of shelterbelts for climate change.
- The need for better communication and coordination between programs and organizations to maximize efficiency.
- Development of a long term strategy for conservation of lands with low runoff potential
- Permanent protection through the acquisition from willing sellers of free title or easement of native prairies, wetlands and other habitats (including land to be restored).

19.7 CONCLUSIONS AND RECOMMENDATIONS

Provincial initiatives to reduce flood risk through land use policies / regulations have focused on wetland preservation. The impacts of urbanization, deforestation and agricultural intensification are not as controversial as wetland drainage and are more subtle. Urbanization represents a

very small portion of the Assiniboine and Lake Manitoba Basins and is normally addressed on a local basis. Current practices associated with Forest Management Plans appear to have been reasonably successful in managing deforestation of large forests for timber harvesting. For small parcels of land, Manitoba Habitat Heritage Corporation (MHHC), Ducks Unlimited, and the Nature Conservancy have voluntary, stewardship-based programs to conserve habitat for wildlife. There are no government policies and/or regulations to control conversion of marginal grasslands to cultivation and encourage land use for EG&S on marginal lands.

Integrated Watershed Management appears at present to be the best approach to identifying priority areas to reduce the land's runoff potential and development action plans. As recommended in Section 18.0, there is a need for better linkages between Integrated Watershed Management Plans (IWMPs) and Land Use Planning to ensure the IWMP recommendations related to riparian and shoreline protection, flooding and surface water management are adequately considered and adopted into land use plans. CD partnerships provided a means to deal with preserving, expanding or enhancing areas with low runoff potential.

The water retention capacity of grasslands and woodlands must be considered as part of their overall ecological goods and services. The preservation of grasslands and woodlands is through voluntary programs by MHHC, Ducks Unlimited Canada, Nature Conservancy and CDs. The primary tool for long-term habitat conservation is the conservation agreement, whereas CDs work with landowners to implement beneficial management practices (BMPs) on farms to conserve and enhance ecological goods and services.

Manitoba's Surface Water Management Strategy does endorse the principles associated with managing land use for the purpose of reducing runoff, however, it focuses on regulatory approaches for wetlands. Other land use changes associated with cultivation of grasslands and deforestation should be emphasized.

The review of land use initiatives has identified the following areas that could assist with preserving, expanding or enhancing areas with low runoff potential:

- Retain Crown land as a provincial resource to improve drought resiliency, reduce flood risk, improve water quality and support healthy ecosystems.

- Use of caveats with Crown land sales to maintain the natural ecological goods and services of the land.
- CDs identify lands for removal and protection based on their contribution to the watershed in agriculturally developed regions, where habitat loss and disturbance has been most significant.
- Permits for the development (improvements) of EG&S lands.
- A policy with regulations to limit forest clearing and conversion along the base of Manitoba escarpment to protection the most sensitive landscape from erosion and flooding.
- Property tax credit on lands that are planted with trees or covered with natural wetlands, which contribute to stored carbon or water on the landscape.
- Tax landowners who have changed property resulting in a loss of EG&S. Those tax dollars would go back into securing, protecting and enhancing lands to mitigate for the loss.
- Financial assistance to agricultural landowners to implement beneficial management practices (BMPs) to conserve and enhance ecological goods and services (EG&S) on the landscape.
- Increased research identifying fiscal and non-fiscal values of ecological goods and services provided in grasslands, woodlands and forests.
- Additional programing to protect and restore EG&S of grasslands.
- The need for better coordination between programs and organizations to maximize efficiency.
- The development of a long term strategy for accelerating conservation of lands with low runoff potential.
- Enhancing programs such as Growing Forward 2: Growing Assurance that provides financial assistance to agriculture producers for adoption of targeted beneficial management practices (BMPs) that improve the environmental sustainability of their operations.

For these study recommendations to be to successfully implemented government(s) will require additional resources. As well, the policies recommended will need to be accompanied by regulations that will facilitate their implementation.

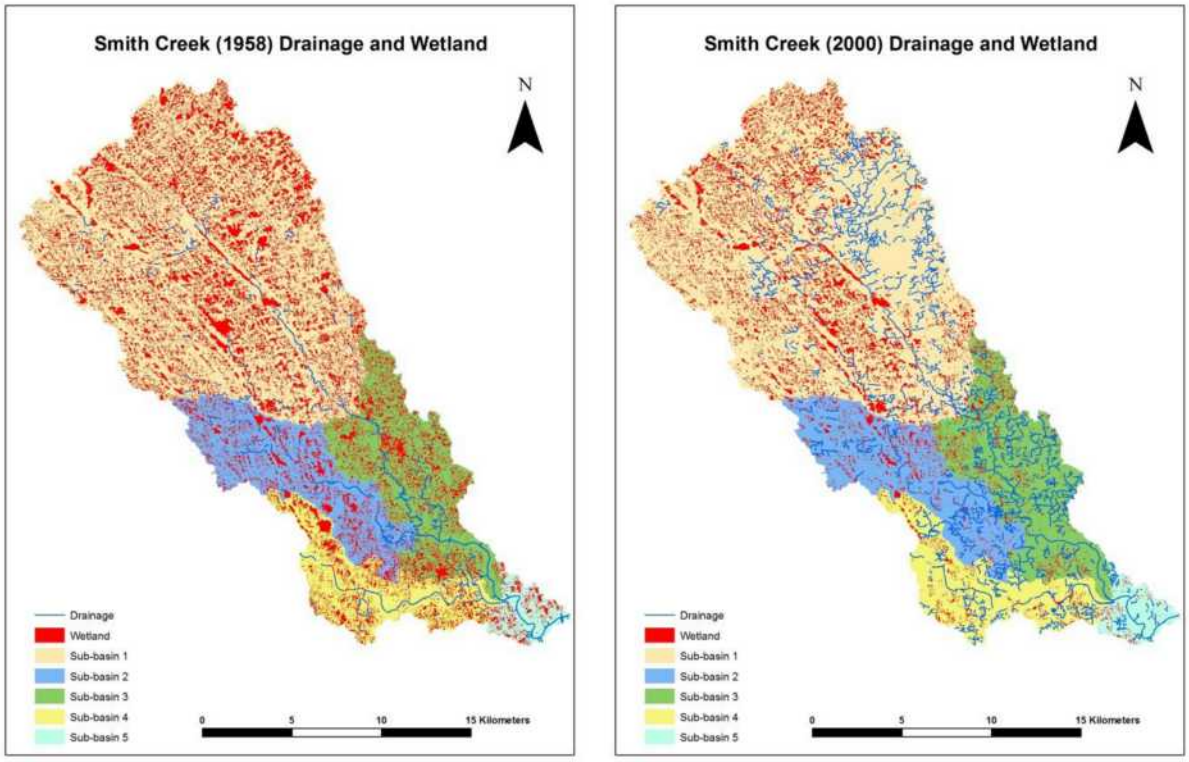
20.0 ROLE OF PRAIRIE WETLANDS IN FLOOD ATTENUATION

20.1 INTRODUCTION

20.1.1 Background

Recent flooding throughout southern Manitoba has raised questions regarding the value of wetlands for flood reduction. Although wetlands are extensive throughout western Manitoba, their numbers have decreased dramatically. In south-western Manitoba, Ducks Unlimited "estimates that we have lost over 100,000 ha (almost a quarter million acres) of wetlands over the last 40 to 60 years" (DUC, Website). The natural functions of wetlands can be altered or impaired by artificial manmade drainage. Although slow incremental changes in the natural landscape can lead to small changes in wetlands, the accumulation of these small changes can permanently alter the wetland function (Brinson, 1988). Although hydrologic conditions are paramount to the maintenance of a wetland, it is often more difficult to evaluate hydrology than it is to assess substrate or biota (National Research Council, 1995). Figure 20.1, which was taken from Pomeroy (2014) and was developed based on data provided by Ducks Unlimited Canada, depicts the realities of wetland drainage on the Smith Creek basin, a tributary of the Assiniboine River.

Figure 20.1
Changes in Wetlands and Drainage on Smith Creek Basin from 1958 to 2000



In 2011, many portions of the Assiniboine River Basin experienced record or near record flooding. There are diverse opinions on the effect that wetland drainage had on the 2011 Flood. There are those contending the drainage of marshes, sloughs and potholes in southern Manitoba and Saskatchewan increased the severity of the catastrophic 2011 Flood. They contend that what was already a significant flood was compounded by the elimination of wetlands from agricultural lands as their natural features and processes that could have helped buffer against such events had been lost. Wetlands hold and slowly release water, thereby reducing and delaying peak flows of water, which helps to decrease the impacts of severe events and extreme weather conditions. Others contend that wetland drainage did not have a significant impact on the flood, as the wetlands were already full or nearly full to capacity and, therefore, had little or no flood attenuation effect.

Wetlands are widely regarded as important components of a natural landscape, because of their value to local and regional biodiversity and particularly their functional hydrological role.

Wetlands function as natural sponges that trap and release surface water. It is the water storage function of wetlands that allows for flood attenuation. The specific effects of artificial drainage as a contributor to increased stream flow are not well understood but give rise to increased seasonal and annual water yields (Schottler et al., 2013). Given the extent of past wetland drainage and current intensification of subsurface drainage (Sugg, 2007; Blann et al., 2009), artificial drainage networks in total have the potential to alter water budgets and river flows on a watershed scale and must be quantified before management strategies can be fully developed.

As part of the Assiniboine River & Lake Manitoba Basins Flood Mitigation Study, it has been requested that the study assess and quantify the benefits of wetlands in reducing flood damages, and indicate how the benefits vary for different flood magnitudes. To quantify the benefits of wetlands, an understanding of the hydrologic functions of wetlands' flood storage and storm flow modification is required.

20.1.2 Study Objective

This document presents a comprehensive review of the research findings on the impact that agricultural drainage of wetlands has had on flood attenuating effects with specific reference to the surface drainage of North American prairie wetlands. The objective of the review is to understand the role of prairie wetlands' flood attenuating effects for different flood magnitudes, and the implications of the research on Assiniboine River floods.

Based on this literature review, two general topics surfaced: one being the impact of antecedent conditions on runoff response including the effect on the dynamic contributing drainage area; and the other the quantification of the flood attenuating effects of wetlands. Section 20.2 provides a summary of two previous studies on the topic. Section 20.3 discusses research on the impact that antecedent conditions have had on flooding. Section 20.4 contains a short synopsis of the earlier articles that contain information pertinent to the review. Section 20.5 provides a summary of John Pomeroy's (Pomeroy et al., 2014) current research on Smith Creek and provides a determination of what the findings could imply for Assiniboine River floods. Section 20.6 compares the 1922 and 2011 flood hydrographs for the Assiniboine River near Russell to shed some light on the effect of wetlands on low frequency flood events. Section 20.7 examines wetland preservation and the Province's proposed action on no net loss of wetland

benefits. Section 20.8 discusses issues associated with the research findings. Section 20.9 contains the study conclusions.

20.2 PREVIOUS STUDIES

Two earlier studies that examined the impact of wetlands on flood attenuation were reviewed: the Upper Assiniboine River Basin Study (Manitoba Conservation et al., 2000) and the An Taisce Study (2012). A short synopsis of the two studies is presented herein.

20.2.1 Upper Assiniboine River Basin Study

The Upper Assiniboine River Basin Study was undertaken by the governments of Saskatchewan, Manitoba and Canada because of concerns about the potential contribution of agricultural drainage to the 1995 Flood. The 1995 Flood was estimated as approximately a 1:100 year event immediately downstream of the Shellmouth Dam. Of particular interest was the drainage of wetlands associated with agricultural development and practices in the basin. The Drainage and Flood Control Committee (DFCC) was specifically charged with the task of determining the extent to which the basin landscape had changed because of wetland drainage, the clearing of bush, and the re-allocation of grasslands, as well as evaluating the impacts of these land use changes on the stream flow within the basin.

The DFCC found that the high flows in 1995 were attributed to a combination of meteorological events. Runoff from high precipitation in the fall of 1994 had nearly filled many of the natural wetlands; and soil moisture levels were high, reducing the natural storage available for spring snowmelt runoff. There was an above-normal snowpack followed by a mid-April rainstorm, which contributed additional runoff water and caused rapid melting of the snowpack. This rapid melt across much of the basin resulted in an unusual hydrologic condition, the nearly coincident peak flows on the Assiniboine and Whitesand rivers at Kamsack, as well as on smaller local tributaries between Kamsack and Shellmouth Dam. To compound the situation locally, high levels in the receiving streams reduced or prevented flow from many drainage ditches. It was concluded that “drainage works in the basin did not contribute significantly to the observed 1995 flood peaks and volumes. These can be attributed to natural conditions”.

The SLURP (**S**emi-distributed **L**and **U**se-based **R**unoff **P**rocesses) hydrologic model was successfully applied in modelling flows for larger watersheds of the Upper Assiniboine River Basin with 1995 to 1997 land cover. However, it was not possible to determine the effect on basin hydrology of changes to land use using the SLURP model. It was felt that at least two important interrelated prairie hydrologic processes were missing from the model: 1) the flow of surface runoff and interflow from one land cover type to another prior to reaching the stream network; and 2) the variation of surface area contributing to stream flow in any given year due to storage of runoff in wetlands and surface depressions. The basin has innumerable small wetlands, each with an individual small contributing upland. In many years, the evaporation from, and storage in, the wetland is sufficient to prevent any contribution to stream flow out of the wetland reaching the stream network. However, in some years such as 1995, a combination of high wetland storage carry-over, as well as winter and early spring precipitation result in many wetlands all contributing to the spring snowmelt runoff peak.

The conclusions reached regarding the hydrologic impacts of agricultural wetland drainage were:

- Climatic processes dominate the generation of major flood events.
- The greatest relative impact of the drainage of wetlands is during moderate runoff events, with the effect diminishing for increasingly larger flood events.
- Drainage of wetlands has little effect on peak flows due to major flood events.
- Effects of land cover changes and wetland drainage in the basin could not be quantified, but a trend of increasing runoff volume was detected.
- Increased runoff volumes into lakes caused level management issues and reduced the operational flexibility of reservoirs.

A limitation of the study was the inability to hydrologically model changes in wetland drainage because of the effects of dynamic contributing areas. Thus, there were limitations in its capacity to assess the role of wetlands in flood attenuation.

20.2.2 An Taisce Study

An Taisce National Trust of Ireland (2012) commissioned the Aquatic Services Unit of University College Cork to conduct a review of the role of wetlands in flood attenuation in Ireland. The

review investigated literature in Ireland and abroad to examine what is known about the effectiveness of wetlands in a flood attenuation role.

The findings of the report were that wetlands provide a potential ecosystem service for flood attenuation; but their role is also often exaggerated, as many wetlands only play a relatively small role, if at all, in attenuating floods. This is due to the diverse nature of wetlands in terms of location within a catchment, hydrological function and physical dimension. The conclusions reached relevant to the hydrologic impacts of agricultural wetland drainage were:

- Wetlands represent a heterogeneous group of habitats, each with unique hydrological processes. Attenuation potential is affected by many geographic, climatic, seasonal and land management variables, but is primarily a function of wetland water storage capacity at any specific time (i.e., when a wetland is saturated its water retention capability is diminished, or nil).
- The review led to making a distinction between “hydrological” floods (high frequency, low to medium rainfall events that occur commonly without economic damage) and “economic” floods (low frequency events following high intensity rainfall, potentially causing economic damage). Wetlands may readily attenuate “‘hydrological” floods but may fail to attenuate “economic” floods. The literature does not distinguish between these events, and tends to group “floods” into a single category.
- In general, the influence of wetlands in reducing flood peaks is greatest for high frequency, low to medium rainfall events that occur when wetlands have a large capacity for storage. It is least for large events when soil and wetland storage are saturated in advance.
- Most evidence of flood plain attenuation effects is derived from modelling rather than empirical studies. Factors influencing flood attenuation properties of floodplains include: surface water levels, contribution of hill-slope flow, soil moisture deficit, topography (particularly surface depressions), antecedent rainfall conditions, surface vegetation characteristics, drainage characteristics and the degree of connectivity between channel and floodplain.

20.2.3 Terminology

One of the difficulties in comparing the literature is the terminology used to describe the magnitude of the flood event.

For the purpose of this study the “hydrological” floods of the An Taisce Study (2012) and the “moderate” floods of the Upper Assiniboine River Basin Study (UARB), Appendix A (Manitoba Conservation et al., 2000).are considered as high frequency flood events (Manitoba

Conservation et al., 2000). The “economic” floods of the An Taisce Study (2012) and the “major” flood event of the UARBS are considered as low frequency flood events.

In the context of this literature review, hydrologic floods are considered events that occur frequently without significant economic damage or hardship; whereas, economic floods are low frequency events that cause significant economic damage and hardship similar to the 2011 Assiniboine River and Lake Manitoba floods.

The difficulty in using the term “economic flood” is that flood damages are impacted by the level of flood preparedness. What was an economic flood (i.e., the 1997 Flood in the Red River Valley) that caused hundreds of millions of dollars in damages may now be considered a hydrologic flood as a result of the flood proofing program undertaken in the valley since the 1997 Flood. Another example is the 2011 flood damage around Lake Manitoba. Many structures around the lake are being flood protected to withstand the 2011 Flood. For this report, floods are referred to as high (less than or equal to a 10-year event), moderate (between a 10-year and 100-year event) and low (equal to or greater than a 100-year event) frequency events.

20.3 IMPACT OF ANTECEDENT CONDITIONS

The available storage in a wetland can have a significant impact on its potential flood reduction capability. Generally, when the storage capacity is high following dry periods, the wetland can be effective in retarding runoff. When water tables are high and storage capacity is low, during a wet cycle, any additional water entering the wetland tends to run off the wetland. When the volume of suitable wetland storage is small compared with the volume of flood entering, the peak discharge remains unaffected. However, while an individual wetland may be full, it does not necessarily mean it is spilling or contributing to downstream runoff. Isolated prairie wetlands may connect to other isolated wetlands before contributing to downstream flows. These critical aspects of wetland water storage have major implications.

Relevant information concerning the impact of antecedent conditions on the flood response of wetlands is summarized as follows:

- Shaw et al. (2012a) looked at water level data for the St. Denis National Wildlife Area, Saskatchewan, to examine the connectivity between wetlands and contributing drainage

area for runoff. St. Denis is a 3.6 km² (1.4 mi²) undrained area with more than 100 wetlands. It was observed that in dry years the downstream pond only received local runoff; however, after successive wet years when upstream storage had been satisfied, the runoff volume increased by a factor of 20. The study concluded that basin conditions such as antecedent surface storage affect the connectivity between wetlands, which in turn determines the percentage of the basin that can contribute to the outlet. The study showed that after successive wet years much of the surface storage would be filled. Such antecedent basin storage conditions may result in high frequency precipitation events producing water volumes similar to low frequency, high volume runoff events.

- Shook and Pomeroy (2011) simulated the behaviour of prairie basins dominated by wetlands by applying precipitation and allowing it to runoff. Only direct evaporation from the water surface was computed. Knowing the depth of water uniformly applied and the volume running off and remaining, the fraction of the basin contributing flow was determined. The modelling indicated the existence of hysteretic relationships between the fractional water-covered area, the fractional contributing area of a basin and the fractional water storage. Hysteresis means that there is a non-linear relationship (and temporal delay) between input precipitation and catchment runoff or flow. The nonlinear input-output relationship is driven by varying storage conditions, which are themselves determined by the interplay between antecedent moisture / storage conditions and evaporation dynamics. It is explained by the fact that evaporation is taken from the water surface only while runoff is delivered from the much larger drainage area of each wetland. This memory explains the sudden changes in contributing drainage during low frequency events. The existence of the hysteresis indicates that a hydrologic model using a single transfer function for basins affected by wetland storage does not accurately estimate the dynamic contributing drainage area. Current research (Pomeroy, et al., 2014) aims at characterizing pothole wetlands in a binary manner, with two (or more) transfer functions embedded in a single hydrologic model. However, it requires high resolution LiDAR data for the basin to develop the functions.

- Shaw et al. (2012b) developed and tested conceptual curves for the hypothesized relationship between decreasing potential storage and the contributing area. The curves reflected the fill and spill process of wetlands. The conceptual curves for the relationship between pre-threshold storage and the contributing area illustrated the complexity of runoff response from a prairie pothole region. The curves indicated that, for the test basin, once storage filled above approximately 85%, the contributing drainage increases dramatically. These conceptual curves have the potential for use in a modelling framework to dictate the use of specific, time-variable transfer functions where LiDAR data is not available.

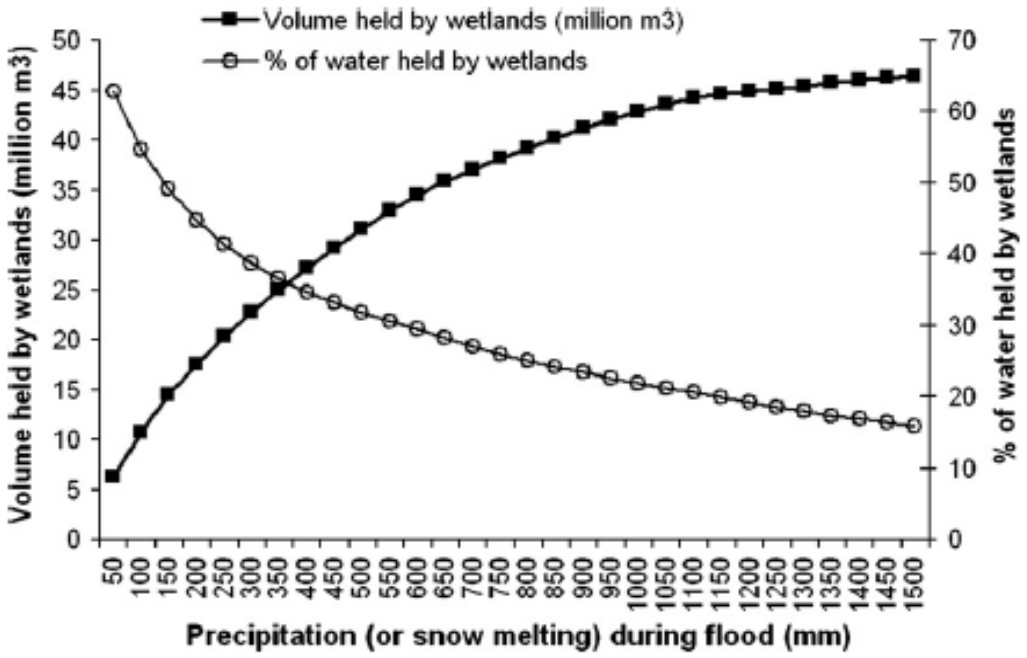
- Taylor and Pierson (1985) evaluated the effect of a small wetland's runoff response during the 1980 spring runoff season on a small catchment in south central Ontario. The majority of the runoff occurred during the spring when the water-table was high enough to produce extensive saturated surfaces. The wetland was found to have a large influence on runoff production in the upper portion of the catchment. The authors concluded that "An examination of individual storm hydrographs shows that the wetland can have a strong regulatory effect on quick flow yields, peak runoff rates and lag times, but the magnitudes of these effects vary with antecedent conditions and storm magnitude. When wetland water levels were elevated and had little storage capacity, it

had minimal effect on the runoff hydrograph; but the effect increased greatly when the system drained and more storage became available in the wetland".

- Huang et al. (2011) used high-resolution LiDAR to illustrate the connectivity between prairie wetlands within a 196 km^2 (76 mi^2) area of North Dakota and the need to simultaneously consider the actual floodwater storage and the potential catchment contribution when considering the flood reduction potential of wetlands. While wetlands are often viewed as topographically isolated, many may connect to one another during wet conditions through the ''fill and spill'' mechanism. Isolated wetlands commonly are integral parts of extensive groundwater flow systems, and isolated wetlands can spill over their surface divides into adjacent surface-water bodies during periods of abundant precipitation and high water levels (Winter and LaBaugh, 2003; van der Kamp and Hayashi, 2009). In the study, a total of 2,245 unique wetland basins were identified with a total potential flood storage estimated at 52,440,000 m^3 (1,800,000,000 cu ft). When basin connectivity was considered, 413 catchments were merged and the total volume increased to 80,240,000 m^3 (2,800,000,000 cu ft). Using this information, the Basin Floodwater Regulation Index (BFRI) shown on Figure 20.2 was developed. The BFRI demonstrated the effect of wetland storage for flood events of different severities. When the model was applied after accounting for May 17, 2008 water level conditions, which reduced the storage capacity, it was observed that, "When this filled capacity is excluded, we can see that 48% of the area on the west side and 5% of the area on the east side has BFRI scores greater than 200 mm (7.9 in); this implies that these lands would still prevent runoff from contributing to stream flow during a 200 mm (7.9 in) equivalence snowmelt or storm event. With the existing water detected on May 17, 2008 excluded, the water that could be held by the wetland basins increased from 6,000,000 to 46,000,000 m^3 (211,900,000 to 1,600,000,000 cu ft) when the precipitation increased from 50 to 1500 mm (2 to 60 in). However, when the precipitation is above 1000 mm (39 in), the water volume held by wetland basins could only increase very slightly. Furthermore, while the absolute water volume held by the basins increased with precipitation, the relative percentage of the water held correspondingly decreased from 63% to 15% when the precipitation increased from 50 to 1500 mm (2 to 60 in)". This showed that under extremely wet conditions (i.e., 60% full) not a lot of further accumulative precipitation would be required to fill and spill the wetlands.

The Huang study illustrates the need to consider wetland connectivity when modelling the hydrologic effects of wetland storage. Modelling to date, except for an initial attempt in the Prairie Hydrologic Model (PHM) (Pomeroy et al., 2010), has considered wetlands as separate water bodies. These earlier studies appear to have underestimated wetland storage and its effects as wetland storage connectivity was not considered. Recently, Ducks Unlimited Canada (DUC) funded research to improve and test the PHM to better consider wetland connectivity on the Smith Creek Research Basin. This is discussed in Section 20.5 of this report.

Figure 20.2
Basin Floodwater Regulation Index



Antecedent conditions were a key factor in contributing to the 2011 Flood in the Assiniboine River & Lake Manitoba Basins. Falling within a somewhat wet cycle, 2011 was preceded by a very wet 2010 fall when fall precipitation averaged about 175% of normal over a wide area in Manitoba and Saskatchewan. This produced extremely high regional antecedent moisture conditions whereby moisture levels were 100% to 250% of normal (MFRTF, 2013a). With these extremely wet antecedent conditions, it was likely much of the depressional storage in the Assiniboine River & Lake Manitoba Basins would have been near capacity. This is illustrated by the high water levels on Whitewater and Shoal lakes.

Research indicates that antecedent surface storage affects the connectivity between wetlands. This in turn determines the percentage of the basin that can contribute to the outlet, which has a significant impact on the flood response of a basin. The existence of surface water storage memory explains the sudden changes in contributing drainage during low frequency events. The existence of this hysteresis memory indicates that a hydrologic model using a single transfer function for basins affected by wetland storage may not accurately estimate the dynamic contributing drainage area. There is a need to consider wetland connectivity when modelling the hydrologic effects of wetland storage. Modelling to date, except for an initial attempt by the

Prairie Hydrologic Model (PHM) (Pomeroy et al., 2010), has not considered wetlands connectivity. These earlier studies appear to have underestimated wetland storage and its effects as wetland storage connectivity was not considered.

20.4 WETLAND FLOOD ATTENUATION RESEARCH

The effectiveness of flood attenuation by a wetland depends largely on the amount of rainfall, the flood event, size of the area, vegetation, slope, soil saturation and storage before flooding occurs and location of the wetland in the flood path (US EPA, 2006). The exact hydrological function of a wetland differs based on the frequency and magnitude of high water, duration, timing, and temporal sequences of high and low water and location in the watershed. To preserve or restore wetlands, there must be an understanding of the value of wetlands on flood flow modification. A number of studies have attempted to quantify the value of wetlands on flood flow attenuation. Most evidence of the attenuation effects of wetlands is derived through hydrologic modelling and empirical studies. A literature review identified a number of case studies that quantified the flood attenuating effects of wetlands. Relevant studies are summarized below and grouped into categories based on their conclusions.

1. Volumes

- A study (Doeing and Forman, 2001) assessed the impact of restoring drained depressions on the inflows to Devils Lake. Different levels of restoration (25, 50, 75, and 100%) of a total possible 15,768 ha-m (127,835 ac-ft) of storage were analyzed. The study indicated that the annual flow reductions resulting from depression restoration varied significantly for individual water years. In dry years, the percentage of flow reduction is larger than in wet years. The average annual runoff reduction is less than the restored volume. The average annual volume captured was consistently about 25% of the restored volume across the different restoration scenarios. Four restoration scenarios were considered: 25%, 50%, 75%, and 100% of the 156,400 dam^3 (127,835 ac-ft) of potential storage. The study indicates that depression restoration will reduce the volume of runoff entering a lake or reservoir, which will impact its water management.

- Through a case study of the Rat River watershed in southeastern Manitoba, the role that wetlands played in reducing the runoff volume in high and low frequency floods events was investigated (Juliano and Simonovic, 1999). The 1,550 km^2 (599 mi^2) watershed was composed of 3% wetland area. Three scenarios, with increased wetland areas of 5%, 8% and 13%, were considered. The results of the analysis shown in Table 20.1 indicated that wetlands could play a role in reducing the total volume for low frequency flooding events such as in the 1997 Flood (about a 100-

year event). They played a slightly larger role in reducing total volume for higher frequency flooding events, such as in 1996.

Table 20.1
Results of Rat River Wetland Drainage Scenarios

Scenario	Percent Increase in Wetland Area above 3%	Total Wetland Area (km^2)	Total Volume Reduction 1996 (%)	Total Volume Reduction 1997 (%)
1	2	46.04 (11,400 ac)	3.7	2.2
2	5	73.66 (18,200 ac)	9.3	5.6
3	10	119.70 (29,600 ac)	18.6	11.1

- A study was undertaken to derive the maximum storage capacity of natural wetland depressions in the Devils Lake Basin of North Dakota and to estimate the actual volume of water in the wetlands for selected frequency of runoff events (Ludden et al., 1983). The study determined that the depressions stored about 72% of the total runoff volume from a 2-year frequency runoff and about 41% of the total runoff volume from a 100-year frequency runoff. The study showed that depressions retained a greater percentage of the water from a small runoff than from a large runoff event. A limitation of the study was that it had not considered antecedent moisture in storage.

2. Peak Flows

- Hydrologic model studies in the upper Mississippi River Basin were undertaken to assess the potential flood flow reduction of wetlands (USACE, 1994). For the analysis, average soil moisture conditions were assumed. However, for the 1993 Mississippi River Flood, near saturated antecedent soil moisture conditions were experienced. Therefore, the modelled peak discharge reductions were considered to be greater than what would occur for an event like the 1993 Mississippi River Flood.

  In the watersheds modelled by the USACE, the maximum reduction for floodplain wetlands was 6% of the 25- and 100-year storm events. Wetlands were found to be more effective in upland areas with deeply incised potholes (Redwood River watershed) where reductions were 23% of the 1-year event, 11% of the 25-year event, and 10% of the 100-year event. In areas of shallow depressions (Boone River watershed), restored wetlands reduced peak discharge by 9% of the 1-year event, 7% of the 25-year event, and 5% of the 100-year event.

  The USACE concluded that “Wetland restorations in the uplands could function much the same as small upland reservoirs. It was shown more than three decades ago that small flood damage reduction dams are effective in the reach of stream immediately downstream but their effect diminishes rapidly with distance. As far as a series of small headwater dams is concerned, they are essentially ineffective under conditions in which major floods occur on large rivers”.

- A study (Bengtson and Padmanabhan, 1999) quantified the effectiveness of wetlands on peak flow rates in two watersheds in the U.S. portion of the Red River Basin: the Maple River in North Dakota, and the Wild Rice River in Minnesota. Each watershed has a drainage area of approximately 4,144 km² (1,600 mi²) with about 0.25% of the Maple River and 1.3% in the Wild Rice River watershed by area with drained wetlands. The average bounce or available wetland water storage of 0.3 m (1.1 ft) of modelling showed that the maximum reduction in flood stage was about 3% for a 10-year storm. For a 100-year event, the reduction was less than 0.4%. For a four foot bounce, the flood stage was reduced by 10% for the 10-year event, but only 1.26% for the 100-year event. For the 100-year event, the reduction in flow volume was 2.7%.

  To investigate the effect of much greater wetland storage, the model was run with 1%, 2.5% and 5% restored wetlands by area. The best results were for the higher-frequency events. With 1% restored wetlands by area, the flood stage was reduced by nearly 19%. With 5% wetlands, the flood stage was reduced by 34%. For a low-frequency event such as the 100-year storm, storage from 1% restored wetlands reduced the flood stage by less than 0.9%. For 5% restored wetlands, the flood stage was reduced by 8.4%. This case represents nearly 20 times the existing drained wetlands by area.

  The authors concluded that “Based on the results of this study, restoring drained wetlands could not have significantly affected the flood stage for a flood in the order of the 1997 Red River Valley Flood in the Maple River watershed. Even increasing the additional wetland storage represented by restoring drained wetlands by four times did not result in a significant reduction in the flood stage. The findings from the Miller study (1999) support these results that for flows in the order of magnitude of a 100-year event, even 40% wetlands by area did not have a significant effect. Restoring drained wetlands is highly unlikely to significantly affect the low-frequency flood events such as the Red River Valley flood of 1997”.

- The effects of agricultural land use and riparian changes on stream flow from the Little Cobb River, a tributary to the LeSueur River in the Minnesota River Basin were examined (Miller, 1999). The Little Cobb River is a 335.4 km2 (129.5 mi2) tributary watershed of the LeSueur River. The modelling found that the drainage of wetlands increased the magnitude of peak flows up to a 50-year recurrence interval. Historical drainage of wetlands increased annual peak flood discharge by up to 57% during high-frequency flood events but had a negligible effect on reducing the magnitude of large (low-frequency) flood events.

- The hydrological equivalent wetland (HEW) concept in the Soil and Water Assessment Tool (SWAT) was used to assess the effects of wetland restoration in the Otter Tail River watershed located in northwestern Minnesota. (Wang et al., 2010). The study area was the 4,506 km^2 (1,740 mi²) upper portion of the Otter Tail River watershed. The 1,513 wetlands covered 27% (2,378.2 ha) (5,876.6 ac) of the study area. The effects of wetland restoration were measured as reductions of flood runoff. The assessment was based on the period December 1969 to May 31, 1975. The period experienced high to moderate flood frequency events.

  The study concluded that, “the loss of the first 10 to 20% of the wetlands in the study area within the Otter Tail River watershed would drastically increase the peak

discharge (by 40%). On the other hand, the restoration of 460 to 550 ha (1,137 to 1,359 ac) wetlands in the Otter Tail River watershed would reduce the peak discharge by 20%." The impacts for various flood magnitudes are not discussed. The results were not transferred downstream to the confluence with the Red River.

3. Peak Flows and Volumes

- The role that wetlands play in low frequency floods was investigated through a case study using the Rat River watershed in southeastern Manitoba (Simonovic and Juliano, 2001). The 1,550 km^2 (599 mi^2) watershed is composed of 3% wetland area. Three scenarios, with increased wetland areas of 5%, 8% and 13%, were considered. The results based on the 1997 flood were:

  - For the 2% increase in wetland area (46.04 km^2 (17.78 mi^2)), spring flow started on April 19, rather than April 18. The peak of the hydrograph remained at approximately 752 m^3/s (26,557 cfs) on April 23 with a decreased runoff volume of 2.2%.

  - For the 5% increase in wetland area (73.66 km^2) (28.44 mi^2), spring flow started on April 20, rather than April 18. The peak of the hydrograph remained at approximately 752 m^3/s (26,557 cfs) on April 23, with a decreased runoff volume of 5.6%.

  - For the 10% increase in wetland area (119.704 km^2) (46.22 mi^2), spring flow started on April 21, rather than April 18. The peak of the hydrograph remained at approximately 752 m^3/s (26,557 cfs) on April 23, with a decreased runoff volume of 11.1%.

  Based on the results of this case study, it was found that a reduction in total flood volume could be accomplished with an increase in wetland area. The results showed that given a low frequency flood event, a reduction in the flood peak was non-existent.

- The goal of a multiphase research project on Broughton's Creek in southwestern Manitoba was to determine the impacts of wetland loss (between 1968 and 2005) and associated drainage activity and then focus on developing a hydrologic model to evaluate the environmental impacts of this loss at a watershed scale (Yang et al., 2010). The project confirmed that 5,921 wetland basins, or 70% of the total number of wetlands in the watershed, have been degraded or totally lost due to drainage activity. This loss resulted in 21% of the wetland area being lost. The developed modelling system showed a 37% increase in peak flow and a 62% increase in runoff as the result of a 31% increase in area draining downstream due to drainage. The modelling supports wetland drainage having an impact on high to moderate frequency flood events. Ongoing monitoring in the Broughton's Creek watershed paired with a detailed inventory of intact and drained wetlands indicates that, even under the extreme moisture conditions of 2011, drained wetlands contributed significantly to increased flow volumes out of the watershed.

- Through a series of stream flow simulations, the sensitivity of the spring runoff to the changes in land use and drainage was examined for Smith Creek (Pomeroy et al., 2010). The approximately 435 km^2 (168 mi^2) watershed located in eastern Saskatchewan within the Upper Assiniboine River Basin has undergone rapid

changes over the past 50 years, transitioning from areas of extensive woodlands and wetlands to grain and oilseed cultivation. There was a reduction in wetland areas from 17% to 9% during the period 1958 to 2001, as well as changes to the drainage network. The simulation results found that the effects of land use and drainage on spring runoff volumes were highly variable from year to year. The basin spring runoff volume appeared to be most sensitive to changes in wetland storage. The simulations indicated the long-term average spring discharge volume had been increased by 1%, 19%, and 36% through forest conversion, agricultural conversion and wetland losses respectively. For high frequency events, the peak daily discharge tended to increase in the range of approximately 40% with wetland drainage. For moderate floods, the peak actually decreased under the drainage scenario. This was attributed to the fact that the length of the channel increased; and consequently, it took a longer time for the flow to travel to the basin outlet and for the hydrograph to become more attenuated. Under the wetland drainage scenario, the 1995 peak (a low frequency event) was increased by approximately 10%. However, it was noted in the study that the complex response of peak discharge to drainage and restoration required that substantial caution be exercised in interpreting the results.

4. Location in Watershed

- It was reported that a strong inverse correlation existed between the size of flood peaks and basin storage (percentage of basin area occupied by lakes and wetlands) in many drainage basins throughout the United States (Carter V., undated). Basins with 30% or more areal coverage by lakes and wetlands have flood peaks that are 60 to 80% lower than peaks in basins with no lake or wetland area. Downstream wetlands are more effective at reducing downstream flooding than are headwater wetlands, largely as a result of larger storage capacities.

- The causes of flooding and the estimated contribution to flooding from historic land use changes and the extension of the drainage network since the mid-1960s in the Fisher River Watershed in the Manitoba Interlake were examined (AECOM, 2009). Since the mid-1960s the upper portion of watershed has experienced an extensive expansion of the drainage network in the upper reaches with a reduction of water bodies from 16.6% to 14.0% over the period from 1966 to 2002. The HEC–HMS hydrologic and MIKE II hydrodynamic models were developed for the watershed. The hydrologic model generated runoff hydrographs from 148 catchments reflecting land use and drainage conditions existing in 1966 and 2002. The loss in wetland area was accounted for and the aggregate curve number (CN) computed for each catchment. Inflow hydrographs representing upstream and lateral inflows were routed through a network of streams that existed in 1960s land use. The Mike II model was used to physically route the flows through the complex system of main channel, tributaries, branches and drainage works. The hydrodynamic model included natural constrictions and man-made obstructions such as bridges, culverts and low level crossings. The pre-1966 model treated the recently developed drains as unimproved shallow swales with increased roughness consistent with floodplains.

  The modelling indicated the following impacts at the downstream portion of the watershed due to land use changes since the 1960s:

- For the 2006 flood (about a 1:5-year event), a 5% increase in volume, a 2.9% increase in mean daily peak flow, and a 1.4% increase in instantaneous peak flow. The increase in peak stage was 0.02 m (0.07 ft).
- For the 100-year flood, a 4% increase in volume, a 5.5% increase in mean daily peak flow, and a 4.4% increase in instantaneous peak flow. The increase in peak stage was 0.007m (0.023 ft).
- The timing of the peak flow remained unaffected.

The impacts were much more pronounced in the upper and middle reaches of the Fisher River. Peak flows were found to increase in the range of 5% to 12%.

The study found that the greatest effect of wetland drainage on flood peaks was localized. Hydrodynamic modelling determined that, as the flood peaks were transmitted downstream, they decreased because of the attenuating impact of the throttling effect of culverts, roads and floodplain storage and the timing of the upstream runoff, which was later than the downstream local runoff. To verify this, an extreme network analysis was considered with the total removal of newer drains through stream truncation in the model. The elimination of the contributing runoff from the areas serviced by these newer drains resulted in no change in the peak flow rate of the 2006 or 100-year flood events at the watershed outlet but reduced the total volume of flow. The study illustrated that the effects of wetland drainage on peak flow rates lessened as they were transmitted downstream. Also any potential limitations on the dynamic potential of surface storage and the effect this storage had on contributing area in the watershed was irrelevant due to the location of the *drainage* in the upper reaches of the watershed.

5. General / Other

- A watershed-scale methodology was developed for identifying potential sites for wetlands in a tile-drained landscape and for optimizing the spatial distribution of these wetlands for reductions in peak flows (Babbar-Sebensa et al., 2013). The study area was the Eagle Creek Watershed (ECW) located 16 km (10 mi) northwest of Indianapolis, Indiana. Results showed that, with fewer sites and smaller wetlands covering only 1.5% of the watershed, runoff from 29% (almost a third) of the watershed area could be captured. Peak flow reduction could be obtained ranging from 6.5% to 14.6% for saturated hydraulic conductivities of 0.5 mm/h to 50 mm/h, respectively.
- A comparison was made of changes in precipitation, crop conversion and the extent of drained depressional areas in 21 Minnesota watersheds over the past 70 years (Schottler et al., 2013). The comparison was based on the two hydrologic periods: 1940 to 1974, and 1975 to 2009. More than half of the watersheds had increases in water yields in the later period. The most significant increase was observed in the spring (May to June) with a much smaller increase in the fall (September to October). The seasonal and annual water yields were found to have increased by greater than 50% since the 1940s on half of the watersheds, with no statistical change in the others. Artificial drainage was identified as the largest driver of the increase in flow volumes, exceeding the effects of precipitation and crop conversions. The majority of the increase in flow was attributed to changes in water residence time on the

landscape and subsequent reductions in evapotranspiration from the drainage of wetlands.

- Gleason et al. (2007) developed and applied a simple model for estimating / simulating the water storage capacity of potentially restorable and existing wetlands under various restoration, land use, and climatic scenarios. The model was developed and applied in the 527.8 km^2 (204 mi^2) upper Mustinka sub-basin within Grant County, Minnesota to simulate wetland water storage increases that would result from restoring 25 and 50% of the farmed and drained wetlands. Existing wetlands had a surface area of 2,397 ha (5,923 ac) and intercept 20.6% of the sub-basin while restorable wetlands had a surface area of 2,819 ha (6,966 ac) and intercept 21.7% of the sub-basin.

  The model simulations were performed during the growing season (May–October) for relatively wet (1993; 0.67 m (2.2 ft) of precipitation) and dry (1987; 0.32 m (1 ft) of precipitation) years. Results from the simulations indicated that the 25% restoration scenario would increase water storage by 27 to 32% and that a 50% scenario would increase storage by 53 to 63%. It was estimated that wetlands in the sub-basin had the potential to store 11.57 to 20.98% of the total precipitation that fell over the entire sub-basin area during the study period.

- A study was conducted to simulate stream flow and wetland storage within a part of the Starkweather Coulee sub-basin, which is a major sub-basin of the Devils Lake Basin. It has a drainage area of 803 km^2 (310 mi^2) of which 210 km^2 (81 mi^2) contributes stream flow (Vining, 2002). Within the study area, the average wetland depth was about 0.67 m (2.21 ft). The total maximum wetland area was about 12,500 ha (30,890 ac) at the overflow elevation, and the total maximum wetland volume was about 84,210 dam^3 (68,270 ac-ft). The extent to which water could be retained in the Devils Lake Basin was revealed when greater-than-average precipitation in the 1990s filled many of the wetlands in the basin.

  Model runs were made for water years 1981 to 1998 to calibrate the model to the observed stream flows obtained from the Starkweather Coulee Gauging Station The modelling indicated that during April 1997, the simulated daily water volume in the open and closed wetlands combined, attained a maximum of about 143,000 dam^3 (16,000 ac-ft). It was concluded from the modelling that the greatly increased size of the closed wetlands during water years 1994 to 1998 probably allowed for increased water storage and decreased simulated stream flow.

- A study was conducted to simulate runoff and wetland storage in the Hamden and Lonetree watersheds in the Red River of the North Basin (Vining, 2004). These sites were selected because they contained a variety of wetland types. The impact of wetland storage was only evaluated for the Hamden watershed as Lonetree watershed was considered to have too small of a drainage area and wetland capacity. The Hamden watershed site is in west-central Minnesota and encompasses about 26 km^2 (10 mi^2) just south of the Buffalo River near Callaway, Minnesota.

  For the Hamden watershed, a hydrologic model was calibrated for the runoff period 1999 to 2002. To simulate flood conditions, the January through May 1997 daily temperature and precipitation data were substituted for the January through May

2000 data set. The substituted data were used to simulate the occurrence of the 1997 Flood in the Red River Basin followed by the occurrence of the June 2000 rainfall event. The simulated runoff during flood conditions in April (1997 dataset) and June 2000 and March and April 2001 was reduced by 1% to 6% for an increased spillage threshold from wetland storage. Reductions for most other spring, summer, and fall months ranged from 18% to 80%. Total runoff volumes for the period of record were reduced by approximately 31% for the increased spillage threshold. Simulation results indicate total stream flow from a flood event could be reduced by wetland storage. However, the flood peak during a flood event might not be affected substantially.

- The amount of water in small prairie pothole wetlands shortly after spring thaw on several tracts of land in northeastern South Dakota was documented (Ludden and Linder, 1986). The volume of water in 213 small wetlands on 648 ha (1,601 ac) of the Altamont Moraine in northeastern South Dakota was measured in April 1982, immediately after the vernal thaw. The 213 wetlands comprised 50% of the water surface area that occurred in the study area and contained an estimated 19.58 ha-m (158.7 ac-ft) of water.

  Visits to the sites prior to the spring thaw revealed that many of the wetlands had little or no ponded water beneath the snow cover. The authors commented that, if these wetlands had been artificially drained, the volume of spring water would have been located elsewhere in the watershed; and under the right conditions, this water would have contributed to flooding at lower elevations in the watershed.

Table 20.2 summarizes the findings from the various hydrologic studies that quantified the impacts of wetland drainage. As indicated by Table 20.2, a wetland's ability to attenuate peak flows appears to be highest for high frequency, low to medium precipitation events that occur when there is a large capacity for water storage. Flood attenuation ability is lowest for low frequency events, particularly following a long period of high precipitation where the capacity for water storage is low. Also, wetland drainage results in increased seasonal and annual water yields. However, very few studies differentiate the increase in runoff between high and low frequency flood events. It would appear that the percentage increase in runoff volume decreases with the severity of the flood event.

A number of studies from the Red River Basin (Juliano and Simonovic, 1999; Simonovic and Juliano, 2001) appear less applicable to the Assiniboine River Basin. The Red River Basin has a less spatial aggregation of wetlands than the Assiniboine River Basin, resulting in the ratio of effective contributing area to gross contributing area for the Red River Basin being higher than the Assiniboine River Basin. However, these studies appear to give a general indication of the impact of wetland drainage on peak flows and volumes for low flood frequency events at the mouth of the watershed. Generally, a reduction in total flood volume may be accomplished with

an increase in wetland area; however, the reduction in the flood peak event may be non-existent.

The research to quantify antecedent conditions indicates the importance of explicitly modelling wetland connectivity to accurately estimate the dynamic contributing drainage area of the prairie wetland landscape. Very few of the modelling studies described in this section did this. The results presented should thus be interpreted with caution. This limitation will be discussed further in Section 20.5.

Studies on flood attenuation focus a lot on flood peak and total flood volume reduction but less on flood peak timing (Simonovic and Juliano, 2001). Flood timing can be critical in determining the success of a flood mitigation strategy, and it is definitely an aspect that has been understudied in relation to wetland drainage. The location of the wetland drainage has a significant impact on the peak (AECOM, 2009). Also, the percentage of change in peak flow rate cannot be directly converted to a change in river stage, as at higher flow rates the river channel normally has more flow capacity. For assessing flood damages, the percentage change in flow rate cannot be applied directly. Nonetheless, an analysis of streamflow data for several hydrometric gauging stations was undertaken for this study and is discussed in Section 21.4.

Table 20.2
Summary of Quantified Wetland Drainage Impacts Studies

Watershed	Drainage Area (km^2)	Degree of Drainage	Impact on Peak Flow Rate			Impact on Runoff Volume	Impact on Flood Peak Timing
			High Frequency	Moderate Frequency	Low Frequency		
Devils Lake						Decrease equal to 25% of the restored volume	
Little Cobb River, MN	868.7 (215,000 ac)	10% to 40% internally drained	37%-57% increase		Negligible effect	n/a	
Maple River	4,144 (1,024,000 ac)	0.25% of watershed Drained	24% increase		6% increase		
Rat River Watershed	1,550 (383,000 ac)	Sensitivity analysis 2%, 5% and 10% increase in wetland area			Negligible effect	Decrease proportional to % increase in wetland area	No impact on date of peak
Broughton's Creek	251.9 (62,200 ac)	519 hectares or 70% of the wetlands degraded or lost	37% increase			62% increase	
Redwood River, MN	1813 (448,000 ac)	Restored wetlands	23%* decrease	11%* decrease	10%* decrease		
Boone River, IA	984 (243,000 ac)	Restored wetlands	9%* decrease	7%* decrease	5%* decrease		
Smith Creek	435 (107,000 ac)	Reduction in wetland from 17% to 9% (during 1958 to 2001)	30% to 50% increase	Negative	10%* increase	PP% increase in long-term average	
Fisher River	2,200 (545,000 ac)	Reduction in wetlands and water body areas 16.6% to 14.0% from 1992 to 2002		2.90% increase	5.50% increase	5% increase for moderate frequency flood event 4% for low frequency flood event	No impact on date of peak at watershed outlet

Watershed	Drainage Area (km^2)	Degree of Drainage	Impact on Peak Flow Rate			Impact on Runoff Volume	Impact on Flood Peak Timing
			High Frequency	Moderate Frequency	Low Frequency		
Mustinka Sub basin	527.8 (130,000 ac)	Restored wetlands				Potential to store 12% to 21% of the total precipitation	
Hamden watershed, MN	26 (6,400 ac)	Restore wetlands	18% to 80% decrease		Not affected	1% to 6% decrease during low frequency flood events 31% decrease in average flow	
Otter Tail River watershed, MN	4,506 (1,113,000 ac)	1513 wetlands covered 27% Loss of the first 10% to 20% of wetlands	40% increase				

Note: *Assumes average soil moisture conditions.

20.5 REVISED UNIVERSITY OF SASKATCEWAN PRAIRIE HYDROLOGIC MODEL

20.5.1 Study Summary

The most recent and advanced research on the topic of the impact of prairie wetland drainage on flood attenuation involves the University of Saskatchewan's redevelopment of the 2010 Prairie Hydrological Model (PHM) and its application to the Smith Creek watershed (Pomeroy et al., 2014). The work involved revising the 2010 Prairie Hydrological Model to include improved snowmelt and evaporation physics and a hysteretic relationship between wetland storage and runoff contributing area. The significant results of simulations of runoff changes on the effects of drainage on runoff volumes and peak flows based on the wetland drainage in the Smith Creek watershed during the period from 1958 to 2008 are presented. The year 2008 was assumed to represent the 'present condition' since it was the last year that detailed information on wetlands was available.

The most significant change to the model is that it now incorporates the concept of the dynamic contributing drainage area of the Prairies, which changes with the amount of water in depressional storage. The initial PHM (Pomeroy et al., 2010) represented all of the surface water storage within each sub-basin as a single sub-hydrologic response unit (HRU). Although this representation was an advancement on earlier models, it was not capable of accurately simulating the dynamic nature of depressional storage and contributing area within a prairie basin.

A hysteretic relationship between the volume of water stored in the sub-basin and the area of water and the contributing area fraction provides the bases of the algorithm implemented in the model to simulate the dynamic nature of depressional storage and contributing drainage area (Shook and Pomeroy, 2011). This hysteretic relationship is driven by varying storage conditions of wetlands in a sub-basin, which are themselves determined by the interplay between antecedent moisture / storage conditions and evaporation dynamics and runoff into the wetlands. It is explained by the general water budget of the wetlands in which evaporation (drying) from the wetland continuously lowers wetland storage. However, it may be interrupted by runoff events (wetting), which have the potential to generate large runoff volumes into a wetland significantly raising the water level. This volume is greater than the evaporative losses

(drying) because a wetland's contributing drainage is larger than its surface area. When low frequency flood events occur, the increase in storage may result in the wetland spilling and conductivity between wetlands. To simulate this concept, an algorithm based on a small set of depressions with a frequency distribution matching that of a basin was used to simulate the filling and spilling sequence of a wetland. Including this Algorithm improved the simulated annual water budget and the annual variability in runoff volume of Smith Creek over the original PHM.

The revised model was used to simulate stream flow runoff from Smith Creek, Saskatchewan for six years (2007-2013). The Smith Creek watershed has a drainage area of 393.4 km^2 (151.9 mi^2) and is relatively flat, with slopes fluctuating from 2% to 5% and elevations ranging from 490 m to 548 m (1608 ft to 1798 ft). The basin has been partially drained, with the extent of wetland area estimated to have decreased from 24% to 11% of the basin area from 1958 to 2008. Over the simulation period, Smith Creek experienced the highest annual flow volume on record (2011) and high flows from heavy summer rains in 2012. The model was run over this period for various wetland extent scenarios including the 1958 historical maximum, measured extents in 2000 and 2008, and an extreme minimum extent involving complete drainage of all wetlands in Smith Creek Basin.

The results of the computer simulation model indicates that the annual stream flow volume and peak daily discharge has a remarkably strong sensitivity to wetland drainage, both over the historical period (1958-2008) and to the minimum possible wetland extents. The model indicated wetland drainage has a strong impact on both snowmelt (2011) and rainfall (2012) floods. Overall, Smith Creek total flow volumes over six years increase 55% due to drainage of wetlands from the current (2008) state, and decrease 26% with restoration to the 1958 state. The increase was largest for years with low runoff, ranging from 200% to 300% increases. The increase in runoff volume was the largest for the flood of record (2011). For the 2011 Flood, the annual flow volume and the peak daily discharge were estimated to increase from 57,317 to 81,227 dam^3 (2,020,000,000 to 2,890,000,000 cu ft) (42%) and from 19.5 to 27.5 m^3/s (688.6 to 971.2 cfs) (41%), respectively, due to wetland drainage that had already occurred in Smith Creek.

The model simulation exercise indicated that wetland drainage can increase annual and peak daily flows substantially, and that notable increases to estimates of the annual volume and peak daily flow of the flood of record had derived from wetland drainage and will proceed with further wetland drainage. While the effects of wetland storage available for runoff reduction vary with antecedent moisture conditions, the Smith Creek results suggest that, for major floods preceded by the relatively empty natural wetlands of the 1958 era, wetland restoration to a 2008 condition could reduce runoff by about 30%. The reduction could be much less for flood events preceded by major floods in the antecedent year or two, which would have filled the wetlands existing in 1958, leaving little or no space to store additional water. The summer flood of 2012 is an example of such a condition. Wetland restoration to the 1958 condition produced virtually no reduction in runoff for 2012, since the extreme flood of 2011 would have filled the wetland existing in 1958, leaving no room to store water in 2012.

The authors indicate:

> *"It must be noted that these results are from hydrological model simulations, some of which are based on interpolated basin wetland conditions that cannot be verified with field observations. The lack of stream flow observations before wetland drainage commenced and the impossibility of verifying simulations for future drainage make the model results for scenarios impossible to verify independently. The hydrological model simulations only include the wetland drainage components of environmental change in Smith Creek Basin and do not include concomitant climate change and other land management change impacts on the basin over the last 50 years. As such, they should only be used for assessing the impacts of wetland drainage under conditions of constant climate and agricultural land use.*
>
> *Smith Creek Basin had a very large percentage of its area covered with wetlands in the historical period, 24% in 1958, and has subsequently had substantial drainage to a current coverage of 11%. Therefore, the specific results presented here reflect the substantial drainage in Smith Creek and should not be directly extrapolated to other basins. Despite these caveats, the physical basis of the Prairie Hydrological Model, its lack of calibration to current conditions and its detailed representation of wetland dynamics and variable contributing area mean that it is the most reliable and sophisticated estimate now available of the hydrological response to wetland drainage in the Canadian Prairies. This model simulation exercise shows that wetland drainage increases the annual flow volumes and peak daily discharges substantially, with notable increases in the flow volume and the peak discharge of the flood of record due to wetland drainage that has already occurred in Smith Creek. Relative increases in annual flow volumes and peak daily discharge increase with decreasing annual flow volume, showing that the relative hydrological impact of wetland drainage is*

greatest in moderate to low flow years, but the magnitude of the hydrological impact of wetland drainage is greatest in flood years".

20.5.2 Application to Assiniboine River Floods

A conceptual level analysis was conducted to determine what the Pomeroy findings could imply for Assiniboine River flood magnitudes (Pomeroy et al., 2014). Storage available for runoff reduction varies with antecedent moisture conditions. Pomeroy's results on Smith Creek suggest that for major floods preceded by the relatively empty natural wetlands of the 1958 era, wetland restoration of a 2008 condition could reduce runoff by about 30%. The reduction could be much less for flood events preceded by major floods in the antecedent year or two, which would have filled the wetlands existing in 1958, leaving little or no space to store additional water. Three wetland restoration scenarios equivalent to a 10%, 20% and 30% reduction in runoff volume for large floods were examined. The 30% being taken as the upper limit for large floods was based on Pomeroy's study. The analysis was restricted to the Assiniboine River. The portion of the Souris River Basin upstream of Westhope, North Dakota was excluded, as it presented particular difficulties due to international issues and was highly regulated by reservoirs such as Rafferty, Alameda, Boundary in Saskatchewan and Lake Darling and the Fish and Wildlife Refuges near Upham in North Dakota.

The analysis focused on the potential effect of wetland restoration on three recent large Assiniboine River floods, which occurred in 1976, 1995 and 2011. The floods of 1976, 1995 and 2011 were chosen since they were the largest observed floods during the past 50 years for which stream flow data on tributaries and the mainstem was available. Interest for the current Assiniboine River & Lake Manitoba Basin Study is primarily focused on what effect wetland restoration might have on low frequency floods, which cause the greatest damages. There was insufficient hydrometric data for the floods of the 1800's and early 1900's to enable analysis of the pre-drainage era. The three selected floods have different characteristics in terms of spatial severity and timing of runoff from various portions of the watershed at large, and thus the analysis of wetland effects would include various types of floods. The 1976 snowmelt flood produced record high peak flows (at the time) on the Assiniboine River and Souris River with peaks flows at Brandon and Wawanesa being nearly equal. The 1995 Flood was very severe in the Upper Assiniboine River, while the 2011 Flood produced new record high flows from St. Lazare to Portage la Prairie, as well as on the Souris River. The 2011 Flood was extended in

time and produced a much higher volume of flow than that of 1976, even though peak flows at Portage la Prairie were similar. Other flood events, such as design floods with frequencies of 50- to 500-year floods could have been selected; but analysis for such floods would have been more complex due to the need to generate flow hydrographs for all the sub-watersheds.

A map for the prairie region showing the natural non-contributing drainage areas was used to estimate the percentage of each tributary's non-contributing (natural undrained) drainage area (PFRA, 2008). A 0.9 m (3 ft) average depth for wetlands was assumed for the analysis. This average depth was a very approximate estimate as no inventory of the millions of prairie wetlands in terms of their size and depth was available. This estimate was based on discussions with a number of farmers in western Manitoba and was considered a reasonably achievable target. The available storage volume was then estimated based on an average incremental depth of 0.9 m (3 ft) due to restoration works. Details of the analysis are included in Appendix M and include a summary of the drainage areas and estimated wetland areas for each tributary.

It was found that for each tributary the storage volume opportunity due to wetland restoration was larger than 30% of the 2011 runoff volume. This indicated that it would be possible to store the volume of water associated with 30% of the runoff for the largest flood on record (30% being taken as the upper limit for runoff reduction for large floods based on Pomeroy's study (2014) as previously discussed). In the analysis, recorded tributary hydrographs were reduced by 10%, 20% and 30% due to wetland restoration and routed through the system using the Muskingum flood routing procedure discussed in Section 8.2.3 of the report. Details of the analysis are included in Appendix M and include flood hydrographs that were generated at the following locations:

- Lake of the Prairies levels (Shellmouth Reservoir).
- Assiniboine River flows near Russell.
- Assiniboine River flows near Miniota.
- Assiniboine River flows at Brandon.
- Souris River flows at Wawanesa.
- Assiniboine River flows at Portage la Prairie.

The analysis showed significant main stem flow reductions due to wetland restoration could be achieved. The effect was relatively small on the Souris River due to the assumption that no wetland restoration would occur on the large basin area upstream of Westhope, North Dakota.

On the Assiniboine River mainstem, as shown in Table 20.3, flow reductions would be significant. The largest values generally applied to the upper reaches of the Assiniboine River near Shellmouth, whereas the lower values were generally applicable to the Assiniboine River at Portage la Prairie.

Table 20.3
Potential Assiniboine River Peak Flow Reductions

Type Flood	Wetland Restoration Scenario		
	10%	20%	30%
1976	8 to 10%	16 to 20%	24 to 30%
1995	9 to 18%	20 to 51%	28 to 52%
2011	8 to 4%	19 to 30%	29 to 43%

The cost of wetland restoration was estimated to compare the option of wetland restoration to other flood mitigation alternatives analyzed for the Assiniboine River & Lake Manitoba Basins Flood Mitigation Study. The cost of a restoration project varies substantially depending on a few variables such as size of the project, topography, extent of drainage, and local land values. Additionally, there are different ways to restore wetlands depending on site conditions and location. Based on experience, an average cost for wetland restoration was provided by DUC considering the variables noted above and the different types of wetland projects. The average cost for wetland restoration was estimated at $1,000 to $2,000/ha ($400 to $800/ac), and included administration, licensing, surveys and construction costs. The additional cost for the securement or purchase of land would vary substantially depending on location. Using the standard rates adopted for the study, non-arable land would be purchased at a rate of $1,000/ha ($400/ac), and arable land at a rate of $5,000/ha ($2,000/ac).

The cost estimate for wetland restoration was developed for the 10%, 20% and 30% reduction in runoff volume scenarios described previously that were based on Pomeroy's findings. Based on the results of the analysis, approximately 73,000 ha (180,386 ac) of wetlands restored would be necessary to achieve a 10% reduction during a 2011-type flood. For the 20% and 30% reduction scenarios, respectively, 146,000 ha and 219,000 ha (360,772 ac and 541,159 ac) would be necessary. Based on an average cost of $1,500/ha ($600/ac) for wetland restoration, and assuming that 50% of the land purchased is arable, the estimated cost of wetland

restoration would be approximately $512 million to achieve a 10% reduction in peak flow. This estimate also included a 20% allowance for indirect costs such as engineering, approvals, administration and owner's costs, and a 30% allowance for contingency. The estimated costs for the 20% and 30% reduction scenarios were approximately $1.0 billion and $1.5 billion, respectively. The estimated costs have been summarized in Table 20.4.

Table 20.4
Estimated Cost of Wetland Restoration Scenarios

Item	Rate	Estimated Cost (Million)		
		10% Scenario	20% Scenario	30% Scenario
Restoration Cost	$1,500/ha ($600/ac)	$ 109.5	$ 219.0	$ 328.4
Land acquisition – non-arable	$1,000/ha ($400/ac)	$ 36.5	$ 73.0	$ 109.5
Land acquisition – arable	$5,000/ha ($2,000/ac)	$ 182.5	$ 364.9	$ 547.4
Indirect	20%	$ 66.7	$ 131.4	$ 197.1
Contingency	30%	$ 118.2	$ 236.5	$ 354.7
Total Cost		$ 512.4	$ 1,024.7	$ 1,537.1

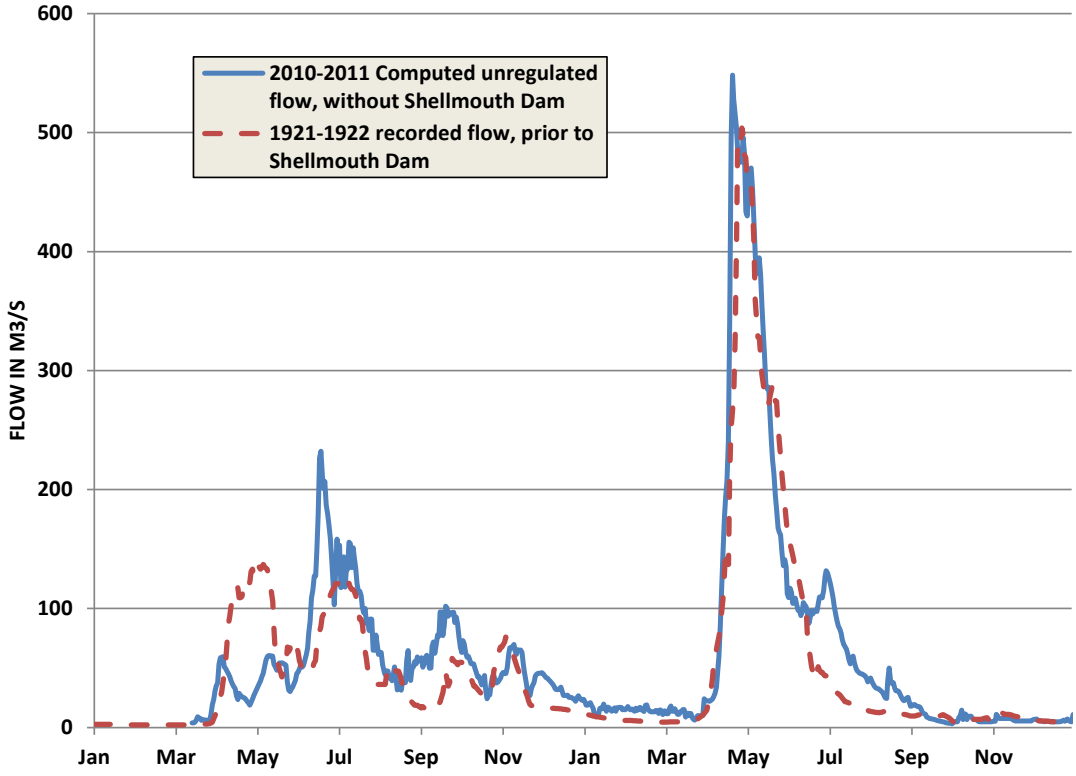
The results of this study suggest that wetland restoration could be an effective way to reduce flood hydrographs on the Assiniboine River for various types of floods. The costs associated with the restoration option are significant; however, a more detailed assessment would be needed to enable a reliable benefit / cost analysis, before an extensive wetland restoration program might be considered.

20.6 THE 1922 AND 2011 FLOODS

A comparison of the 1922 and 2011 flood hydrographs for the Assiniboine River near Russell may help shed some light on the effect of wetlands on low frequency floods. Figure 20.3 shows the 1921 to 1922 and 2010 to 2011 flood hydrographs for the Assiniboine River near Russell. The 2011 Flood was the largest volume runoff event to be experienced in the Upper Assiniboine River Basin since 1922. To compare the 1922 and 2011 floods, unregulated flows (without the effects of Shellmouth Dam) for the Assiniboine River near Russell were computed for 2010 and 2011. The area under each hydrograph represents the total runoff volume. Both the 1922 and

2011 floods produced similar runoff volumes estimated at approximately 1,500,000 dam^3 (1,200,000 ac-ft).

Figure 20.3
Comparison of 1922 and 2011 Floods on the Assiniboine River near Russell



Note: Computed unregulated peak flow for 2010 / 2011 may be slightly overestimated due to the effects of a long and narrow Shellmouth Dam reservoir, which is not considered in the Shellmouth Inflow Available for Outflow calculations.

While data networks in 1922 were sparse, indications are that winter precipitation, plus effective spring rain for the 1922 Flood only, exceeded that of the 2011 Flood by a small margin. The floods produced virtually identical runoff volumes and peak flows near Russell. Based on the Pomeroy report (2014), one might have expected a much greater runoff volume in 2011. A closer analysis suggests some evidence of the effect of antecedent wetland conditions on flood magnitude. It appears that antecedent conditions for the 1922 Flood were even wetter than those of the 2011 Flood, which were largely due to the 140 mm (5.5 in) of rain in September 2010. This likely saturated soils and filled the multitude of wetlands existing at the time, resulting in an effective drainage area similar to that with the partially drained wetlands of 2011. Thus, the

1922 Flood might have been a little different if it had taken place under 2008 wetland conditions, much as the 2012 Flood on Smith Creek showed little modelled effect of drainage on runoff volume due to the 2011 Flood having filled wetlands. The 1922 Flood thus suggested that some low frequency floods were not greatly affected by wetland drainage if antecedent conditions were very wet. It also raised the question as to whether the modelled increase in the 2011 Smith Creek runoff was over-estimated due to possible inaccuracies in the PHM model. Since wetland storage was computed based on running the model for four years prior to 2011, even a relatively small model bias could have a cumulative effect and under-estimate wetland storage at the start of the 2011 Flood. If so, this would explain the large modelled increase in runoff due to wetland drainage. The 1922 Flood information also serves as a reminder that extrapolation of the Smith Creek findings to other portions of the Assiniboine River watershed should be done with caution.

20.7 WETLAND PRESERVATION

Wetlands are being lost at a greater rate than they are being restored. Ducks Unlimited Canada (DUC) conservatively estimates that an average of 6 ha (15 ac) daily of wetland are being lost or almost 2,200 ha (5,436 ac) of wetlands every year in southwestern Manitoba (DUC, 2012). Even with the Wetland Restoration Incentive Program (WRIP) established in 2008 and partnered by DUC and the Manitoba Habitat Heritage Corporation (MHHC), which provided incentives to landowners to restore previously drained wetlands, wetlands were lost ten times faster than the program was able to restore them. The program, at best, even if it was fully subscribed would have had the limited potential to offset 11% of the wetland loss rate in the region. Reports indicate the program was never fully subscribed and has since been cancelled. The WRIP was the most financially attractive wetland restoration program on the prairies.

Ducks Unlimited proposes that wetlands should be “conserved first, restored later” (Pascal, 2012). Their reasoning is that:

- There are ample opportunities to restore prairie wetlands.
- Restoring just 25% of the wetland lost in southern Saskatchewan and southwestern Manitoba could cost $250 million.

- Watershed modelling in Manitoba and Minnesota indicates that 50% of the wetlands need to be restored to be cost effective, and wetland protection deserves higher priority (Wang et. al., 2010).
- During Manitoba's Wetland Restoration Incentive Program (the most financially attractive wetland restoration program on the prairies), Ducks Unlimited believes that 10 ha (25 ac) of wetland were being lost for every one restored (Pascal, 2012, slide 62).

Currently, Manitoba does not authorize the drainage of Class 3, 4, and 5 wetlands (Stewart and Kantrud Classification System, 1971). Illegal drainage, however, continues despite the restriction on wetland drainage. Manitoba is currently engaged in developing new drainage regulations, which will enshrine wetland protection. Protection measures include increased enforcement including stiffer fines and penalties, as well as the development of a proposed wetland mitigation strategy, where if proponents of projects want to drain a wetland, the project has to be done in a manner that ensures there is no loss of wetland benefits. Consultations on the new drainage regulations are currently on-going.

The recently completed provincial Surface Water Management Strategy seeks to end further loss of the benefits provided by wetlands and includes a plan for overhauling drainage licensing that would streamline approvals for routine drainage while protecting seasonal wetlands. The strategy proposes that the following actions be implemented by 2020 to preserve wetlands:

- **No Net Loss of Wetland Benefits** – Drainage licenses will be generally unavailable for permanent, semi-permanent and seasonal wetlands; and where they must be drained with no alternative, there must be mitigation to more than compensate for the loss of wetland benefits.
- **Run-off Retention Pond Network** – Research by the University of Manitoba will lead to proven retention pond models that will manage wet period runoff.
- **More Protected Areas for Wetland Benefits** – Additional protected areas in Agro-Manitoba will hold more water on the land in natural grasslands and wetlands.
- **Implementation of a Watershed-Based Planning Framework for Drainage** – Through cooperation with conservation districts, a watershed-based planning framework for local drainage that incorporates the concept of no net increase in water export (on a watershed basis).
- **Provincial Wetland Strategy** – A strategy that inventories and provides adequate protection to all types of wetlands in Manitoba. This strategy will ensure that Manitoba's wetlands are adequately protected and provide strong provincial leadership to ensure collaborative participation among stakeholders to conserve and protect wetlands.
- **Protected Areas** – Increase the number and extent of small protected areas in Agro-Manitoba to conserve remnant prairie, vulnerable wetlands of significance, and

fragmented habitat and habitat corridors for species at risk through the new Protected Areas Strategy currently being developed. The Province will continue to work with partner agencies to secure conservation agreements on privately owned land to protect valuable habitat and natural areas in perpetuity.

- **Crown Lands** – By minimizing drainage of Crown land, and where appropriate, using this provincial resource to retain water in the watershed to improve drought resiliency, reduce flood risk, improve water quality and support healthy ecosystems.

In addition to provincial actions to preserve wetlands, $320 million has been dedicated to flood protection and water control infrastructure including surface water management, drainage, retention, dams and control structures. This is in addition to approximately $4 million that is expected to be invested in on-farm water retention projects over the next five years through conservation districts, the Manitoba Habitat Heritage Corporation, the Nature Conservancy of Canada or Ducks Unlimited Canada (Manitoba, 2014b and 2014c). The Manitoba Habitat Heritage Corporation (MHHC) works with landowners to address on-farm and community-level water concerns through wetland restoration. Restoring wetlands under paid 10-year term contracts or perpetual agreements is a simple and effective way to reduce nutrient loading to Manitoba's lakes and rivers, while improving the land's natural resilience to extreme wet and dry weather events.

Consideration should also be given to conducting a pilot study to better define the costs and benefits of wetland restoration on flood attenuation and to expand on the proposed investments listed above such as through the MHHC. A paired watershed approach could be considered for the study with one watershed maintained in its drained state and the other selected for wetland restoration. The project would be a multi-year research initiative lasting at least ten years. Streamflow should be monitored on both watersheds prior to any wetland restoration to understand the comparative hydrologic response of the two watersheds in their drained state and develop baseline data. Following wetland restoration, streamflow monitoring on both watersheds is needed to assess the benefits of wetland restoration on flood attenuation locally and on a regional scale. The assessment of wetland restoration on water quality could also be a component of the study. The total cost of a multi-year pilot study on wetland restoration to define the costs and benefits would be in the range of $10 to $20 million.

In summary, the Province of Manitoba has taken a number of actions to limit flood damages due to the loss of wetlands. The most significant action is "No Net Loss of Wetland Benefits".

20.8 DISCUSSION

The changes in drainage patterns in the prairie landscape due to human activity have been thought to contribute to increased flooding in recent years, but the relation between changes in the landscape and the severity of flooding is not clear (Winter et al., 1984). Prior to basin development and to any drainage activities, there were historical accounts of major flooding occurring in the Red and Assiniboine River Basins. Major floods of 1776 and 1826 in the Red River Valley far surpassed the severity of the flood of 1997. One of the longer-term records of changing hydrologic conditions in the prairies comes from measurements made of the lake level in Devils Lake, North Dakota. The wet periods of the early mid-1990's to the present, reflected in large rises in Devils Lake, and were also periods of high water in prairie potholes in North Dakota. Wet and dry episodes evident in the Devils Lake record since 1930 are superimposed on a general rise in lake level to heights similar to those last recorded in the mid-1800s. Thus, using the Devils Lake record as a proxy for hydrologic conditions in the prairie of North Dakota, the recent wet episode may have brought wetland water levels to heights not encountered for 130 years (LaBaugh et al., 1998).

When considering the impacts of wetland drainage, it is difficult to separate land use changes such as the conversion from grass lands and forest cover to agriculture crops. Van Der Kamp et al. (1999) explained that adopting farming practices, which represent a land use approaching 'natural or original' prairie conditions, has the potential to reduce runoff. The conversion of upland cover to permanent grass cover had profound effects on the hydrologic balance of the wetlands. Wetlands in the grassed areas became much drier. In fact, some wetlands dried up and remained dry even in years of heavy precipitation (van der Kamp et al., 1999).

The impact of a wetland on peak flows is dependent on its location in the catchment. Small numbers of headwater wetlands may have little effect on downstream hydrographs. This effect was illustrated by the Fisher River Study (AECOM, 2009) where drainage occurred in the headwater of the watershed. The impacts decreased as they travelled downstream because of the flood attenuation effects of roadway crossings that restrict flow and cause backwater storage effecting flood plain storage once flows exceed channel capacity and the timing of the downstream tributary runoff. This is the reason why floodplain storage restoration is considered to have more effect than headwater wetland restoration. When looking at individual wetlands,

the relative location of a wetland in a catchment can affect its flood attenuation capabilities. It was found that, for downstream wetlands, the attenuating effect extended a considerable distance downstream. In upstream wetlands, which tend to be much smaller, the detectable percent increase in flow due to the degradation of the wetland declined quickly with distance downstream. In addition, downstream mainstem wetlands are more effective in reducing downstream flooding than upstream wetlands. Downstream wetlands, such as those along the middle and lower reaches of the Mississippi River and its tributaries, are more effective at reducing downstream flooding than are headwater wetlands, largely as a result of larger storage capacities (Ogawa and Male, 1986). Wetlands, however, cannot be seen as single units. When collectively examined on a catchment scale, they are all interlinked. Therefore, although an isolated wetland may perform a significant flood control function, effective control is more often a result of the combined effect of a series of wetlands within a particular catchment. It is this "spatial aggregation of wetlands" in association with their "cumulative effects" that has an important bearing on the role of wetlands in flood attenuation. Thus, when considered singly, upstream wetlands may appear to have an insignificant effect on flood attenuation. However, upstream wetlands are often numerous, and when considered collectively their cumulative effect may be considerable (Kotze, 2000).

Early research on the role that prairie wetlands play in high and low frequency flood events is primarily based on hydrologic modelling. Hydrologic modelling requires a number of simplifications because the topography of the prairie pothole region is unique and poses challenges for modelling the dynamic contributing area of a watershed that varies from year to year. This is due to the complex fill-spill connectivity between wetlands, which are irregularly distributed and, as a result, do not follow a simple, cascading flow path to the outlet. Many current hydrological models (such as SWAT, HEC-HMS, HSPF) do not incorporate the dynamic potential of surface storage and the effect that this storage has on contributing areas in the basin. Consequently, delineated sub-catchment drainage areas are almost always kept static when in reality the sub-catchment boundary is not static but dynamic and is defined by the basin's antecedent storage conditions. This is an issue when modelling a sequence of years as shown in the Smith Creek watershed study. However, the Prairie Hydrologic Model now accounts for variable contributing areas by including a wetland module that permits open water evaporation and fill and spill from wetlands (Pomeroy et al., 2014). When simulating a single

year such as a 100-year flood, the variable contributing drainage area is not as big an issue as it can be pre-set based on the hydrologic conditions.

The PHM modelling results obtained for Smith Creek (42% increase) versus about 5% for most previous studies for a low frequency event is very significant. This estimate is vital in understanding the role of wetland restoration effects on Assiniboine River floods. Pomeroy's study of Smith Creek is based on high resolution LiDAR data, which provides reliable information of the fill and spill dynamics of wetlands. Previous studies did not have access to this type of information. The PHM now includes the concepts of variable contributing drainage based on this data. Because of this data, the PHM should provide a definite improvement in the simulated annual runoff volumes. However, the volume estimates should be used with caution as the initial antecedent conditions used in the analysis are based on running a four-year period prior to the major flood of 2011 and not on recorded conditions. Even a relatively small model bias could potentially have a cumulative effect and might result in an error in computed wetland storage at the end of four years. The revised PHM does not appear to consider the hydraulic controls on wetland drainage. These include the routing through wetland basins, which consider the outflow characteristics of retained versus drained wetlands, hydraulic capacity of drainage ditches and channels, and backwater from receiving streams, rivers and other water bodies. These controls dampen and attenuate flows. This limitation appears to affect the timing and shape of the simulated runoff hydrographs, which are not consistent with the recorded flow hydrographs. Statistical analyses of long-term precipitation and stream flow records in the Assiniboine River Basin did not find statistically significant effects of drainage on peak flows and annual flows (Ehsanzadeh et al., 2012). It should be noted that this result is intriguing in light of modelling studies such as Pomeroy's, which have suggested that wetland drainage in particular leads to increased runoff. Therefore, the stated impact of wetland drainage on peak flows for Smith Creek determined by Pomeroy is likely overestimated. In cases where only volumes are of concern (for example dead drainage basins where there is no outflow, such as Whitewater Lake), such a shortcoming is of no concern. However, in cases where peak flows are of primary concern (overbank flooding) particularly during moderate and large flood events, caution needs to be taken in interpreting the Pomeroy findings.

Wetland drainage results in increased runoff volumes. This sensitivity in flow volume to wetland change is crucially important for the water balance of downstream water bodies. Increased

runoff volumes can limit the operational flexibility of lakes and reservoirs during flood events. Examples are Shellmouth Dam, Lake Manitoba and Dauphin Lake, which have a limited capacity to manage flood flows.

20.9 CONCLUSIONS

The issue of quantifying the role of prairie wetlands' flood attenuating effects on different flood magnitudes and the implications on Assiniboine River floods is very complex. A statistical analysis of long-term precipitation and stream flow records in the Assiniboine River Basin did not find statistically significant effects of drainage on peak flows and annual flows (Ehsanzadeh et al., 2012). Early research indicated that the drainage of prairie wetlands had a relatively minor impact on the peak flows of low frequency flood events such as the 2011 Flood in the Assiniboine River & Lake Manitoba Basins. However, the most recent research on the role of prairie wetlands on flooding by Pomeroy, which indicates wetlands drainage may play a significant role in flooding, is unquestionably the best available tool for simulating the hydrology of prairie watersheds, including intact and drained wetlands (Pomeroy et al., 2014). Because of the complexity of the wetland drainage issues, there is a need for more thorough basin-wide and geographically-based assessments to verify the Pomeroy findings.

Based on Pomeroy, it would generally appear that the relative increases in annual flow volumes and peak daily discharge increase with decreasing annual flow volume. This shows that the relative hydrological impact of wetland drainage is greatest in moderate to low flow years, but the magnitude of the hydrological impact of wetland drainage is greatest in flood years.

The Prairie Hydrologic Model appears to reasonably simulate the impact on flood volumes that would result from wetland drainage or wetland restoration; but it appears to overestimate the impact on peak flows, particularly during moderate and low frequency flood events, due to the lack of hydraulic controls in the model.

Caution should be used in interpolating the simulated Smith Creek results to other watersheds. The results, some of which are based on interpolated basin wetland conditions, could not be verified with field observations. Streamflow records on Smith Creek start in 1975, thus there is a lack of pre-drainage stream flow observations of the 1950's with which to verify the modelling.

Antecedent conditions play a significant role in wetland flood reduction. The Smith Creek results suggest that, for major floods preceded by relatively empty natural wetlands, wetland restoration could reduce runoff by about 30%, for flood events preceded by major floods. In the antecedent year or two, which would have filled the wetlands leaving little or no space to store additional water reductions, the reductions would be much less.

With the current information, it appears that wetland drainage was a contributing factor, along with the extreme climatic conditions, to the 2011 Flood in the Assiniboine & Lake Manitoba Basins.

The percentage increase in peak flow rates cannot be applied directly in assessing flood damages, as river channels have greater flow capacities with increasing stages.

The proposed provincial action of “No Net Loss of Wetland Benefits” should limit flood damages from the future loss of wetlands.

A multi-year pilot study on wetland restoration should be considered to determine the costs and benefits of wetland restoration on flood attenuation and to expand on proposed investments on water retention projects such as through the Manitoba Habitat Heritage Corporation. A paired watershed approach should be considered. The total cost of a multi-year pilot study would be in the range of $10 to $20 million. Water quality assessment could also be included in the study.

21.0 DRAINAGE ISSUES IN SASKATCHEWAN

21.1 INTRODUCTION

The issue of artificial man-made drainage has been long standing and contentious in the Assiniboine River Basin. Drainage practices, to some degree, are required to sustain agriculture. Drainage issues within Manitoba exist, and illegal drainage continues despite restrictions on wetland drainage. In Saskatchewan, drainage issues also exist, which has had impacts in both Saskatchewan and downstream in Manitoba. Manitoba is currently engaged in developing regulations, which will enshrine wetland protection. Saskatchewan is also examining regulations to deal with illegal drainage.

Manitobans believe that drainage in Saskatchewan has exacerbated their flooding and are unaware of any limitations being imposed on Saskatchewan drainage. If limitations are being imposed, they believe they are being ignored and uncontrolled drainage is still occurring because penalties are not being enforced. Manitobans believe that Saskatchewan drainage has significantly increased in anticipation that stricter enforcement may be forthcoming. One of the greatest concerns is that Manitoba producers cannot file a drainage complaint directly against a Saskatchewan offender but must deal with it through the political process. Artificial man-made drainage has also occurred, and continues to occur, in other interprovincial watersheds including the Souris, Qu'Appelle, Swan, Woody, Red Deer, Overflowing and Carrot Rivers, which affects Manitoba.

The issue of interprovincial drainage has been complicated by the perception of the roles that prairie wetlands play in flood attenuation, a lack of clear scientific information on the flood attenuating effects of wetland drainage for varying flood magnitudes, the inter-jurisdictional aspects, which require bilateral discussions and agreement, and differences in provincial policies and legislation of the two Provinces regarding agricultural drainage practices.

The level of issue has not been either as apparent, or as public, as in the Upper Assiniboine River Basin specifically in the Russell / Langenburg area. This area is immediately downstream of the Shellmouth Dam and affects producers along the Assiniboine River Valley. The issue has been ongoing for at least 40 years without a resolution in sight. The extent of Saskatchewan

drainage is illustrated by the Smith Creek Watershed. The basin has been partially drained, with the extent of wetland area estimated to have decreased from 24% to 11% of the watershed area from 1958 to 2008 (Pomeroy et al., 2014). Even as early as the 1990s, Manitoba residents along the Manitoba-Saskatchewan border requested a drainage moratorium on development of the proposed Langenburg Project, Saskatchewan (Briere, 1994).

Under the *Saskatchewan Watershed Authority Act*, 2005, drainage is *"any action taken or intended for the removal or lessening of the amount of water from land..."* (Ministry of Environment et al., 2010). Cited examples of "drainage" are clearing, deepening, widening or straightening a watercourse to move more water or to move water more quickly, creating a channel to redirect a watercourse, creating a ditch to remove water from a wetland or other surface water body, infilling a wetland or water body to remove or reduce the depression where water collects, installing tile drainage to drain sub-surface water, building a structure to prevent flood waters from entering all or part of a floodplain, placing culverts and constructing ditches alongside or under roads.

21.2 STUDY SCOPE

To help understand the scope and complexity of the interprovincial drainage issue the following work has been undertaken:

- An overview of the various studies and agency activities directed to resolve the interprovincial drainage issue.
- An analysis of streamflow data in an attempt to capture the influence of Saskatchewan drainage practices.
- A discussion of Saskatchewan's current drainage policies including the status of the 25-Year Saskatchewan Water Security Plan, published October 15, 2012 and the potential for the plan to resolve some of the ongoing issues.
- A description of recent Saskatchewan drainage activities and policy enforcement.
- Defining the impacts and providing recommendations for further assessment as appropriate and in coordination with other ongoing initiatives.
- Concluding statements regarding the concerns of Manitobans related to the complexity of the subject, the past efforts that have been undertaken, the current policies that exist and enforcement practices.

21.3 OVERVIEW OF JOINT ACTIVITIES

Over the years, numerous cooperative activities have been undertaken between Manitoba and Saskatchewan to resolve the interprovincial drainage issue. The following is a summary of activities undertaken to understand and resolve the interprovincial drainage issue.

21.3.1 Initial PPWB Drainage Research Initiative

In 1979, it was recognized that limited knowledge existed for prairie hydrology; and there were no adequate technical means of determining the effects of drainage. The Committee on Hydrology (COH), a technical committee of the Prairie Provinces Water Board (PPWB), was asked to develop an applied research program that might determine the effects of artificial drainage on downstream flows of prairie streams. It was at this time that a temporary moratorium was applied to organized drainage projects in the Saskatchewan portion of the Assiniboine River Basin.

The Prairie Province Water Board (PPWB, 2012) was created in 1948 by the Provinces of Alberta, Saskatchewan, and Manitoba and the Government of Canada with the goal of ensuring that water resources are shared fairly. In 1969, the four governments changed how the Prairie Provinces Water Board operated by signing the Master Agreement on Apportionment (MAA). This Agreement established an intergovernmental framework to manage transboundary waters. The MAA is administered today by Environment Canada, Agriculture and Agri-Food Canada, Alberta Environment and Sustainable Resource Development, Saskatchewan Water Security Agency and Manitoba Infrastructure and Transportation. The purpose of the MAA is to apportion or share water equitably between the Prairie Provinces and to protect transboundary surface water quality and groundwater aquifers.

From the PPWB Charter, the mission of the Board is:

1. To ensure that transboundary waters are protected and equitably apportioned and protected in accordance with the Master Agreement on Apportionment.
2. To provide a forum for the exchange of information in order to prevent and resolve transboundary water management conflicts.
3. To promote cooperation in transboundary water management.

In 1983, the Board adopted the COH report "A Recommended Procedure to Deal with the Impact of Artificial Drainage on Downstream Flows" (COH, 1983). The method has since been used by water resource agencies to assess the impacts of drainage on stream flow. The technique implies that drainage projects increase the drainage area directly contributing to flow.

The technique is based on a contributing drainage area concept. The technique implies that drainage projects increase the drainage area directly, contributing to flow above the effective drainage area (the drainage area that contributes to runoff during a median runoff event). It is hypothesized that, ultimately, at some extremely high flow event between 1:500 or 1:1,000 years, the contributing area becomes coincident with the gross drainage area. As meteorological input and other watershed conditions contribute to a runoff event that exceeds the magnitude of the median event, a progressively greater proportion of the watershed is assumed to contribute to the flow, due to the filling and spilling of water otherwise detained in the sloughs and potholes. This technique was used with mixed success in a number of watersheds in the Assiniboine River Basin.

21.3.2 1984 Study on the Effects of Drainage

Because of the proposed Langenburg East Conservation and Development Area Authority drainage project, the technique described above was applied in a December 1984 Prairie Farm Rehabilitation Administration (PFRA) Hydrology Report No. 101, "Evaluation of the Effects of Drainage Developments in the Rural Municipalities of Churchbridge and Langenburg" (Scott and Martin, 1984). This report represents an early effort to assess the effects of drainage on peak flows and runoff volumes in the Assiniboine River Basin. Seven small watersheds in the Rural Municipalities of Churchbridge and Langenburg in Saskatchewan were the focus of the study. These watersheds included Smith Creek on the south to Blackbird Creek on the north, and five tributaries to either the Assiniboine River or Blackbird Creek, which cross the provincial boundary between Smith and Blackbird creeks. This area contributes to the Assiniboine River immediately downstream of Shellmouth Dam. For each of the seven watersheds, the objectives of the study were to estimate the effect of the 1981 level of agricultural drainage and potential future drainage on design flood hydrographs at the Saskatchewan-Manitoba boundary. Design hydrographs were developed for all seven watersheds for various drainage conditions and return periods of flooding. As expected with the study method, the design flood peaks all show

an increase over natural conditions, with increased drainage conditions. However, again as the technique dictates, the amount of increase in peak values decreases for the less frequent events. The study conclusions recognized that the approach taken in the study does not account for road crossings of the streams in the study watersheds and that those crossings likely provide significant reduction of peak flows due to limited capacity and temporary storage of flows upstream of the crossings. In considering the applicability of the results of the Upper Assiniboine River Basin Study (UARBS), the Drainage and Flood Control Committee (DFCC) had several reservations (Manitoba Conservation et al., 2000). *"By taking this approach, it is predetermined that drainage will increase both peaks and volumes"* (Scott and Martin, 1984). The DFCC felt that while this result may indeed be correct, an approach was needed for the study, which addressed some of the weaknesses of the drainage area approach".

21.3.3 Upper Assiniboine River Basin Study

When an exceptionally high spring runoff in 1995 caused flooding throughout the Upper Assiniboine River Basin, downstream Assiniboine River valley land producers in Manitoba reiterated a long-standing criticism that flooding in their area was significantly magnified due to drainage of wetlands in the upper reaches of the basin. The 1995 Flood was about a 100-year event. In July 1995, officials from the governments of Manitoba and Saskatchewan met to discuss interprovincial drainage issues. These discussions resulted in a Memorandum of Understanding between Manitoba, Saskatchewan, and Canada in October 1996, to undertake a comprehensive study of water management issues in the basin (Manitoba Conservation et al., 2000). Of particular interest was the drainage of wetlands associated with agricultural development and practices in the basin.

For the duration of the study, a moratorium on all new drainage activities was in place, in the interests of good water management and to maintain constant conditions throughout the basin during the study period and avoid compounding existing issues. Many Manitobans felt the drainage moratorium was ineffective.

The study was not able to model the effect on basin hydrology from individual changes to land such as wetland drainage, deforestation and agricultural intensification. A hydrologic analysis of streamflow records in the Upper Assiniboine River Basin indicated that runoff volumes have

been increasing (Manitoba Conservation et al, 2000) and that changes in basin land cover, including loss of wooded area, loss of wetlands, improved drainage networks, and increased road development have resulted in increased runoff volumes over what would have occurred naturally. However, it was not possible to determine the hydrologic effect of individual land cover changes on runoff by examining only recorded streamflow data.

The study concluded that climatic processes dominated the generation of major flood events; the greatest relative impact of drainage of wetlands was during moderate runoff events, with the effect diminishing for increasingly larger flood events; and, drainage of wetlands has little effect on major flood events. The study recommended the continued need to develop a comprehensive runoff simulation model for prairie conditions such as those found in the Upper Assiniboine River Basin.

21.3.4 Prairie Provinces Water Board Drainage Workshop

A Prairie Provinces Water Board Drainage Workshop was convened on September 24 to 25, 2001 to: review the current state of knowledge and practice for assessment of the impact on hydrology of agricultural drainage on local, watershed and basin-wide scales; identify what operational tools were needed for the assessment of agricultural drainage projects; and outline a course of action, which will lead to the development of the necessary operational tools (PPWB, 2004). In the workshop, the need for the following tools was identified:

- Decision Support System.
- Drainage inventory and database.
- Prairie hydrologic model research and development.
- Open source policy for data.
- Predictive model to assess downstream effects.

21.3.5 Prairie Hydrological Model (PHM)

In 2010, because of the broad interest in the issue of wetland drainage, the Prairie Habitat Joint Venture Policy Committee, Agriculture and Agri-Food Canada, Prairie Provinces Water Board, Saskatchewan Watershed Authority, Manitoba Water Stewardship and Ducks Unlimited Canada

funded the University of Saskatchewan's developed of a Prairie Hydrological Model (PHM) (Pomeroy et al., 2010). The study objectives were to develop and test a hydrological model suitable for wetland-dominated prairie drainage basins and to use the model to estimate the sensitivity of streamflow to changes in drainage and land use. This study was the predecessor to 2014 Pomeroy's research findings on the Smith Creek watershed (Pomeroy et al., 2014) that appeared to confirm the annual streamflow volume and peak daily discharge have a remarkably strong sensitivity to wetland drainage.

The 2014 Pomeroy report suggested that drainage of wetlands since 1958 had greatly increased runoff volumes and peak flows on Smith Creek, a tributary of the Assiniboine River which flows out of Saskatchewan and joins the Assiniboine River downstream of Shellmouth Dam near the old PTH 16 crossing. Simulations of runoff changes were based on the effects of wetland drainage during the period from 1958 to 2008. Over this period, the extent of wetlands is estimated to have decreased from 24% to 11% of the watershed area. Streamflow simulations for Smith Creek indicated an average 55% increase in annual runoff volumes based on wetland loss from 1958 to 2008. The increase was largest for years with low runoff, ranging from 200% to 300% increases. The increase in runoff volume for the largest flood on record (2011) was computed at 42% based on wetland changes. Information pertaining to the effects of wetland changes on Smith Creek peak flows were similar, but were considered less reliable. Even with any limitations of the Smith Creek modelling, the results support the contentions of Manitoba producers that wetland drainage is a contributor to their flooding. Pomeroy's research has helped advance the scientific understanding of the impacts of wetland drainage.

21.3.6 2009 Saskatchewan – Manitoba Drainage Task Force

In September 2008, the Minister of Manitoba Water Stewardship wrote to the Minister responsible for the Saskatchewan Watershed Authority requesting support for interprovincial meetings of staff responsible for licensing drainage works, investigating complaints, and enforcing laws against illegal drainage activities. A cooperative approach was agreed upon to understand and resolve bilateral drainage issues.

A bilateral Saskatchewan – Manitoba Task Force was created in 2009 to develop a strategy for dealing with drainage in Saskatchewan watersheds that could affect lands in Manitoba. A

consultant prepared a report to assess the causes of on-going erosion problems and potential erosion control mitigation along six tributaries of the Assiniboine River (Hatch, 2010). The PPWB was kept informed of the Task Force's progress. The Task Force appears to be no longer active.

21.3.7 Fishing Lake

In November 2010, Saskatchewan submitted a proposal to federal and provincial authorities to license the Fishing Lake Emergency Drainage Project to address flooding issues on Fishing Lake, Saskatchewan located in the Assiniboine River Basin northwest of Russell, Manitoba. The project involved enhancing outlet channel flows into the Assiniboine River that crosses the Manitoba border. Construction work was completed in 2011 to 2012. Saskatchewan and Manitoba reached an agreement on Saskatchewan's Flood Protection Plan for Fishing Lake, as Manitoba had expressed concern that outflows from Fishing Lake could impact flood peaks in the Province and affect water quality. The joint agreement includes conditions under which Saskatchewan will operate the control structure in the channel to reduce water levels without increasing flooding or hurting water quality in Manitoba. A Manitoba-Saskatchewan Operations Committee was formed, created a plan, and cooperatively manage releases to minimize downstream impacts.

21.3.8 Annual Report on Interprovincial Drainage Projects

The PPWB COH prepares an annual report on drainage projects approved in Saskatchewan that have the potential for downstream impacts in Manitoba. However, the report only identifies licensed drainage projects. This is a small proportion of the actual man-made drainage occurring in the Saskatchewan portion of the Assiniboine and Qu'Appelle River basins.

21.3.9 Assiniboine River Basin Initiative (ARBI)

The Assiniboine River Basin Initiative (ARBI) has recently been established to encourage improved conversations and connections between a number of stakeholders from different organizations including conservation groups, scientists, agricultural producers, municipalities,

First Nations, water agencies and affected provincial, state and local governments in Manitoba, Saskatchewan and North Dakota.

The ARBI was modelled along the lines of the Red River Basin Commission. For more than a decade, the grassroots-led Red River Basin Commission has served as an effective forum for cross-border dialogue and planning in the Red River Valley.

The basin-wide approach to water management across the Assiniboine River is intended to complement local watershed planning initiatives and formal government-to-government work through the International Joint Commission and the Prairie Provinces Water Board.

The Manitoba government committed an initial contribution of $50,000 toward the start-up of this grassroots initiative. The Manitoba government's participation in the initiative builds on other recent work to prepare for flooding including implementing a surface water management strategy. The strategy supports an integrated approach to managing surface water including:

- Looking at ways to more effectively consider cumulative downstream effects in terms of flooding or drought.
- Proposing drainage regulation changes to ensure more effort and penalties are focused on illegal drainage.
- Supporting the implementation of a no-net-loss-of-wetland-benefits policy within a sustainable drainage framework.

21.3.10 Summary

Over the years, there have been numerous efforts to resolve the interprovincial drainage issue. On several occasions the issue has been referred to the Prairie Provinces Water Board (PPWB) by governments. However, the PPWB's role is only to provide a forum for the exchange of information to prevent and resolve transboundary water management conflicts, and to promote cooperation in transboundary water management. There have also been direct discussions between Manitoba and Saskatchewan and a number of studies and interprovincial working groups established to resolve the issues of the impacts of new drainage works on downstream jurisdictions, some of them through the PPWB. To date, no resolution has yet been reached. As of the date of this report, the only issue that has been resolved through bilateral discussions has

been the Fishing Lake Project. This is because it required federal government approval. Small scale drainage works in Saskatchewan do not require federal approval.

21.4 EFFECTS OF SASKATCHEWAN DRAINAGE PRACTICES

An analysis of streamflow data for several hydrometric gauging stations was undertaken in an attempt to capture the influence of Saskatchewan drainage practices. The analysis was limited to a literature review, direct data comparison of streamflow data, and mass curve and trend analyses years before and after the best estimate of the change in drainage practices were analyzed. Such an analysis was complicated because of the non-stationary impact of climate and other land management practices. The Upper Assiniboine River Basin was selected for the analysis in collaboration with MIT. It is an interprovincial basin with long-term streamflow records, which allow a comparison of pre- and post-drainage periods and the area with the most interprovincial drainage issues.

The issue of interprovincial drainage of wetlands associated with agricultural development and practices following the 1995 Flood led Manitoba, Saskatchewan, and Canada to undertake a comprehensive study of water management issues in the UARB (Manitoba Conservation et al., 2000). The reason for re-examining the Upper Assiniboine River Basin in this study was that there are now an additional 15 years of record streamflow data and the basin recently experienced extreme floods in 2011 and 2014. The UARB has the long-term hydrometric data required to capture the influences of drainage practices. The data includes a relatively wet period in the mid-1950s. This allows a comparison of pre- and post-drainage conditions during wet periods. Appendix N contains a detailed description of the literature review, direct data comparison of streamflow data, and mass curve and trend analyses.

21.4.1 Literature Review

A literature review was undertaken to assist in understanding the knowledge of the impact of drainage practices on runoff response and application to the Assiniboine River Basin. The purpose was to identity approaches that could be applied in this study, or modified, and the relevancy of the findings to the Assiniboine River Basin. Two relevant studies were identified and are cited as follows:

1. During the Upper Assiniboine River Basin Study (Manitoba Conservation et al., 2000), an analysis of streamflow data was undertaken to determine if land use practices may have had a significant effect on the hydrologic response of the watershed. The analysis included: a comparison of analogue years, mass curve analysis of peak flows and runoff volumes, the derivation of mass curves, analysis of stationarity, analysis of trend, sub-watershed travel times, and regression analysis (split sample). The hydrologic analysis of streamflow records in the basin indicated that runoff volumes have been increasing.

2. Ehsanzadehab et al. (2012) investigated the integrated impacts of precipitation change / variability and landscape changes, specifically wetland drainage practices, on streamflow regimes in wetland dominated landscapes in the Assiniboine and Saskatchewan River Basins of the North American Prairies. Precipitation and streamflow metrics were examined for gradual (trend type) and abrupt (shift type) changes using the modified Mann-Kendall trend test and Bayesian change point detection methodology. Results of statistical analyses indicated that precipitation metrics did not experience statistically significant increasing or decreasing changes; and there was no statistical evidence of streamflow regime change over the study area, except for one of the smaller watersheds. The absence of widespread streamflow and precipitation change suggested that wetland drainage did not lead to detectable changes in streamflow metrics over most of the Canadian portion of the prairies from 1967 to 2007.

In a study of the impact of climatic variability and change in the hydroclimatology of the Lake Winnipeg watershed, Ehsanzadehab et al. (2012) applied the nonparametric Mann–Kendall test modified to account for diverse memory properties (i.e., short term versus long-term) and a Bayesian change point detection model to identify possible segments of time series with inconsistent non-stationary behaviour. It was found that there was no evidence of trend-type, non-stationarity behaviour in the hydroclimatology of the Assiniboine River watershed. Annual runoff in this watershed, however, was characterised with abrupt changes echoing a sequence of wet and dry episodes each lasting over a decade. These wet and dry epochs were not detectable in precipitation inputs, indicating again that the precipitation–runoff relationship in the studied watersheds is highly nonlinear. The non-proportionality of watershed response behaviour to changes in climate inputs suggests that major or even minor precipitation changes in the North American Prairie landscapes may trigger certain thresholds, which govern the hill-slope runoff and wetland fill-spill processes, and lead to significant changes in the areas contributing to runoff at the outlet of the watershed. The sudden expansion (or contraction) of contributing areas, in turn, superimpose abrupt changes on the significant / insignificant trends in runoff caused by precipitation anomalies. The significance of detected trends under a Long-term Persistence (LTP) hypothesis in the Red River watershed suggests that the observed non-stationarity is not solely due to natural climate variability; and it may be attributed, in part, to climate change. The lack of a long-term trend of the annual flow of the Assiniboine River

indicates that long-term cumulative changes in the watershed, including wetland drainage and changes in farming practices have not had a net detectable influence on the flow. This result is intriguing because modelling studies have suggested that wetland drainage in particular leads to increased runoff. The reasons for the absence of identifiable effects were not clear and could probably be best established through more detailed studies of smaller sub-watersheds.

The most commonly used statistical test to investigate the presence of trends in hydrometeorological time series is the nonparametric Mann–Kendall (MK) test. The nonparametric statistical tests, with parametric tests, are thought to be more suitable for non-normally distributed and censored data (censoring is a condition in which the value of a measurement or observation is only partially known i.e., because of for example missing data), which are frequently encountered in hydrometeorological time series. However, the scope of this study was to undertake a simple analysis of available flow data in an attempt to capture the influence of Saskatchewan drainage practice approaches, similar to those applied in the UARB Study.

The findings of the UARB study and Ehsanzadehab et al. appear to contradict one another. The UARB Study (Manitoba Conservation et al., 2000) found that runoff volumes have been increasing in the Upper Assiniboine River Basin. Whereas, Ehsanzadehab's research indicates the lack of a long-term trend for the annual flow of the Assiniboine River indicates that long-term cumulative changes in the watershed including wetland drainage and changes in farming practices have not had a net detectable influence on the flow. This could be because of the size of the two areas examined. It is likely the response of drainage practice may be more pronounced and detectable on small watersheds than larger watersheds. Impacts may diminish with increasing basin size because of other downstream influences.

21.4.2 Assiniboine River near Russell Trend Analysis

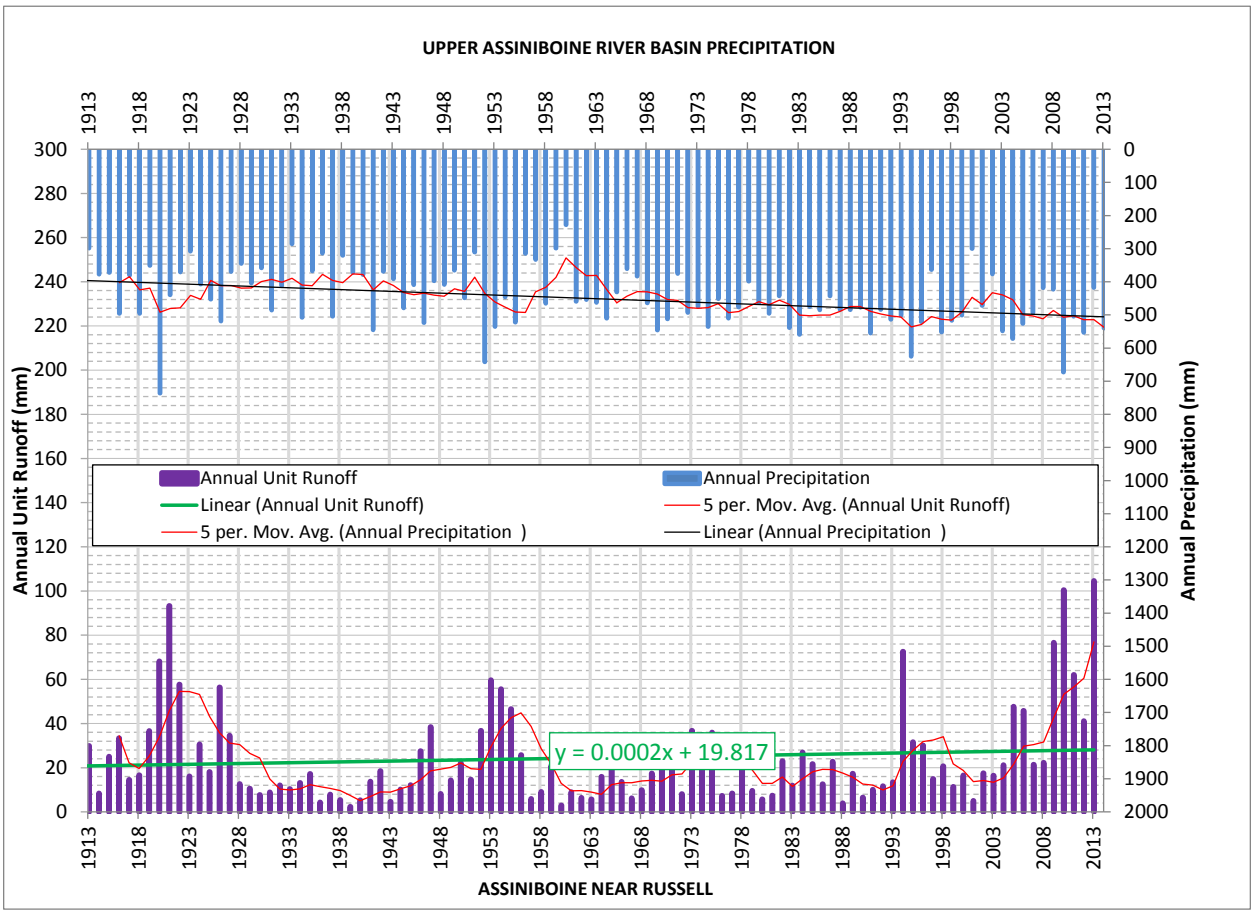
Annual streamflow records for the Assiniboine River near Russell dating back to 1913 for unregulated conditions (with the presence of Shellmouth Dam) were examined. The Upper Assiniboine River Basin has experienced a number of wet periods: in the 1920s, mid-1950s, mid-1990s and 2010.

The Assiniboine River near Russell exhibits an increasing trend (green trend line) in annual runoff over time as shown on Figure 21.1. Total annual precipitation in the basin also appears to be increasing as indicted by the black trend line on Figure 21.1. To normalize the data and to determine the impact of increasing basin precipitation, the unit runoff was divided by the basin's annual precipitation. A trend analysis was then undertaken on the normalized data. The normalized data did not exhibit any trend. Therefore based on this, it appears that the increased runoff on the Assiniboine River near Russell is due to increased basin precipitation.

A mass curve of the annual unit runoff indicates the influence of climatic conditions. Whenever there was an abnormally wet climatic period, there was a significant increase in the slope of the mass curve indicating significant increases in the annual runoff volumes. This was likely due to the triggering of certain thresholds, which govern the hill-slope runoff and wetland fill-spill processes and lead to significant changes in the areas contributing to runoff at the outlet of the watershed. This response appeared to be consistent throughout the period of record for the pre- and post-drainage periods.

These analyses, which indicate that the increased runoff is from increased basin precipitation, appear to support the Ehsanzadehab et al. (2012) findings that long-term cumulative changes in the watershed including wetland drainage and changes in farming practices have not had a net detectable influence on the flow. The increase is caused by climatic conditions.

Figure 21.1
Assiniboine River near Russell
Precipitation and Annual Run-Off Assessment



Note: To convert millimetres to inches, divide metric values by 25.4.

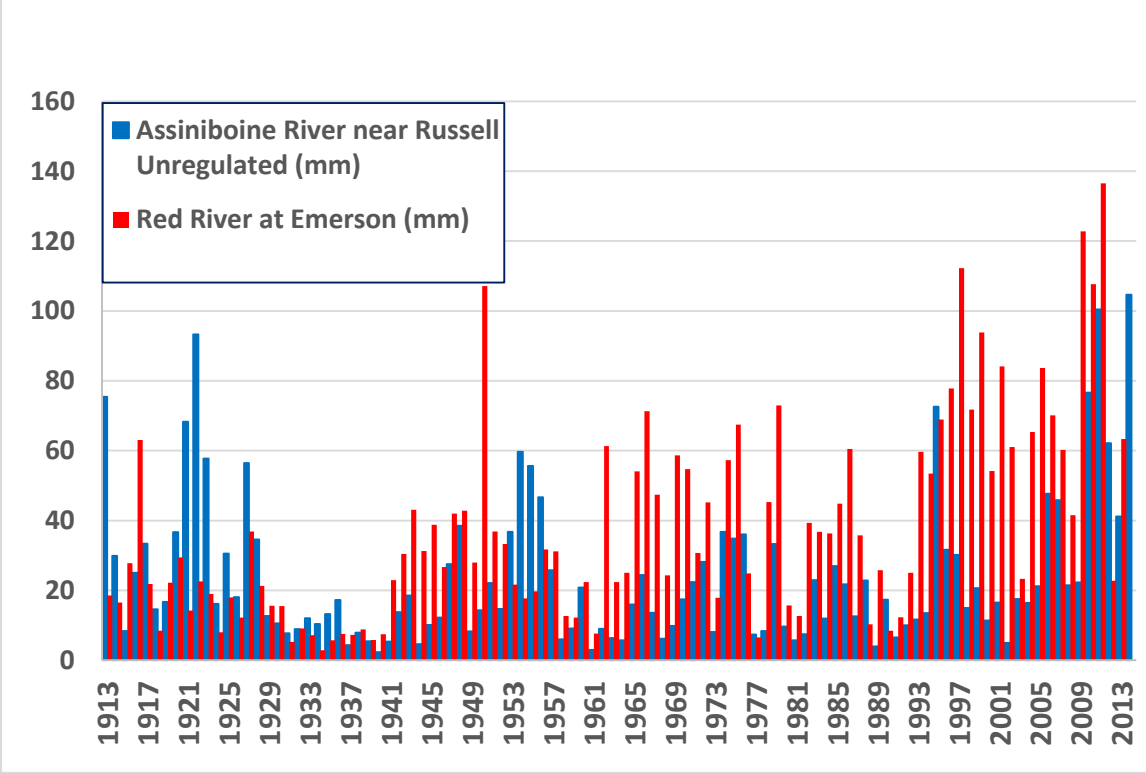
21.4.3 Direct Red and UARB Basin Comparison

The UARB was compared to the Red River Basin, which has had extensive drainage since its settlement in the late 1880s. The Red River Basin has little streamflow regulation. Both basins have streamflow records dating back to early in the decade beginning in 1910. The annual unit runoff for the Red River at Emerson was compared with the Assiniboine River near Russell annual unit runoff.

A mass curve comparison of the annual unit runoff indicated a greater stationarity in the Upper Assiniboine River Basin. This was unexpected as it was believed that drainage practices had

increased in the UARB overtime. It was concluded that it was not possible to directly compare two different basins having slightly different climates, soils, topography and land use to capture the influence of drainage practices. Generally, the Red River's annual unit runoff is about 60% greater than the UARB. This could be in part due to the clay soils in the Red River Basin, topography of the Red River Basin, which has a less spatial aggregation of wetlands than the Assiniboine River Basin, and the annual precipitation over the Red River basin, which on average is about 15% greater than the UARB (Figure 21.2).

Figure 21.2
Comparison of Annual Unit Runoff Red River and Upper Assiniboine River Basins



Note: To convert millimetres to inches, divide metric values by 25.4.

21.4.4 Shellmouth Dam Catchment Comparison

An analysis of the sub-catchments of Shellmouth Dam was undertaken to identify areas experiencing increased runoff response and to see if the increase could be quantified.

"The drainage area of Shellmouth Dam encompasses about 19,350 km^2, with about 79% in Saskatchewan and 21% in Manitoba. It is situated within two major physiographic regions – the Saskatchewan Plains and the Saskatchewan Uplands. The northern headwater reaches of the basin lie within the Boreal Plains Ecozone, an area characterized by groves of brush, wooded bluffs, steeper flow gradients and reasonably well defined drainage patterns. To the south, the lower and largest portion of the study area falls within the Prairie Ecozone, a much flatter terrain with less brush and fewer trees. During the retreat of the most recent glacier, some 10,000 years ago, a condition known as ablation moraine or dead-ice moraine predominated in this area of the prairie resulting in a hummocky landscape, an area pock-marked with a myriad of dimpled depressions, sloughs and potholes" (Manitoba Conservation et al., 2000). If drainage practices are having a noticeable effect on runoff volumes, it was felt that it should be evident in the portion of the watershed situated in the Prairie Ecozone.

To examine this hypothesis, the Shellmouth Dam gross drainage area was subdivided into six catchments, based on the available hydrometric station data. The six catchments have been described in Table 21.1.

Table 21.1
Shellmouth Dam Catchments

Catchment Name	Description	Gross Drainage Area Km^2
Shell River	Used Shell River near Inglis	1967.7
Shellmouth Local	Equal to Shellmouth Dam inflow minus Shell River near Inglis and Assiniboine River near Kamsack	4328.6
Kamsack Local	Equal to Assiniboine River near Kamsack minus Whitesand River near Canora and Assiniboine River near Sturgis	2287.4
Upper Assiniboine Local	Used Assiniboine River near Sturgis	1939.4
Whitesand Local	Equal to Whitesand River near Canora minus Yorkton Creek near Ebenezer	6510.9
Yorkton Creek	Used Yorkton Creek near Ebenezer	2317.0

The annual percentage contribution of each catchment was computed for two periods:

1. 1970 to 2014, which corresponds with the dam's operation.

2. 1950 to 2014, which may predate drainage in the basin and includes the wet period of the 1950's.

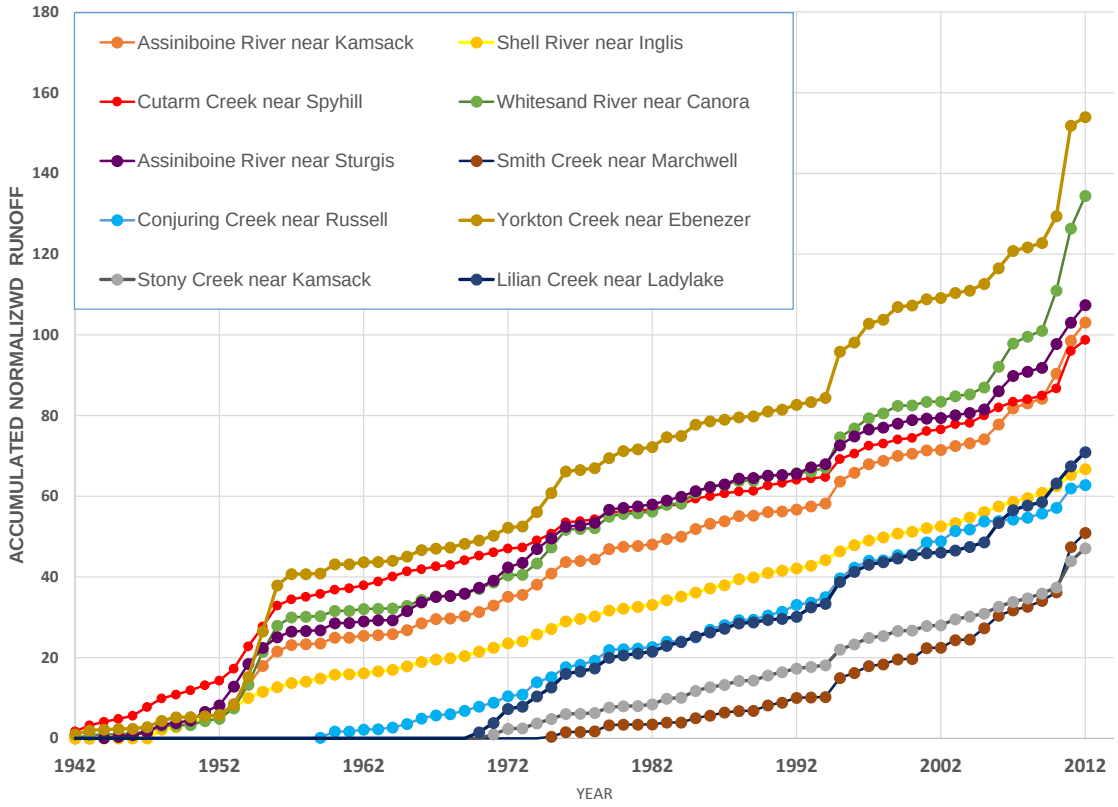
The approach attempted to minimize climate variation by using a single basin and normalizing the data by using an annual percentage contribution. The analysis indicated the catchments exhibited considerable annual variability. A trend analysis for the shorter period indicates that the three catchments associated in the Boreal Plains Ecozone: the Shell River, Upper Assiniboine River and Kamsack were contributing a lesser percentage of the inflow to Shellmouth Dam over time; whereas, the Whitesand Local catchment, which is entirely in the Prairie Ecozone where wetland drainage is prevalent, is contributing a higher proportion of the total inflow over time. When the trend analysis was extended back to the 1950s to capture the wet period of the 1950s, similar trends were observed. The exception was the Shellmouth Local, which now has a declining trend over time. The Whitesand Local still exhibited increasing runoff over time. A limitation of this analysis was that it only indicated a relative trend with time. Also, there were other land use practices in the basin such as the clearing of trees and vegetation, development of road infrastructure, and alterations to land use that impacted the hydrologic response of a basin.

21.4.5 Mass Curve Analysis of Annual Peak Flows and Volumes

The existing hydrometric streamflow data on the tributaries in the Upper Assiniboine River Basin was analyzed using a mass curve analysis to determine if the impact of drainage practices in the basin could be detected on annual runoff peaks and open water runoff volumes. Previously in the UARB Study, spring flood peaks and runoff volumes were analyzed; however, as summer rainfall events have now become prominent, examples include the 2010 and 2014 summer floods, open water period values were examined. Most streamflow gauging is only operated for the open water period.

The annual peak flows and runoff volumes were normalized by dividing the volume by its median value before accumulating the values for the period of record for each station. The mass curves for the open water runoff volumes and annual peak flows are shown on Figures 21.3 and 21.4, respectively.

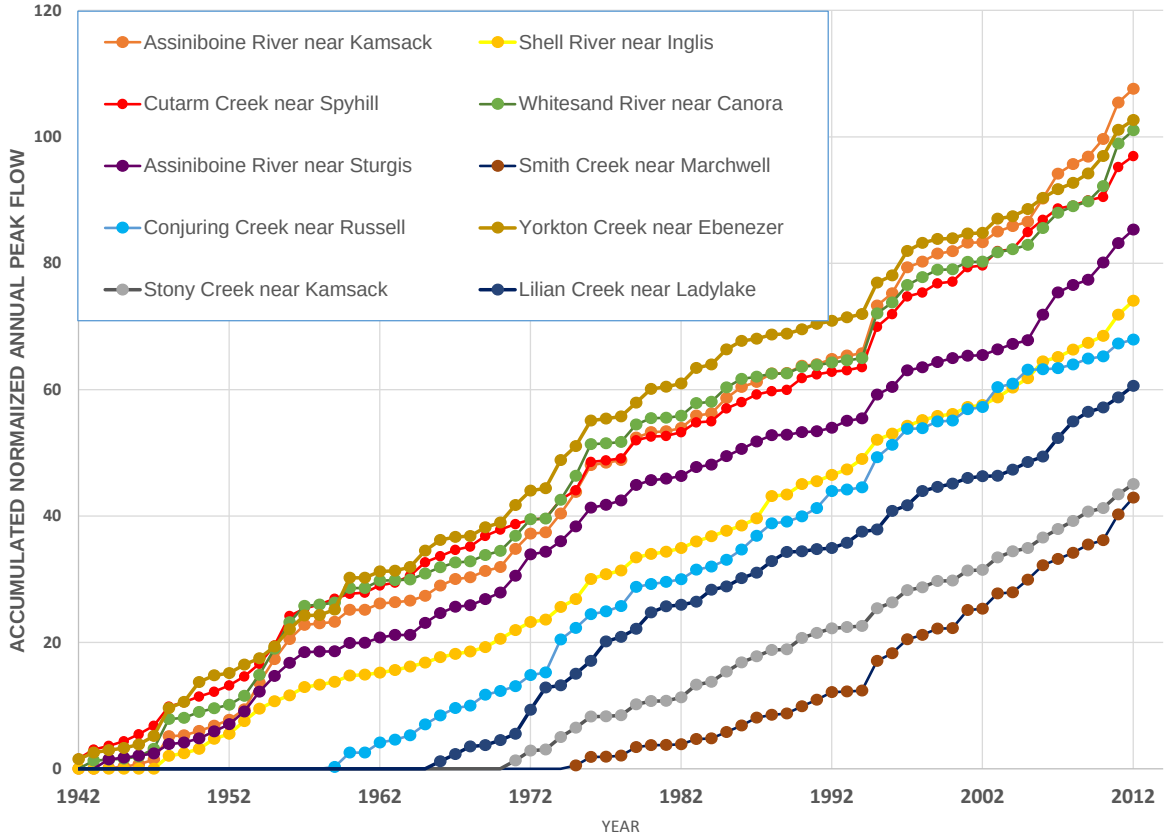
Figure 21.3
Mass Curve of Normalized Open Water Runoff Volumes



For stations with long-term records, such as Yorkton Creek, Cutarm Creek, the Whitesand River, and the Assiniboine River near Sturgis, the mass curves indicated increased runoff responses in the mid-1950s similar to those occurring early in the decade beginning with 2010. The1950s, a wet period, predates the period of major drainage activity in the basin. The Shell River exhibited a constant trend over time, indicating little or no change in runoff response to watershed change and a high degree of stationarity. The Shell River is in the Boreal Plains Ecozone and had only about 17% cropland cover. Conjuring Creek, adjacent to the Shell River in Manitoba, exhibited a similar trend in runoff response and stationarity. Smith Creek appeared to exhibit the greatest increase in runoff response over time and the highest degree of non-stationarity.

The mass curves of annual peak flow showed similar trends but much greater stationarity than for the mass curves of runoff volumes. The Shell River still showed a high degree of stationarity.

Figure 21.4
Mass Curve of Normalized Annual Peak Flow



21.4.6 Direct Watershed Comparison

With the high degree of stationarity exhibited by the Shell River, a paired watershed comparison was considered in an attempt to quantify the impacts of drainage practices in the UARB. The Shell River, being mainly in the Boreal Plains Ecozone and exhibiting a high degree of stationarity, was considered to have experienced minimal drainage activity and was considered the baseline condition. It was compared to the Smith Creek and the Whitesand River watersheds, both located in the Prairie Ecozone. The Smith Creek Watershed has been extensively drained.

The following methodology was developed and applied. First, the annual unit runoff volume and peak flow was computed for each watershed by dividing the respective hydrometric stations that recorded annual peak flow and open water runoff volume by its gross drainage area. The ratio

of the peak flow and the runoff volume for Smith Creek and the Whitesand River to the Shell River, the baseline watershed, was calculated for each year. As the annual runoff response was largely determined by the climatic conditions using a direct comparison of streamflow stations in the same climatic area, it was felt this issue should be addressed, and any variations because of climate, should be minimal. The annual ratios of peak flow and the runoff volume for Smith Creek and the Whitesand River were plotted against time. The plots exhibited considerable scatter. It was felt that the dry years could be responsible for the large scatter. As the primary concern in the UARB was flooding, it was decided to plot only ratios of annual peak flows and runoff volumes for those years greater than median value for the Shell River. Plotting only the ratios of annual peak flows and runoff volumes reduced the scatter. Trend lines were superimposed on the plots with the values greater than the median. The trend plot for Smith Creek shown on Figure 21.5 indicated a minor increase in peak flows and runoff volumes over time. This was surprising considering the extensive drainage that has occurred in the Smith Creek watershed. This increasing trend for Smith Creek was supported by the mass curves of the unit ratio of peak flows and runoff volumes shown on Figure 21.6.

Figure 21.5
Paired Watershed Comparison Smith Creek vs. Shell River

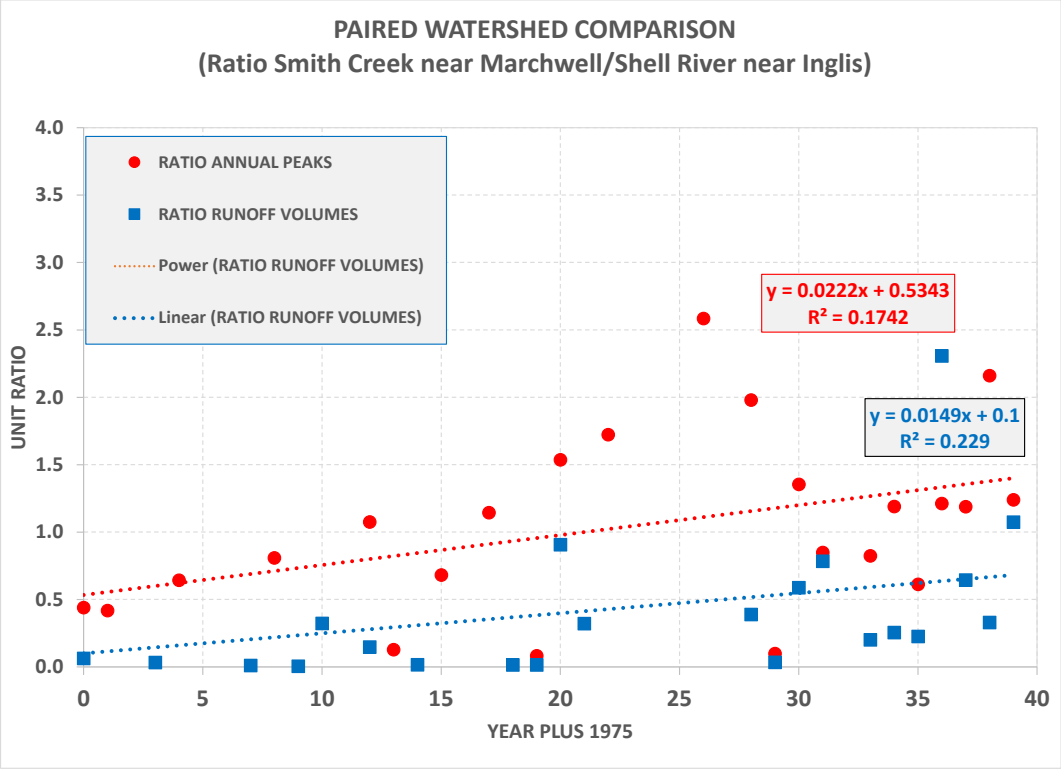
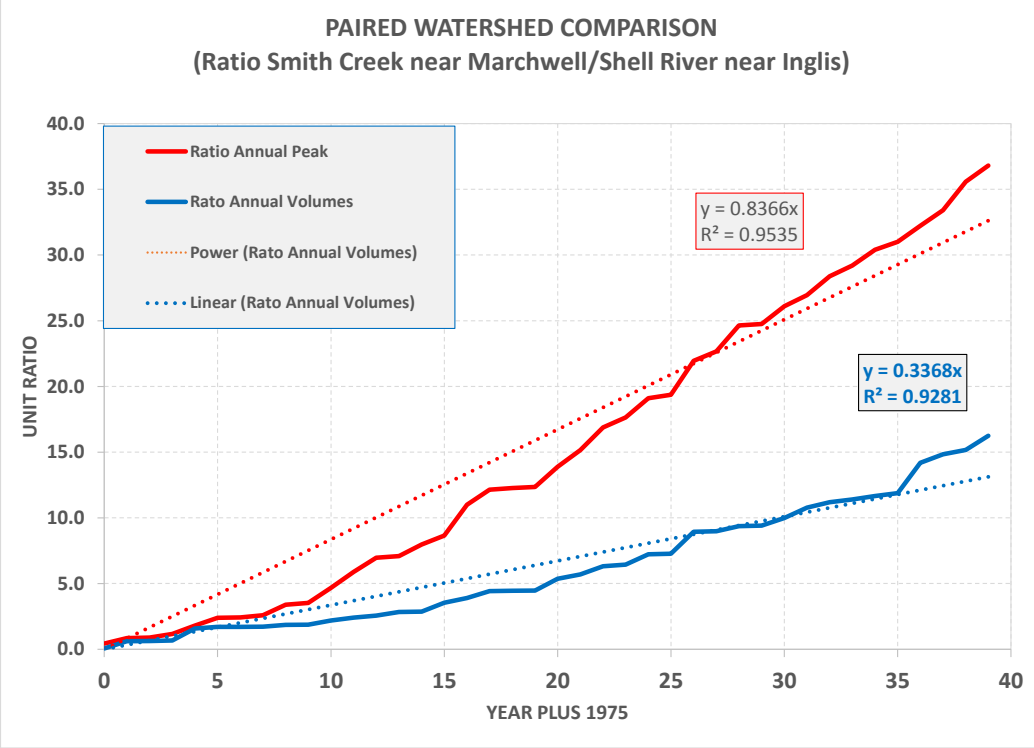


Figure 21.6
Mass Curve of Accumulated Unit Peak Flow and Runoff Volume for Smith Creek and the Shell River



A time series of hydrologic data was said to be stationary if the frequency distribution of the data remained constant over time. To test for the degree of stationarity in the peak flow and volume data, a linear trend analysis was applied to the mass curves for each. This was based on the hypothesis that the degree of linearity indicated the stationarity of the data, which in turn reflects the net impact of changing agricultural practices. From the Handbook of Applied Hydrology, *"If there is no linear relationship, the r (correlation coefficient) = 0. If there is a functional linear relationship, r = ±1. All values of r between these limits describe the various degrees of correlative association. The greater the absolute value of r, the greater the linear correlation"* (Chow, 1964). Therefore, the correlation coefficient was used to provide an indication of stationarity of the peak flows and volumes. The greater the correlation coefficient, the greater is the stationarity (Manitoba Conservation et al., 2000). For Smith Creek, the correlation coefficients for the annual peak flows and runoff volumes were 0.953 and 0.928, respectively, indicating some non-stationarity of the data. Also, the mass curves had an increasing slope indicating a greater runoff response over time from Smith Creek than the Shell River. This likely was as a result of increased drainage practices in the Smith Creek watershed.

The trend plots and mass curves for the Whitesand River exhibit the same non-stationarity but to a lesser degree (Figures 21.7 and 21.8).

The correlation coefficients for the annual peak flows and the runoff volumes were 0.942 and 0.963, respectively. The mass curves exhibited an increasing slope indicating a greater runoff response over time from the Whitesand River than the Shell River, but less than Smith Creek. The analysis supported that there was an increasing runoff response from the UARB, but the signal was not strong. Drainage practices were likely the major contributing factor.

Figure 21.7
Paired Watershed Comparison Whitesand River vs. Shell River

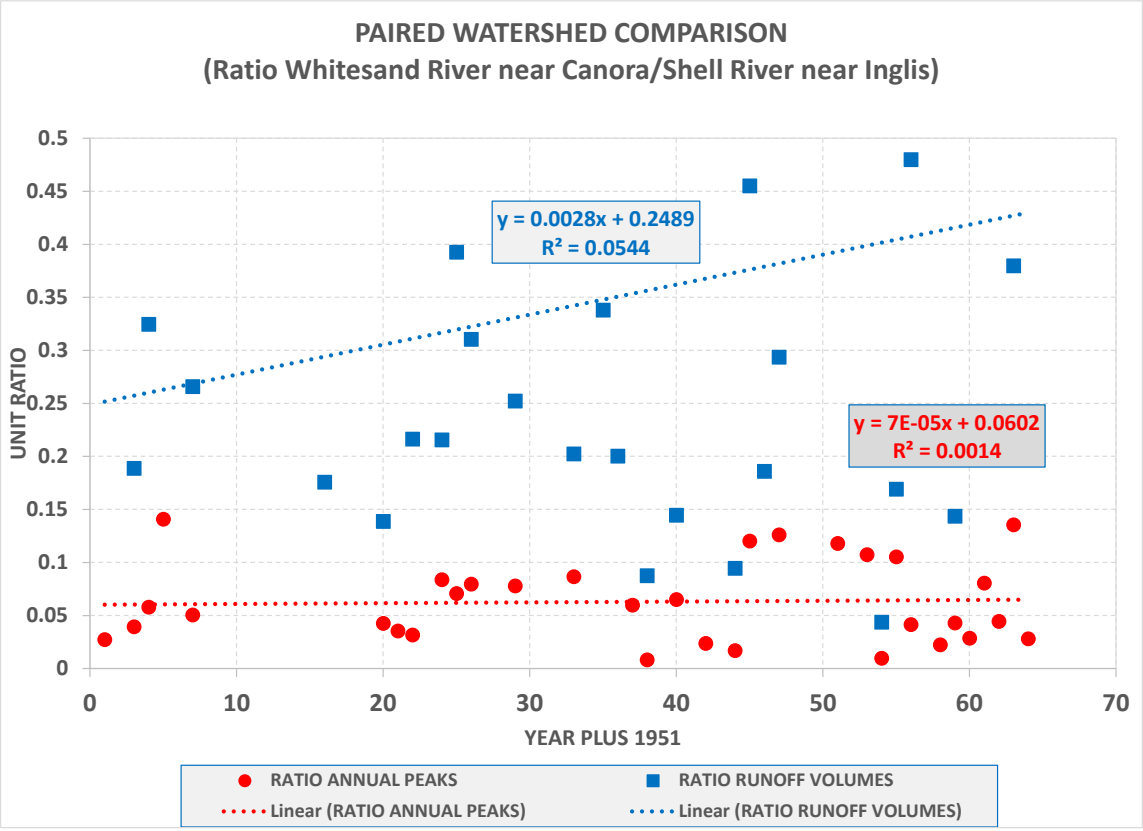
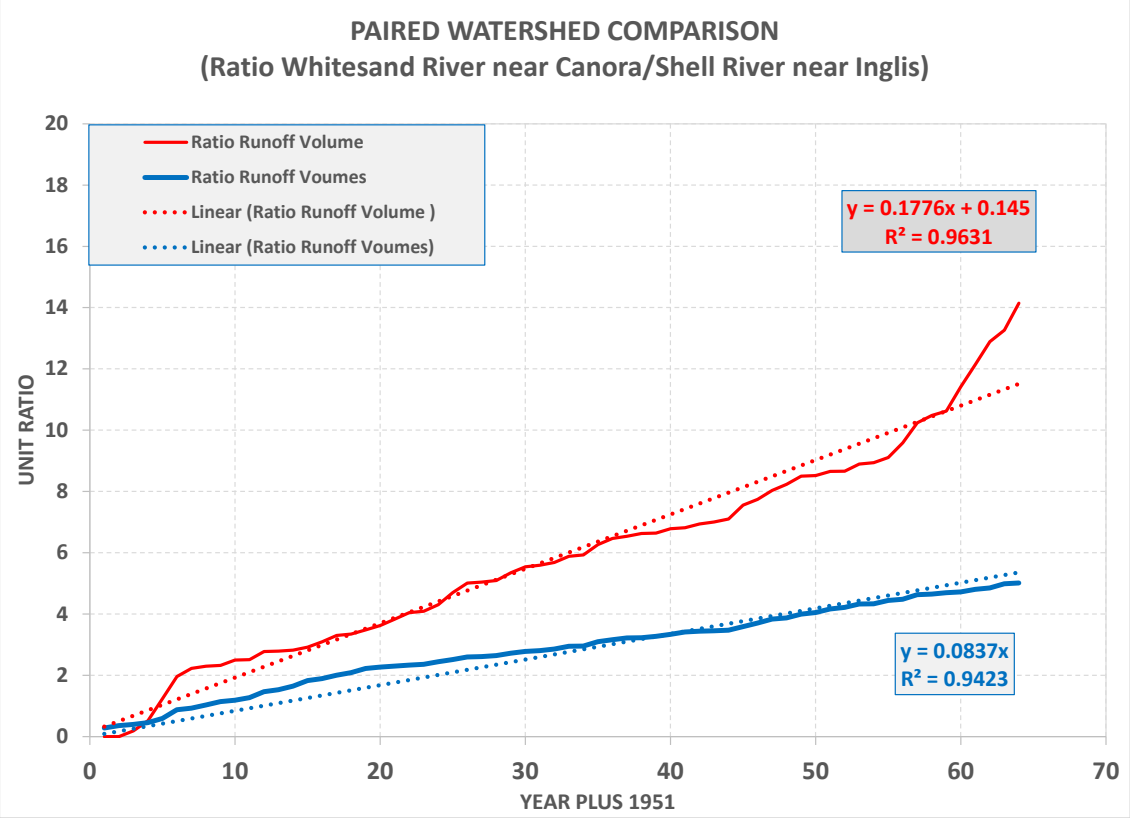


Figure 21.8
Mass Curve of Accumulated Unit Peak Flow and Runoff Volume for Whitesand River vs. the Shell River



21.4.7 Conclusions

The research on the impact of drainage practices on streamflow was inconclusive. The UARB Study (Manitoba Conservation et al., 2000) indicated runoff volumes have been increasing in the Upper Assiniboine River Basin, whereas, Ehsanzadehab's (Ehsanzadeh et al., 2012) research indicated the lack of a long-term trend of the annual flow of the Assiniboine River, which indicated that long-term cumulative changes in the watershed including wetland drainage and changes in farming practices have not had a net detectable influence on the flow.

An analysis of streamflow data identified increasing trends in watershed response in the UARB. Hydrologic analysis of streamflow records in the Upper Assiniboine River Basin indicated that both peak flows and runoff volumes were increasing. However, the trend was weak. The difficulty was isolating and quantifying the impacts of drainage practices from other land

management practices. Changes in drainage practices were likely the predominate factor impacting the increased runoff response from the watershed. The drainage practices generally resulted in a gradual change and not an abrupt change in the hydrologic response of a watershed such as the construction of a major water reservoir. This made the detection and isolation of changing drainage practices extremely difficult from the direct analysis of streamflow data. As illustrated by the direct comparison of the Shell River to Smith Creek and the Whitesand River even with a paired watershed approach, the impacts of drainage practices could not be quantified. Ideally, paired watersheds with the same land use practices, but one in a drained state and the other in an undrained state, would be required.

21.5 CONTROL OF AGRICULTURAL DRAINAGE ACTIVITIES IN SASKATCHEWAN

Water is a natural resource, and its management is a provincial responsibility. Saskatchewan has legislation to deal with drainage under the *Drainage Control Act*. Some Manitobans contend that Saskatchewan is ineffective in managing the Act, and unauthorized man-made drainage in Saskatchewan is worsening their flooding. As experienced by Manitoba, effective management of drainage licensing and enforcement takes considerable resources. The *Saskatchewan Drainage Act* appears to be complaint based. Some Manitobans are, however, frustrated as unauthorized drainage appears to be causing drainage problems in another jurisdiction; and they are unable a file a complaint directly. As such, these issues must be dealt with through the Manitoba Provincial Government, which must deal with it at the political level and with inter-jurisdictional agencies. To date, this process has not resulted in a successful resolution of this issue.

Saskatchewan now appears willing to deal with the issue of unauthorized drainage. The 25-Year Saskatchewan Water Security Plan identified drainage as one of the key areas of action (SWSA, 2012; Briere, 2012).

The plan is to provide a sustainable approach to water use that will protect the quality and quantity of water now and for the future. Action Area 5.2 (Water Security Plan), on agricultural drainage and flooding, identifies three actions:

1. Assess the range of alternatives and implement strategic actions to manage drainage (2013).

2. Develop a results-based drainage works approval process and associated enforcement strategy including the potential use of financial penalties (2014).

3. Develop new strategies to effectively address excessive moisture concerns on agricultural lands including provision of information and advice on proper drainage design and management and consideration of the benefits of wetland retention and restoration (2014).

Currently, the Saskatchewan Water Security Agency is supportive of these actions. Its on-line forum with 491 participants, in person survey of 8 individuals, and a quantitative survey of 480 individuals provided an opportunity to discuss unauthorized drainage, impacts and options (SWSA, 2014). The final report documenting feedback was released in November 2014. Forum participants agreed that drainage provided many benefits to agricultural producers, but that the negative impacts of drainage must be mitigated. Also, forum participants supported implementing regulations that were risk-based; those drainage activities on a larger scale, which pose a greater risk to land and property, would require more stringent regulations. The full report can be viewed at the web address provided in the Reference Section of this report. This activity and some other processes will assist the Water Security Agency in the development of a more proactive approach to drainage and wetland loss. New legislation and drainage regulations will be part of this process.

Through the Assiniboine River Basin Initiative, a body of stakeholders who have an interest in working cooperatively to address water related issues across the Qu'Appelle, Souris, and Assiniboine River basins, Manitobans will have opportunities to express their concerns to all jurisdictions regarding the drainage issue.

21.6 CONCLUSIONS

Interprovincial drainage has been a long standing and very contentious issue between Manitoba and Saskatchewan, especially in the Upper Assiniboine River Basin. There is an apparent frustration by Manitoba farmers, as they cannot register a drainage complaint directly against Saskatchewan drainage. All complaints must be handled by interprovincial negotiations.

In the past number of years, Manitoba and Saskatchewan have cooperatively tried to resolve interprovincial drainage issues. On several occasions, the issues have been referred to the Prairie Provinces Water Board (PPWB). However, no acceptable resolution has yet been

reached through this forum, as the PPWB does not have the mandate to resolve interprovincial water management issues. There is a need for a forum to resolve interprovincial water management issues such as drainage. The United States and Canada have the International Joint Commission to which they refer international disputes for resolution. Possibilities are to expand the mandate of the PPWB or the Western Water Stewardship Council, or to establish some type of an interprovincial commission or dispute board.

A resolution of the interprovincial drainage issue has been restricted by the lack of clear scientific understanding of the role prairie wetlands play on flood attenuation for varying flood magnitudes. Pomeroy's 2014 findings for Smith Creek helped to advance the scientific understanding of the role that wetlands played in flood attenuation. In this study, an analysis of streamflow data identified increasing trends in watershed response in the UARB. Hydrologic analysis of streamflow records in the Upper Assiniboine River Basin indicated that both peak flows and runoff volumes appeared to be increasing. The difficulty was isolating and quantifying the impact of drainage practices from other land management practices. Changes in drainage practices were likely the predominate issue impacting the increasing runoff response of the watershed. Drainage practices generally resulted in gradual change, and as such, were not easily quantifiable and isolatable through the analysis of streamflow data. As illustrated by the direct comparison of the Shell River to Smith Creek and the Whitesand River, even with a paired watershed approach identifying increasing trends, the impacts of drainage practices could not be quantified. Ideally, it could take a paired watershed assessment with the same land use practices, but with different drainage practices: one in a drained state, and the other in an undrained state, to provide more conclusive data.

Saskatchewan has recognized the problems with illegal drainage. However, there appears to be no immediate solution to the problem. Through the actions of the 25-Year Saskatchewan Water Security Plan, the issue of illegal Saskatchewan drainage could be addressed. Saskatchewan was proposing new legislation to address the illegal drainage activities that also impact Manitoba. Development and implementation of new legislation along with enforcement, however, will take time. The flood damages from these drainage activities will continue to affect Manitobans in the meantime. There is an immediate need for a wetland preservation strategy on drainage in the entire Assiniboine & Lake Manitoba Basins to limit future flood damages. Through the Assiniboine River Basin Initiative, a body of stakeholders who have an interest in

working cooperatively to address water related issues across the Qu'Appelle, Souris, and Assiniboine River basins, Manitobans will have opportunities to express their concerns to all jurisdictions regarding the drainage issue.

22.0 EVALUATION OF OPTIONS FOR UPDATING THE FLOOD PROTECTION STUDY AREAS

22.1 BASE ANALYSIS OF INDIVIDUAL OPTIONS

Section 4.2 of this report summarizes the standards that are used by the Province of Manitoba to determine the extent of flood protection that may be justified for infrastructure in each jurisdiction. The study's interpretation of those criteria is repeated below in summary form:

- Protection against the 1:200 year flood or the flood of record, whichever is greater for that location.
- If the existing protection is less than the criteria above, enhancements to upgrade that protection may exceed those criteria, if justified by economic analyses and benefit / cost ratios that exceed 1.0.

It should be noted that the criteria above have only recently included the 1:200 year flood magnitude, and previously included only a 1:100 year flood threshold. As a result, there are some communities for which the Province has achieved or is in the process of achieving the protection against the flood of record, but that fall modestly short of the 1:200 year flood.

Section 6.0 and Sections 8.0 through 17.0 focus on the documentation of the development and analyses of a wide spectrum of individual options for enhanced flood protection in the region.

Section 7.0 summarizes the economic guidelines that have been agreed with MIT to be appropriate for this study. These form the basis of assessing the enhancements in flood protection to reveal whether economic justification (second bullet above) would govern the level of protection that should be adopted for future implementation. The application of these guidelines in many cases required deployment of numerical models of damage potential with and without enhancements in flood protection measures. These damage models are explained in further detail in Section 8.6.

Table 22.1 summarizes the results of the economic analyses for the most prominent options for flood protection in the region. Detailed backup and elaboration of the calculations contributing to Table 22.1 are contained in Appendix O and include the following components:

- Summary table of benefits, costs and B/C ratios of all the individual options.

- Detailed calculation sheets of the economic analysis for each individual option.
- Summary table of benefits, costs and B/C ratios for combinations of options.
- Detailed calculation sheets of the economic analysis for combinations of options.
- Summary table of results of the sensitivity analyses to changes to period of record.
- Detailed calculation sheets of the sensitivity analysis to changes to period of record.
- Summary table of results of the sensitivity analyses to changes to economic parameters.
- Detailed calculation sheets of the sensitivity analysis to changes to economic parameters.
- Frequency damage curves for individual options and combinations of options.

It became clear after completing the analyses of the options summarized in Table 22.1 that application of the economic guidelines did not lead to the justification of flood protection for infrastructure in the study area that exceeded the basic requirements of either the 1:200 year flood or the flood of record. Even combinations of protection measures did not result in the economic justification to provide protection for floods exceeding the basic requirements. This finding was applicable to all options and combinations of options devised in this study.

It should be noted that the option of purchasing the flood prone lands along the Upper Assiniboine River was calculated to equal 1.0. This option is, however, for a proposed flood protection level well below the 1:200 year criteria. An evaluation of the option has shown that the B/C ratio would be less than 1.0 if the land purchases were for a flood protection level that exceeded the 1:200 year criteria.

Table 22.1
Summary of Results of the Economic Analyses of Flood Mitigation Options for the Assiniboine River & Lake Manitoba Basins

Flood Mitigation Options	Capacities Considered	B/C Ratio
Diking along Upper Assiniboine River – 2 options	1:5 and 1:50 year events	0.05 to 0.27
Purchase of Flood Prone Lands along Upper Assiniboine River	1:10 year event	1.0
Large Storage Reservoir – 6 options	40,000 dam^3 to 1,600,000 dam^3	0.40 to 0.61
Diking in Souris, Melita, and Wawanesa – 3 options	1:200 and 1:500 year events	0.16 to 0.50
Upgrades to Lower Assiniboine River Dikes – 3 options	18,000 cfs to 35,000 cfs	0.02 to 0.21
Upgrades downstream of Lower Assiniboine Dikes – 2 Options	18,000 cfs to 35,000 cfs	0.01 to 0.16
Assiniboine River Diversion at Hoop and Holler – 3 options	3,900 cfs	0.14 to 0.52
Portage Diversion – 4 options	34,000 cfs and 46,000 cfs	0.32 to 0.68
Diversion Channels on Dauphin Lake – 4 options	6,600 cfs to 7,520 cfs	0.07 to 0.15
Mossy River Excavation – 3 options	Varies	0.24 to 0.38
Lake Winnipegosis Diversion - 2 options	2,500 cfs to 10,000 cfs	0.08 to 0.39
Lake Manitoba & Lake St. Martin Outlet Channels – 6 options	2,500 cfs to 15,000 cfs	0.31 to 0.50
Multitude of 21 Small Dams	200,000 dam^3	0.24
Combination of Projects – 15 combinations that include Large Storage Reservoirs, Upgrades to Lower Assiniboine River, Upgrades to Portage Diversion, and Lake Manitoba / Lake St. Martin Outlet Channels	Varies	0.27 to 0.47

This conclusion provided a clear direction for the remainder of the study – to identify the best measures that could upgrade the level of flood protection for infrastructure, where necessary, to at least the 1:200 year flood, or the flood of record, whichever is greater, at all locations within the study area. Figure 22.1 (this is a repetition of Figure 4.1 discussed in Section 4.2) shows the locations of flood-prone infrastructure and the current level of protection. For example, the dikes that protect the community of Melita on the Souris River are currently capable of protecting against the flood of record (2011 Flood), but are not able to achieve a level of protection against

a 1:200 year flood. Economic assessment of the options for upgrading that protection to as much as a 1:500 year flood has shown that the costs would far exceed the expected benefits, based on the criteria applied.

Figure 22.1
Flood Risk Zones in Study Area

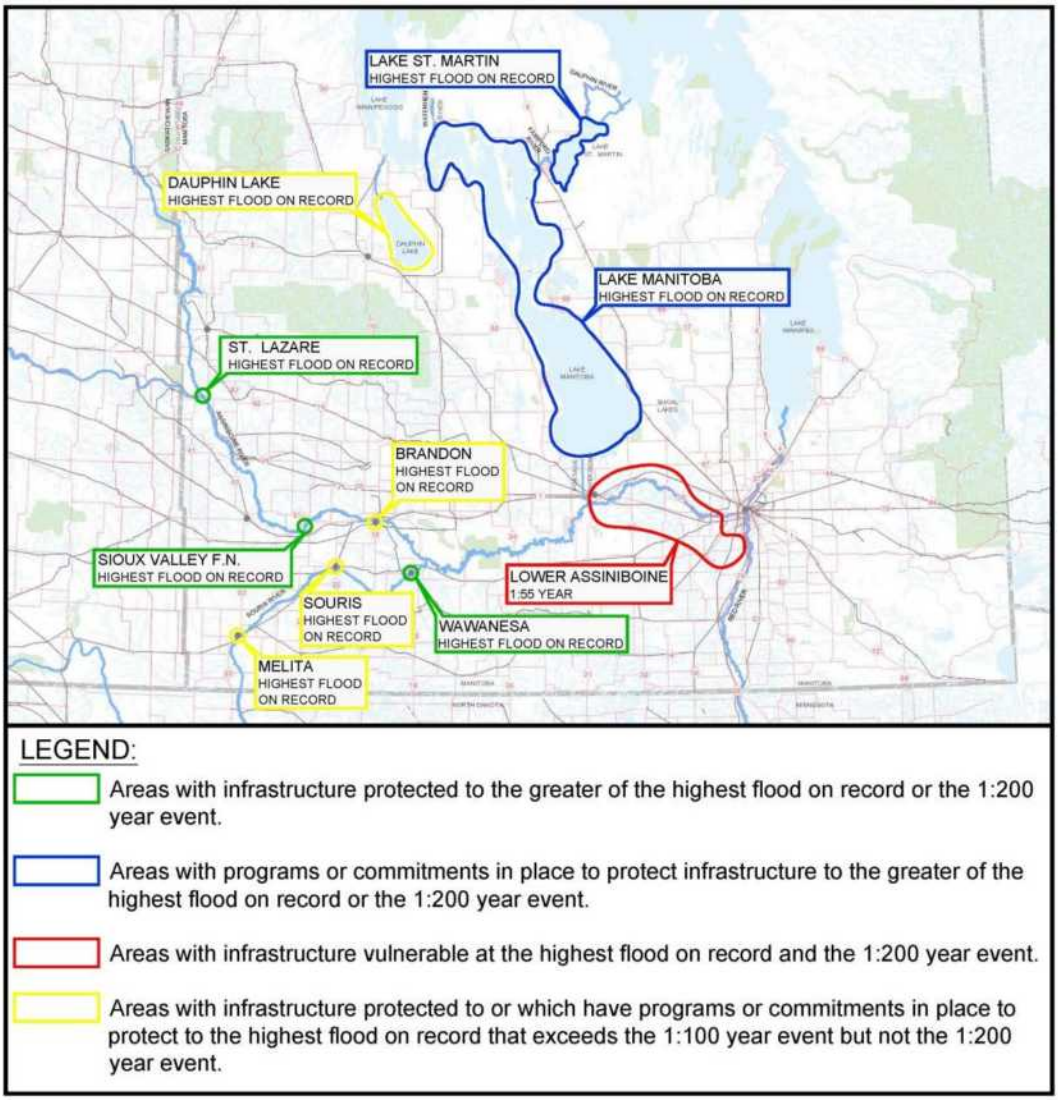


Figure 22.1 shows that the only zone within the study area that does not meet either the 1:200 year flood criterion or the flood of record, is the Assiniboine River east of the Portage Diversion and upstream of Winnipeg. One zone where it would seem counter-intuitive to accept this conclusion is Lake Manitoba and Lake St. Martin. Both of these areas had significant

damages during the 2011 Flood, and upgrading of protection (whether by physical works or the purchase of flood prone lands) would seem a logical need. However, it is important to realize that since the 2011 Flood there have been significant strides towards upgrading flood protection including the following initiatives:

- Individual Flood Proofing Initiatives.
- Financial Assistance for Cottage Owners.
- Community Flood Protection Program.
- Flood 2011 – Building and Recovery Action.

As a result of these comprehensive and in some cases, on-going programs, the levels of flood protection around the shorelines of Lake Manitoba and Lake St. Martin are moving towards achieving the provincial standards summarized previously. Consequently, that zone is depicted in Figure 22.1 as having adequate flood protection exceeding both the 1:200 year flood and the flood of record, in accordance with the provincial standards. It is recognized, however, that the number of stakeholders who have participated in these programs to undertake mitigation measures has been less than originally anticipated. Further discussion on the impacts of these programs on the results of the economic analysis is included in Section 8.6.2.8.

On the basis of the conditions shown in Figure 22.1, the remainder of the study process was consequently focused on identifying the best protection measures for the Lower Assiniboine River, or combinations of measures, that could upgrade the level of protection in that area to meet provincial standards. The estimated current level of protection is only approximately a 1:55 year flood (equal to a flow of 1,133 m^3/s (40,000 cfs) upstream of Portage) based on the long-term flood statistics for the Assiniboine River.

Review of stand-alone options to upgrade protection in this area showed that no single measure could achieve the desired standard, within the limits of practicality. For example, enhancement of the dikes along the Lower Assiniboine River would have to attain a discharge capacity of 900 m^3/s (32,000 cfs) to single-handedly raise the level of protection in the area to meet the provincial standard. This enhancement would be extremely costly, as discussed in Section 10.1, and would exceed the practical limit of river flow that could be tolerated within the City of Winnipeg without significant investment in enhanced flood protection works in that jurisdiction.

There was no clear definition of the existing allowable tolerance for flow in the Assiniboine River within the City of Winnipeg, but it was expected to be at least 750 m^3/s (26,500 cfs), and possibly in excess of 800 m^3/s (28,250 cfs).

It became apparent that combinations of measures would be the most logical and practical means to achieve the desired upgrades. The preferred combination included:

- Increased discharge capacity of the Portage Diversion.
- Increased outflow capacity of Lake Manitoba and Lake St. Martin.
- Enhanced protection and increased safe discharge capacity along the Lower Assiniboine River.

Section 23.0 addresses the combination of measures for the Lower Assiniboine River.

22.2 SENSITIVITY ANALYSES

22.2.1 Variation in Assumed Economic Parameters

Application of the economic guidelines summarized in Section 7.0 could have varying results depending on the parameters assumed. A variety of sensitivity analyses were undertaken to examine what effects could result from variation in the following:

- Interest rates – varied from 5% to 9% per year.
- Inflation rates – varied from 2% to 4% per year.
- Growth – zero to 1% per year.
- Economic life span, varied from 40 to 75 years.

The analyses were focused on the preferred alternatives identified in Section 22.1. Details of the various analyses are provided in Appendix O. The results were consistently supportive of the conclusion that the economics of the options studied do not justify increasing the level of protection in this region to more than the basic standards summarized in Section 22.1.

22.2.2 Sensitivity to Assumed Trends in River Flood Magnitudes

Application of the provincial standards of flood protection must be based on a clear understanding of the flood magnitudes associated with the 1:200 year criterion. The traditional approach in engineering is to adopt the full historical period of record of stream flows in a statistical analysis that leads to the quantification of the 1:200 year flood. For the Assiniboine River, for example, this is a substantial record, extending back into the early 1900s.

There are indications, however, that there may be a trend towards more frequent and greater flood magnitudes, caused possibly by a combination of climate change, changes in land use, and development of highly effective drainage works in the watershed. But it is difficult to prove that as being distinct from statistical variations that are simply due to chance. Based on the feedback received by the experts assembled at the Workshop described in Section 4.1, it was agreed that the traditional standard based on the full history of stream flows would be adopted for the current study.

However, it was also agreed that three representative options of flood protection identified in this study would be selected and subject those options to a range of sensitivity analyses. These analyses would be based on alternative flood frequency curves based on short flow records that were indicative of only the recent decades when floods seemed to be most frequent and of the largest magnitude.

The three selected options were:

1. Upgrades to the Melita Community Dikes.
2. The Holland Dam.
3. The Portage Diversion 963 m^3/s (34,000 cfs) Retrofit Option.

Based on the frequency curves using the short period of record provided in Section 5.2.3 (the Souris River in Melita and the Assiniboine River in Holland), the three options identified above were evaluated; and the results were compared to those based on the frequency curves that used the long period of record. The analysis revealed that:

- Use of only the historical record for recent decades of flood peaks would tend to increase the best estimate of the 1:200 year flood in most areas. For example and as summarized in Table 22.2, the 200-year flood in Holland increased from 1,617 m^3/s (57,100 cfs) to 2,491 m^3/s (88,000 cfs). In Melita, it increased from 790 m^3/s (27,900 cfs) to 985 m^3/s (34,800 cfs). The increase would vary in other locations of the study area.

Table 22.2
Effects of Period of Record to the Estimated 1:200 Year Flood Discharge

Location	Long Period of Record (1882-2011)	Short Period of Record (1971-2011)
Assiniboine River near Holland	1,617 m^3/s (57,100 cfs)	2,491 m^3/s (88,000 cfs)
Souris River in Melita	790 m^3/s (27,900 cfs)	985 m^3/s (34,800 cfs)

- Estimates of damages averted by any specific improvement in flood protection would increase on an average annual basis and enhance the economic incentive to proceed. For example, the average annual benefits in the Lower Assiniboine River of 963 m^3/s (34,000 cfs) for the Portage Diversion increased from approximately $6.2 million/year to $15.1 million / year, which results in an increase of the B/C ratio from 0.7 to 1.8. This is summarized in Table 22.3. The results for Holland Dam and for the Melita Dikes are also summarized in the Table.

Table 22.3
Effects of Period of Record on the
Average Annual Benefit Calculations and B/C Ratios

Flood Mitigation Option	Long Period of Record (1882-2011)		Short Period of Record (1971-2011)	
	Average Annual Benefits	B/C Ratio	Average Annual Benefits	B/C Ratio
963 m^3/s (34,0000 cfs) Portage Diversion	$ 6.4 Million	0.77	$ 15.6 Million	1.82
888,000 dam^3 (710,000 ac-ft) Holland Dam	$ 5.1 Million	0.49	$ 12.6 Million	1.0
790 m^3/s (27,900 cfs) Melita Dikes	$ 1,400	0.16	$ 2,100	0.25

- On the other hand, achieving the minimum protection of a 1:200 year flood would require more extensive protection works and would become more expensive. For example, a 1,303 m^3/s (46,000 cfs) channel capacity for the Portage Diversion has been estimated to cost approximately $540 million. This is $170 million more than a 963 m^3/s (34,000 cfs) channel capacity for the Portage Diversion. This 340 m^3/s (12,000 cfs) increase in capacity was well short of achieving the 875 m^3/s (30,900 cfs) increase to the 1:200 year flood from using the short period of record for the frequency analyses instead of the long period of record. The cost of constructing a 1,838 m^3/s (64,900 cfs) Portage

Diversion (i.e., 875 m^3/s (30,900 cfs) increase to the 963 m^3/s (34,000 cfs) Portage Diversion capacity) to achieve the 1:200 year protection could be in excess of $1 billion . With the added costs of upgrades to the Lower Assiniboine River and Lake Manitoba and Lake St. Martin outlet channels, the preferred measures identified in Section 23.0 would likely exceed $2 billion. Though the average annual benefits would also increase, the resultant B/C ratio would remain below 1.0, due to the very high cost of the flood protection measures. The effect that the Period of Record has on cost is summarized in the three examples provided in Table 22.4.

Table 22.4
Effects of Period of Record on the Cost for a 1:200 Year FPL

Long Period of Record (1882-2011)		Short Period of Record (1971-2011)	
Flood Mitigation Option	**Cost**	**Flood Mitigation Option**	**Cost**
963 m^3/s (34,000 cfs) Portage Diversion(1)	$ 1.1 Billion	1,838 m^3/s (64,900 cfs) Portage Diversion (1)	$ 2 Billion
888,000 dam^3 (710,000 ac-ft) Holland Dam(2)	$ 1.1 Billion	888,000 dam^3 (710,000 ac-ft) Holland Dam (3)	$ 1.8 Billion
790 m^3/s (27,900 cfs) Melita Dikes	$ 0.2 Million	985 m^3/s (34,800 cfs) Melita Dikes	$ 0.9 Million

Notes:
(1) Combined with upgrades to Lower Assiniboine River (654 m^3/s or 23,100 cfs capacity) and with Lake Manitoba and Lake St. Martin Outlet Channels (212 m^3/s (7,500 cfs) and 326 m^3/s (11,500 cfs) capacity).
(2) Combined with upgrades to Portage Diversion (1,020 m^3/s (36,000 cfs) capacity) and with Lake Manitoba and Lake St. Martin Outlet Channels (212 m^3/s (7,500 cfs) and 326 m^3/s (11,500 cfs) capacity).
(3) Combined with upgrades to Portage Diversion (1,840 m^3/s (65,000 cfs) capacity) and with Lake Manitoba and Lake St. Martin Outlet Channels (212 m^3/s (7,500 cfs) and 326 m^3/s (11,500 cfs) capacity).

Details of the analyses for the three locations have been provided in Appendix O. The results were consistently supportive of the conclusion that the economics of the options studied do not justify increasing the level of protection in this region to more than the basic standards summarized in Section 22.1 above.

23.0 PREFERRED MEASURES FOR UPGRADING FLOOD PROTECTION FOR THE LOWER ASSINIBOINE RIVER

Protection along the Lower Assiniboine River that would comply with the standard being adopted by the Province of Manitoba would require the ability to safely pass the 1:200 year flood. That is a flood of approximately 1,620 m^3/s (57,100 cfs) at Holland, and exceeds the flood of record (2011 Flood) by about 10%.

There are very few single options, which could feasibly achieve that level of protection. However, there were combinations of the options with relatively high economic indicators (primarily benefit / cost ratios) that could provide the desired level of protection. They are:

- Expansion of the Portage Diversion.
- Expansion of the outlet works at Lake Manitoba and Lake St. Martin.
- Enhancement of the dikes along the Lower Assiniboine River.

There are other options that could be interchangeable within the above list, but would be far more costly or would have significant environmental and/or socio-economic impacts. The best combination of the extent of each of the above options has been derived through a logical process that is summarized below.

- The Portage Diversion could be readily expanded to nearly double the discharge capacity of its original design. However, as described in Section 11.0, the cost of increasing to a discharge capacity that exceeds approximately 960 m^3/s (34,000 cfs) increases sharply.
- The dike protection along the Lower Assiniboine River could be upgraded to reach river flows well in excess of 655 m^3/s (23,100 cfs), but the option was limited by the river flow that could be tolerated within the City of Winnipeg, and the cost was more than the cost of upgrades to the Portage Diversion.
- The outlets at Lake Manitoba and Lake St. Martin should consider an increase in discharge capacity in the range of 142 to 283 m^3/s (5,000 to 10,000 cfs) since lesser capacities will not adequately reduce lake levels and greater capacities are not justifiable due to the high cost.

Consideration of these limits has identified a combination of options that reached the protection against a 1:200 year flood event on the Lower Assiniboine River. They consisted of the following:

- Expansion of the Portage Diversion as described in Section 11.0 to reach a design discharge capacity of 960 m^3/s (34,000 cfs).
- Upgrades to the Lower Assiniboine River capacity to permit passage of a flow of 655 m^3/s (23,100 cfs), as discussed in Section 10.1.
- Construction of a new Lake Manitoba outlet with a capacity of 213 m^3/s (7,500 cfs) and making the Lake St. Martin outlet channel permanent with a capacity of 326 m^3/s (11,500 cfs), as committed to by the Province of Manitoba and discussed in Section 15.0.

Economic assessment of this combination has revealed that increasing the level of protection beyond this minimum threshold would not be cost effective. It would cost more to provide the flood protection than the value of the damages that could be averted, as estimated using the principles explained in Section 7.0. However, the capacity of the preferred alternatives for the 1:200 year flood protection scheme could be optimized to minimize costs. The optimization process should be considered at a more detailed stage of design and would consist of preparing a more detailed cost estimate for each of the options and for a greater number of capacities. The options would then be compared to identify the most economical combination.

The preferred measures identified above are shown schematically on Figure 23.1. Table 23.1 summarises the total estimated costs of these proposed flood protection upgrades.

Figure 23.1
Schematic of Preferred Measures for Upgrading Flood Protection for Lower Assiniboine River

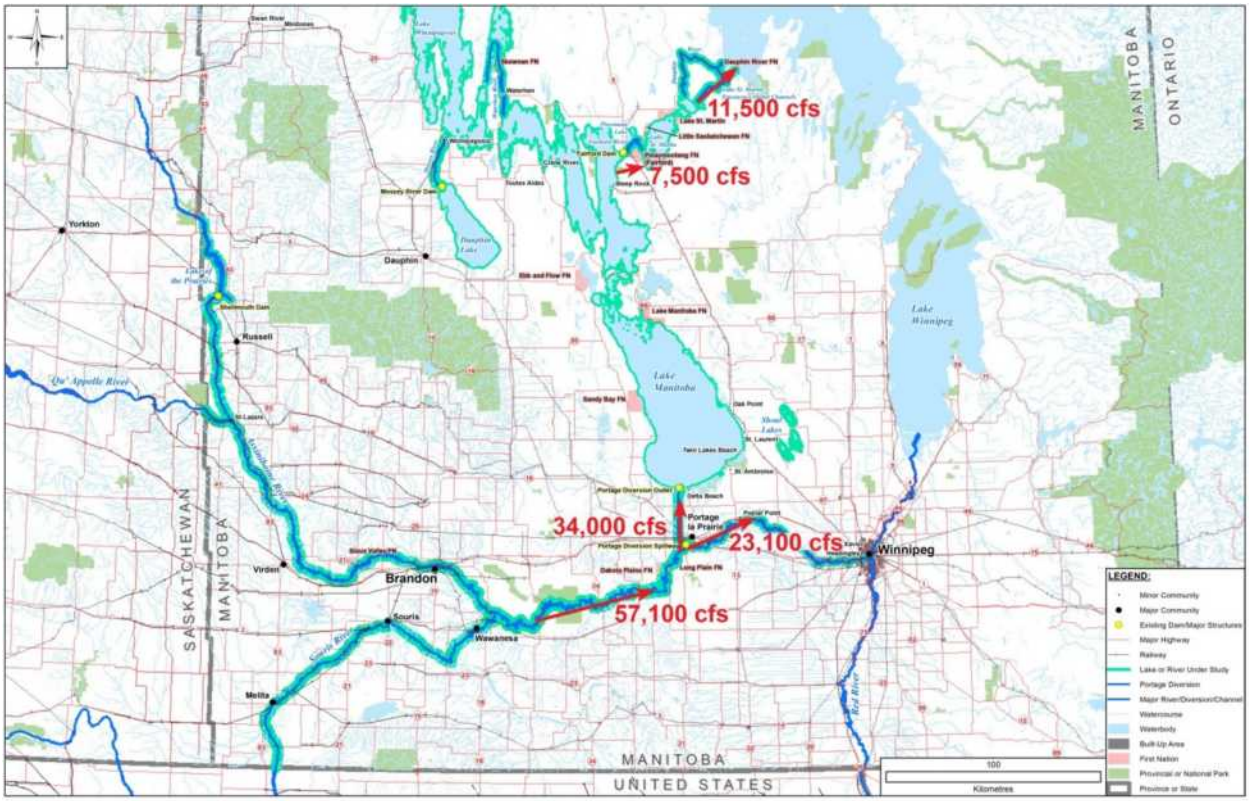
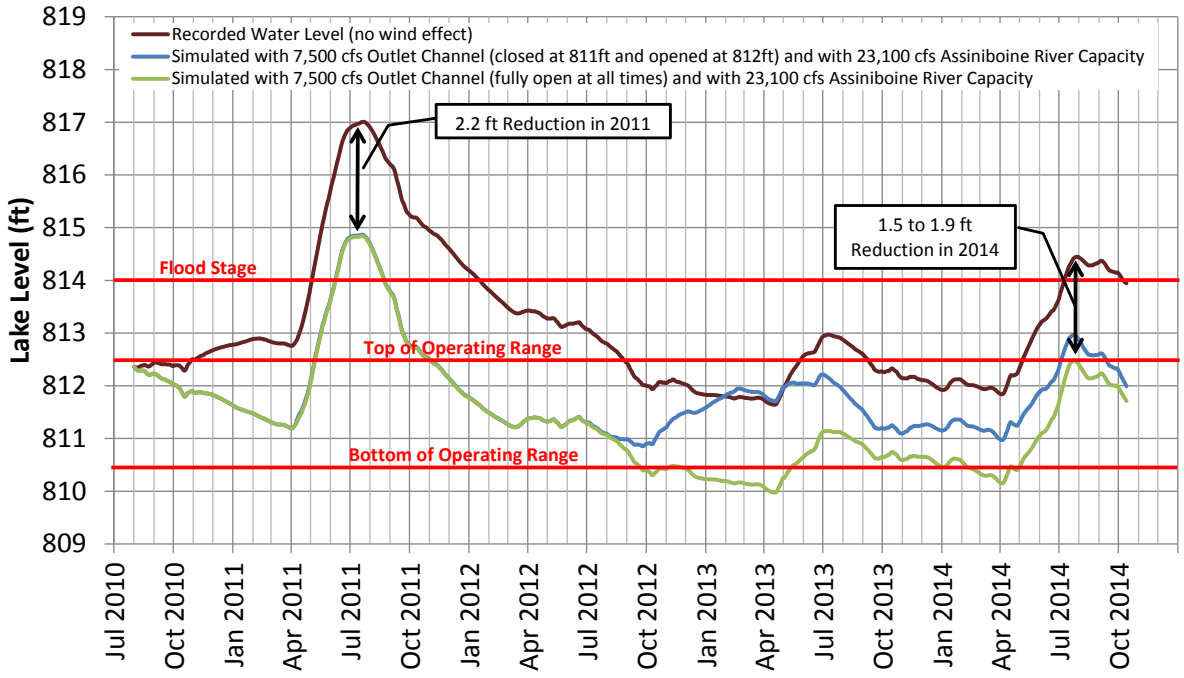


Table 23.1
Summary of Estimated Cost of Preferred Measures for Upgrading Flood Protection for Lower Assiniboine River

Flood Protection Infrastructure	Proposed Capacity	Preliminary Estimated Costs for Upgrade
Provincial Assiniboine Dikes	654 m^3/s (23,100 cfs)	$273 Million
Assiniboine River Baie St. Paul to Headingley	654 m^3/s (23,100 cfs)	$63 Million
Portage Diversion	963 m^3/s (34,000 cfs)	$373 Million
Lake Manitoba Outlet	212 m^3/s (7,500 cfs)	$230 Million
Lake St. Martin Outlet	326 m^3/s (11,500 cfs)	$220 Million
Total Estimated Cost		**$1,159 Million**

A flood routing analysis of the preferred measures identified in Table 23.1 was conducted on Lake Manitoba for the period of 2010 to 2014, and the results are shown on Figure 23.2. The analysis consisted of comparing the recorded water levels on Lake Manitoba (wind eliminated) to simulated water levels with the preferred measures in place. The results shown on Figure 23.2 indicate that, if the measures existed in 2010 or prior, the peak water levels on Lake Manitoba could have been approximately 0.67 m (2.2 ft) lower during the flood of 2011, and 0.46 m to 0.58 m (1.5 to 1.9 ft) lower in 2014, depending on the assumed rules of operation for the Lake Manitoba outlet channel. In 2014, the outlet channel would have maintained water levels below flood stage; and in 2011, it would have reduced the duration of the flood by about 180 days.

Figure 23.2
Recorded Water Levels on Lake Manitoba from 2010 to 2014
Compared to Simulated Conditions with Preferred Measures in Place



Note: To convert elevations to metres, multiply imperial values by 0.3048.

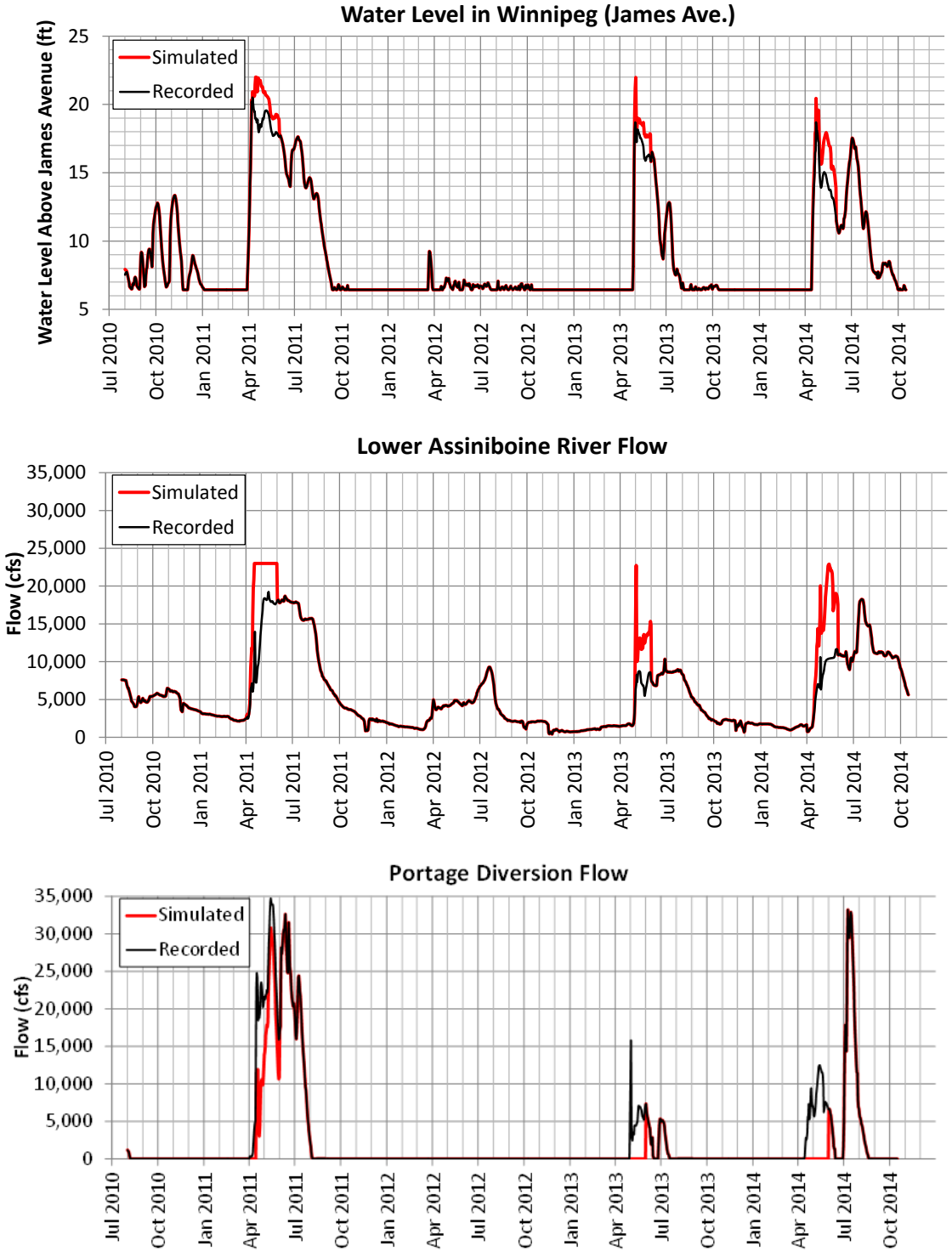
As shown on Figure 23.2, two methods of operating the Lake Manitoba outlet channels were simulated for the analysis. The first method consisted of operating the outlet channels based on the existing operating rules for the Fairford River Water Control Structure and the recommendations of the Lake Manitoba and Lake St. Martin Regulation Review Committee.

This consisted of opening the Lake Manitoba outlet channel when the water level in the lake reached El 247.5 m (El 812.0 ft) and then closing the outlet channel when the water level receded to El 247.2 m (El 811.0 ft). The alternate method consisted of maintaining the outlet channel fully open at all times, including during periods of low Lake Manitoba water levels.

The simulation also considered the increased capacity of the Assiniboine River to 654 m^3/s (23,100 cfs) downstream of Portage. With the increased Assiniboine River capacity, a lower volume of water was diverted into the Portage Diversion and Lake Manitoba during the months of April and May compared to what was actually recorded, resulting in slightly higher, but still manageable, water levels in the City of Winnipeg. However, after June 1st it was assumed that higher water levels in the City of Winnipeg would not be acceptable due to the increased risk of basement flooding. As such, the Assiniboine River flows were assumed to equal recorded flows after June 1 of each year. Simulated water levels in Winnipeg and flows on the Lower Assiniboine River and Portage Diversion compared to recorded conditions are shown on Figure 23.3.

It should be noted that there would be additional benefits on Lake Manitoba if flows on the Red River were below normal. This would provide the ability to pass more flow on the Assiniboine River towards Winnipeg, and reduce the volume of water diverted into Lake Manitoba. Additionally, if changes to the Portage Diversion operating guidelines call for operation to pass more flows on the Assiniboine River, added benefits to Lake Manitoba would accrue. The Provincial Flood Control Infrastructure Review of Operating Guidelines that was launched in the fall of 2013 by the Province of Manitoba is currently ongoing and has the mandate to review the operating rules and guidelines of several of Manitoba's major flood control structures including the Portage Diversion. The review will provide recommendations for possible modifications to operating rules and guidelines.

Figure 23.3
Recorded Water Levels in Winnipeg and
Flows on Lower Assiniboine River and Portage Diversion from 2010 to 2014
Compared to Simulated Conditions with Preferred Measures in Place



24.0 ENVIRONMENTAL AND SOCIAL IMPACTS OF THE PREFERRED ALTERNATIVES

Key environmental and social impacts that could be of concern for each mitigation alternative have already been presented in the report sections discussing these alternatives (Sections 9.0 through 17.0). The potential environmental and social impacts identified included:

- Elimination of flood plain area and oxbows cut-off from rivers.
- Increased water levels and potential flow pattern changes and associated changes to the sedimentation and erosion processes in rivers.
- Inundation of large areas of land from impoundment of reservoirs resulting in a loss of substantial terrestrial habitat, agriculture land (relocation of buildings and farmsteads) and First Nation reserve land.
- Impoundment and flooding of terrestrial habitat would increase nutrient levels and could increase mercury levels in the water that bioaccumulate in fish.
- Dams would affect fish and fish habitat and block fish passage / spawning.
- Dams would impede navigation along the rivers.
- Inter-basin transfer of water associated with diversion channels could impact water quality and biological conditions.
- Increased flood water levels in downstream water bodies.
- Loss of agricultural land acquired for construction footprint and easements.
- Channel excavation and reservoir impoundment could impact groundwater levels and water quality.

An allowance was carried in the cost estimate for each alternative for environmental mitigation costs. Environmental mitigation costs included activities such as long-term monitoring, mitigation of wells affected by a project, compensation for damage to fish habitat, compensation to fishermen or any additional mitigation and compensation measures that could come from Regulator requirements determined during the advancement of the licensing and permitting process. These costs were estimated as a percentage of the direct construction costs with the percentage ranging from 0 to 15% relative to the potential environmental effects. The basis for the cost estimates and a description of how this environmental mitigation allowance was incorporated was previously discussed in Section 7.0. A summary list of all the flood mitigation

alternatives and their estimated level of effort for environmental mitigation has been provided in Appendix K.

The key environmental and social impacts that could be of concern associated with the preferred measures identified in Section 23.0 are discussed in greater detail in the following paragraphs along with a discussion of the anticipated environmental approval process.

24.1 EXPANSION OF THE PORTAGE DIVERSION

The Retrofit Option consisting of upgrading the existing channel to expand the Portage Diversion Capacity, as described in Section 11.3, was selected at this level of concept design as the preferred alternative. This alternative was anticipated to have the least environmental and socio-economic impacts of the Portage Diversion Expansion alternatives as it consisted of raising the existing structures in the current location with minimal land acquisition requirements or expansion of the overall project foot print, although the potential effects have not been evaluated in great detail at this stage. Potential environmental and social impacts could be mitigated using standard best practices such as erosion and sediment control measures and fuel management. Further studies and detailed field investigations would also be necessary to determine the potential impacts of upgrades to the Portage Diversion on existing groundwater and salinity issues in the area.

24.2 UPGRADING THE LOWER ASSINIBOINE RIVER DIKES

The most feasible option to increase the capacity of the Assiniboine River between Portage la Prairie to approximately Baie St. Paul was to upgrade the existing dikes in their current location (Option 1a). From approximately Baie St. Paul to the City of Winnipeg limits in Headingley, the most feasible option was a combination of Individual Flood Proofing, Neighbourhood Diking and Purchase of Vulnerable Properties (Option 2b).

Since the concept consisted of upgrading dikes that already exist (Option 1a), the potential environmental and social impacts would be negligible and could be mitigated using standard best practices such as erosion and sediment control measures and fuel management. There would be no additional flooding of land that was currently protected by the existing dike

alignment, although some local improvements to the dike alignment would likely be incorporated at the detailed design stage to address riverbank instabilities and improve access. Downstream of Baie St. Paul (Option 2b), there would also be no changes to the flooded area; and the flood mitigation works would be local in extent. Therefore, the potential environmental and social impacts would be negligible and again mitigated using standard best practices.

24.3 LAKE MANITOBA AND LAKE ST. MARTIN OUTLET CHANNELS

The proposed construction and future operation of an outlet channel between Lake Manitoba and Lake St. Martin and the subsequent modifications to the existing outlet channel system between Lake St. Martin and Lake Winnipeg would provide a large scale positive effect by helping to alleviate flooding in the region. However, they would likely also result in a number of environmental concerns. Biophysical attributes that could be affected include, but were not limited to: surface water quality, groundwater, terrestrial environment, aquatic environment and fish resources. Social attributes and concerns that could affect the project included land use and ownership, access, and unresolved First Nation litigation claims. Considering the potential environmental and social impacts, Option C was identified as the best for the Lake Manitoba channels and was closely followed by Option D, whereas either of the Johnson Beach or Willow Point options was preferred for the Reach 3 portion of the Lake St. Martin channel.

24.4 ENVIRONMENTAL APPROVAL PROCESS

Environmental approvals for projects on these scales and of this nature could include, but were not limited to the following:

- Licencing under the *Manitoba Environment Act*, if the project is considered a development under the *Classes of Development Regulation* (164/88).
- Approval by the Canadian Environmental Assessment Agency (CEA Agency), if the project is a designated project under the CEAA 2012 Annexed Regulation SOR/2012-147A "*Regulations Designating Physical Activities*".
- Approval by Transport Canada under the *Navigable Waters Protection Act*, if the project could impact navigation on a water body that is on the "List of Scheduled Waters", which includes the Assiniboine River, Lake Manitoba and Lake Winnipeg.

- Approval by the Department of Fisheries and Oceans (DFO) under the *Fisheries Act* if the project activities are likely to cause serious harm to fish that are part of a fishery, or to fish that support such a fishery.
- Approval by other Federal Authorities, in accordance with Section 67 of CEAA 2012, for any portion of the projects on Federal Land.

Based on the amount of land area protected from flooding and the water surface area where levels would be modified, it was anticipated that all three projects of the preferred alternatives would require licencing under the *Environment Act*. In comparison, based on the reservoir surface area and the volume of water diverted in the channels, only the Lake Manitoba and Lake St. Martin channels were considered designated projects requiring approval by the CEA Agency. The requirement for approval by Transport Canada and DFO would need to be determined based on the amount of required in-water work as final design progresses, although it was anticipated that, as a minimum, a request for project review would need to be submitted.

25.0 SUMMARY OF PUBLIC CONSULTATION ACTIVITIES

One major component of the study was the public consultation process, which served as an opportunity to share information and obtain input from the public. To prepare for the public consultation activities, a draft public consultation strategy (PCS) was developed to help facilitate internal discussions related to identification of stakeholders, proposed consultation activities, and overall study messaging. This included creating opportunities for:

- The study team to share information with stakeholders.
- Stakeholders to ask questions and request more information.
- The study team and stakeholders to have discussions about potential effects, enhancement of benefits and mitigation options.
- Addressing concerns as they arose throughout the process.
- Facilitating the regulatory review and approvals by ensuring that an open and transparent public process was documented for the regulators.

The following sections summarize the process, information shared and feedback received during the consultation process. Information received helped inform future stages of the design and project development process.

25.1 PUBLIC CONSULTATION STRATEGY

The following principles guided the development and implementation of the public consultation activities:

- Accessible process: Provide opportunities and sufficient time for potentially affected and interested parties to learn about the study and to provide input.
- Transparent and open process: Encourage open dialogue and provide information in a form easily understood by non-experts.
- Variety of communication mechanisms: Encourage participation through a variety of mechanisms.
- Responsive approach: Consider results of the consultation process and provide feedback on the results of the process.
- Adaptive approach: Adjust the engagement process in response to issues and concerns.

Engagement activities provided information to a broad array of individuals and organizations in the study region. The PCS helped identify an initial list of potential target audiences for engagement activities. This list was later refined by the technical committee and study team. Stakeholders were targeted from the respective community leadership (e.g., Reeves, CAOs, mayors, councillors, First Nation and Aboriginal leaders) as well as the local residents (e.g., businesses, landowners, interested groups, etc.).

The following public consultation events were included as part of the study:

- Round 1 – Study Introduction – June 2013.
- Lake Manitoba and Lake St. Martin Outlet Channels – September 2014.
- Round 2 – Preliminary Results – December 2014.

25.2 ROUND 1 - STUDY INTRODUCTION

The first public consultation events effectively communicated as much information as possible regarding the scope of the study. Input from participants was solicited in order to understand their thoughts on what mitigation options should be considered to reduce flood effects and how these options should be prioritized, as well as their preferences for future consultation processes. The perspectives on preferred consultation mechanisms influenced the design of the following consultation rounds.

Information was shared at a total of six events: three meetings with community leadership and three public open houses. Events were held in Dauphin on June 18, 2013, Brandon on June 19 and Portage la Prairie on June 20. Community leadership meetings were held to provide local leadership with current information regarding the study in advance of the open house. Public open house events were held to provide opportunities for the general public to learn about the study and share their own experience and local knowledge of recent flood events. Total attendance for the first round of public open houses was approximately 49 people. Further details have been provided in InterGroup's November 2013 report in Appendix Q.

25.3 LAKE MANITOBA AND LAKE ST. MARTIN OUTLET CHANNELS

The next public consultation event provided information about Manitoba's flood protection model, including the current Assiniboine River and Lake Manitoba Basins Flood Mitigation Study, and how the Lake Manitoba and Lake St. Martin outlet channels will form part of a larger flood mitigation system. Input from participants was solicited on six options for the Lake Manitoba outlet channel and two options for the Lake St. Martin outlet channel. The information presented provided conclusions and recommendations on two options for people to consider for the Lake Manitoba outlet channel (options C and D) and a preferred option for Reach 3 of the Lake St. Martin outlet channel (Willow Point).

The information was presented at a public open house held in Ashern on September 18, 2014. Stakeholders were provided with an opportunity to review the storyboards and to speak with study team members to learn about the options, ask questions and provide their comments and perspectives. Total attendance for the Lake Manitoba and Lake St. Martin outlet channels open house was approximately 250 people, with a total of 65 comment forms completed. Further details have been provided in InterGroup's November 2014 report in Appendix Q.

25.4 ROUND 2 – PRELIMINARY RESULTS

The final consultation events effectively communicated information about the scope of the study, the objective of the study, a review of the preliminary results and provided an opportunity for feedback prior to the development of the final report. Input from participants was solicited in order to understand their experiences and concerns, as well as their preferences from the list of preliminary results.

Information was shared at a total of six events: three meetings with community leadership, and three at public open houses. Events were held in Dauphin on December 8, 2014, Brandon on December 9 and Portage la Prairie on December 10. Community leadership meetings were held to provide local leadership with current information regarding the study in advance of the open house. Public open house events were held to provide opportunities for the general public to learn about the study and share their own experience and local knowledge of recent flood events. Attendance included a total of 38 for the leadership meetings, 391 for the open houses

with 122 comment forms completed. Further details have been provided in InterGroup's January 2015 report in Appendix Q.

25.5 FEEDBACK FROM PUBLIC CONSULTATION EVENTS

Stakeholders came to the leadership meetings and open houses to share their personal perspectives and to discuss the different options presented with the open house team. While stakeholders shared different perspectives on the design options, many common themes emerged throughout these discussions. Some of the recurring issues brought up by the public included the following:

- Start implementing projects as soon as possible, before 2020. Many believe that the problems have been adequately studied and action is needed.
- Concerns about the fairness of compensation to landowners were raised. Some believe the current compensation package is not sufficient.
- Open the Lake Manitoba outlet now and strive to maintain a lower normal water level so that flood levels will also decrease.
- Increase outflow from Lake Manitoba higher than 212 m^3/s (7,500 cfs) to allow for the additional inflow from the Portage Diversion.
- Some residents are opposed to increasing flows on the Lower Assiniboine River, since they have already witnessed erosion and damage to vegetation and were concerned about higher flood protection costs, and agricultural and business losses.
- Some residents would prefer to increase flows on the Assiniboine River instead of adding more flow to the Portage Diversion.
- Concern was raised about water seeping through dikes, entering basements and damaging property.
- The study limits should have extended beyond Manitoba to fully address the root causes of flooding.
- Carry out regular maintenance of all channels to avoid buildup of silt and trees.
- Wetland restoration was suggested in conjunction with improved farmland drainage regulation throughout the watershed to reduce peak flows.
- A few residents were concerned about increasing salinity and seepage, which could reduce the value of farmland.
- Some were still afraid that another Hoop and Holler breach might be implemented.

- Prepare more detailed cost estimates and consider flood protection on a house by house basis including quality of life issues.
- Have a round table question and answer session with the Minister of MIT or the Premier. More detailed information and plans would be appreciated.
- Provide more notice and information prior to open houses.

Despite frustration with the current flooding situation for many regional participants, the majority of people noted appreciation of the effort that had been made to share the information and receive information from stakeholders. Stakeholders seemed genuinely enthusiastic about the study and appreciated the effort made to share and collect information. Stakeholders were interested in learning about and sharing their perspectives about the mitigation options being presented. A detailed summary of the feedback received for each of the public consultation events has been provided in the Appendix Q.

26.0 CONCLUSIONS

A number of conclusions have been presented at the end of each of the report sections where appropriate. These conclusions have been identified as those relevant to the specific area of study and may not be significant to the overall study. The conclusions presented below are those that will guide the future direction of the Assiniboine River & Lake Manitoba Basins flood mitigation program and are consistent with those presented at the final round of the Study open houses.

1. Manitoba has developed an extensive flood mitigation model. This integrated flood mitigation strategy consisting of floodways and diversions, dams, community diking, linear diking, individual flood protection and development controls has worked well as a system and has avoided billions of dollars of damages.

2. The flood of 2011 highlighted several potential weak links in existing flood control systems. The shortcomings of the system were emphasized again in 2014.

3. Although a massive amount of data and technical input has been assessed and over 70 mitigation options have been evaluated, more studies are needed to address the flooding concerns along tributaries and other areas not directly considered in this study.

4. In addition to this study, a number of related flood mitigation studies and projects are ongoing or have recently been completed that complement or contribute to the key findings in this study. These are listed below:

    a) Manitoba 2011 Flood Review Task Force Report.

    b) 2011 Lake Manitoba and Lake St Martin Regulation Review.

    c) Manitoba Surface Water Management Strategy.

    d) Provincial Flood Control Infrastructure Review of Operating Guidelines.

    e) Individual and Community flood protection province-wide including Lake Manitoba, Brandon, Souris, Melita, Reston, Wawanesa and other communities.

    f) The province's individual flood protection initiatives.

    g) Upgrades and rehabilitation to existing flood control infrastructure.

5. Many of the Assiniboine River & Lake Manitoba Basin communities are protected to or have programs that are in place to protect them to the highest flood on record or the 1:200 year flood event.

6. The level of protection for the Lower Assiniboine River area, which includes the Portage Diversion, is deficient with an estimated level of protection of 1:55 years.

7. Upper Assiniboine River:

   a) Implementation of the proposed Shellmouth Dam Leaf Gate Project will reduce flood peaks on the Upper Assiniboine River immediately downstream of the dam for moderate flood events, however, the gates will have no impact on major flood events. Further downstream from Brandon to Portage la Prairie, the flood control effects diminish and nearly disappear.

   b) Linear diking and purchase of flood prone land alternatives were considered and it was concluded the land purchase option is preferred, primarily due to the cost and long term effectiveness of this option.

   c) Linear Diking has been found to be the most feasible option to increase the level of protection for Brandon. A commitment has been made to upgrade the City's flood protection level to the highest flood on record (2011 and 2014) and is currently underway.

   d) The communities of St. Lazare and Sioux Valley are protected to the highest flood on record which exceeds the provincial Flood Protection Level target of a 1:200 year flood event.

8. Souris River:

   a) Linear diking has been found to be the most feasible means of flood protection for communities on the Souris River. Flood protection projects are currently underway at Souris and Melita to provide flood protection to the highest flood on record (the 2011 Flood).

   b) The community of Wawanesa is currently protected to a level that exceeds the highest flood on record.

9. Flood Protection with Large and Small Dams for the Upper Assiniboine River and Souris River:

   a) The feasibility of six independent large dams was assessed; and it was concluded that that they should not be considered further due to the significant environmental and operational concerns, as well their relatively high costs compared to other flood mitigation solutions.

   b) The operation of a combination of 21 small dams to reduce flood peaks was also assessed; and it was concluded that, since the flow reduction on the Souris and Assiniboine Rivers was relatively small, this was not a feasible option due to difficulties in effective operation of the dams, as well as the relatively high cost of the scheme compared to other flood mitigation solutions.

10. Lower Assiniboine River

   a) The reliable capacity of the existing provincial linear diking system downstream of Portage la Prairie has been estimated to be 425 m^3/s (15,000 cfs). Upgraded dikes in this area should be designed to meet current design guidelines similar to the City of Winnipeg primary dikes.

b) The most feasible option to upgrade the provincial linear diking system between Portage la Prairie and Baie St. Paul is to upgrade the existing dikes in their current location. Estimated costs to upgrade the dikes for a capacity between 510 m^3/s (18,000 cfs) and 793 m^3/s (28,000 cfs) ranges from $150 to $390 million (2014$).

c) Two alternatives were evaluated to increase the flood protection level downstream of the provincial Assiniboine dikes from Baie St. Paul to the City of Winnipeg limits in Headingley. The first option was to extend the provincial dikes; and the second option was a combination of individual flood proofing, neighborhood diking and the purchase of vulnerable properties. It was concluded that the second option was the most feasible based on costs, which were estimated to range from $21 to $123 million (2014$) for Assiniboine River capacities between 510 m^3/s (18,000 cfs) and 793 m^3/s (28,000 cfs).

d) An alternative that considered individual and community flood protection for this area, similar to that implemented in the Red River Valley, was an unfeasible solution due to the potential negative impacts and added costs for damages resulting from the widespread flooding.

e) A permanent wasteway at Hoop and Holler, to divert excess Assiniboine River flows towards the La Salle River was unfeasible due to economics, increased flood risks on the Elm Creek Drain and La Salle River, as well as significant opposition from local stakeholders.

f) Three options were considered to increase the capacity of the Portage Diversion from 708 m^3/s to 960 m^3/s (25,000 to 34,000 cfs). The preferred option considers retrofitting the existing diversion system to meet these objectives at an estimated cost of $373 million (2014$). Upgrades to the Portage Diversion should inherently eliminate the Failsafe.

11. Lake Winnipegosis

Four alternatives were considered to mitigate flooding on Lake Winnipegosis and reduce flooding impacts on Lake Winnipeg. It was concluded that these alternatives were not viable due to the relatively high costs compared to other alternatives, as well as the environmental impacts associated with these four alternatives.

12. Lake Manitoba and Lake St. Martin:

a) Six channel alternatives from Lake Manitoba to Lake St. Martin were considered to mitigate flooding on Lake Manitoba. The province is currently proceeding with preliminary design for the two preferred alternatives (Alternatives C and D) at a design discharge of 212 m^3/s (7,500 cfs). The preferred alternatives were selected on the basis of their cost effectiveness and comparatively high environmental ratings.

b) Two outlet alternatives were considered for the Lake St Martin to Lake Winnipeg channel. The province is currently proceeding with preliminary design of the channel at a design discharge of 326 m^3/s (11,500 cfs). Of the two alternatives under consideration, the Willow Point outlet for Reach 3 is the preferred alternative.

c) Estimated costs for the Lake Manitoba and Lake St. Martin outlet channels are $220 and $230 million (2014$), respectively.

d) For a 200-year event, the estimated peak water levels with the channels in place would be 248.9 m (816.6 ft) on Lake Winnipeg and 245.5 m (805.4 ft) on Lake St. Martin. These levels are approximately 0.3 m (1.1 ft) and 0.8 m (2.7 ft) lower than water levels would be for existing conditions on Lake Manitoba and Lake St. Martin, respectively.

13. Dauphin Lake:

a) The peak water level of 262.47 m (861.1 ft) on Dauphin Lake during the 2011 Flood was the highest on record and was estimated to have a return period of 1:110 year.

b) The study evaluated four auxiliary outlet channel options, river channel improvements and storage development to lower Dauphin Lake levels in times of flood and concluded that:

- Options investigated were not cost effective and had negative environmental impacts.
- Protection to the highest flood on record through Individual Flood Proofing and Development Controls (Designated Flood Areas) would be the most viable means to improve flood protection at Dauphin Lake.

14. Shoal Lakes:

a) Flood mitigation alternatives to address flooding issues within the Shoal Lakes Watershed were previously studied in 2008 to 2010.

b) Three options were considered at the time including:

- Diversion / outlet channel.
- Upland storage.
- Purchase of flood prone lands.

c) It was concluded that the most attractive option from an economic perspective was the purchase of flood prone lands, which is being pursued by the Province.

15. Recent studies by Dr. John Pomeroy (Pomeroy et al., 2014) related to the effectiveness of wetlands concluded that the impact of wetlands on flood peaks and volumes was significant for large floods, which was contrary to the majority of previous studies. Based on an assessment of the effectiveness of large scale wetland restoration, using the findings from the Pomeroy study, it was concluded that this would not be a cost effective mitigation option for the Assiniboine River Basin. Smaller scale flood mitigation projects should, however, be pursued; and this initiative is consistent with the Manitoba Surface Water Strategy, which endorses “no net loss” in wetlands and encourages water retention. A multi-year pilot study on wetland restoration should be considered to determine the costs and benefits of wetland restoration on flood attenuation. The cost would be in the range of $10 to $20 million.

16. Interprovincial drainage has been a long standing and very contentious issue between Manitoba and Saskatchewan, especially in the Upper Assiniboine Basin. There is an apparent frustration by Manitoba farmers as they cannot register a complaint directly against Saskatchewan drainage. Through the Assiniboine River Basin Initiative, Manitoba will have the opportunity to express its concerns to all jurisdictions regarding the drainage issue.

17. Current land use practices associated with forest management plans have been reasonably successful. Although Manitoba's Surface Water Management Strategy does endorse the principles associated with managing land use for the purpose of reducing runoff, it focusses on regulatory approaches for wetlands. Other land use changes associated with the cultivation of grasslands and deforestation should be emphasized.

18. Economic analyses of the individual flood protection options do not lead to the justification of flood protection for infrastructure in the study area that exceeds the basic requirements of either the 1:200 year flood or the flood of record. Even combinations of protection measures do not result in the economic justification to provide protection for floods exceeding basic requirements. This finding was applicable to all options and combinations of options devised in this study.

19. Based on the results of the economic analysis, flood protection measures, where necessary, should be planned for at least the 1:200 year flood, or the flood of record, whichever is greater, at all locations within the study area.

20. Protection along the Lower Assiniboine River that would comply with the standard adopted in the Province of Manitoba would require the ability to safely pass the 1:200 year flood. That is a flood of approximately 1,620 m^3/s (57,100 cfs) at Holland, and exceeds the flood of record (2011 Flood) by about 10%.

27.0 RECOMMENDATIONS

The recommendations presented below are intended to assist the Province in the selection and prioritization of future mitigation alternatives and policy decisions for implementation. These are also consistent with those presented at the final round of open houses. Based on the study results and conclusions, the following recommendations are presented:

1. The Lower Assiniboine River has been identified as an area which has infrastructure vulnerable to floods much smaller than the highest flood on record. It is recommended that this area be protected to a target Flood Protection Level of 1:200 years, which is consistent with the 2011 Flood Review Task Force recommendations. This increased protection would consist of upgrading the Portage Diversion to 960 m^3/s (34,000 cfs), upgrading the Assiniboine River capacity to 655 m^3/s (23,100 cfs), constructing a new Lake Manitoba outlet with a capacity of 212 m^3/s (7,500 cfs) and a Lake St Martin outlet channel with a capacity of 326 m^3/s (11,500 cfs). The estimated cost for this recommended flood mitigation system is $1,159 million (2014$). Considering the current low standard of protection within the Lower Assiniboine River and the Province's current commitments for Lake Manitoba and Lake St. Martin, these areas should be given the highest priority.

2. Development controls were considered as a means to restrict future development. Where there are no Designated Flood Areas (DFA) local municipalities and planning districts should more rigorously enforce flood protection levels in new developments and DFAs should be considered for the Lower Assiniboine River, Lake Manitoba and Lake St. Martin, as well as the Dauphin Lake areas.

3. The provincial flood protection policy should be revised to require flood protection to at least the 1:200 year flood, or the flood of record, whichever is greater, at all locations within the study area. This is consistent with recommendations from the Manitoba 2011 Flood Review Task Force Report.

4. The purchase of the flood prone land along the Upper Assiniboine River between Shellmouth Dam and St. Lazare was preferred over linear diking and should be considered further. The estimated cost was approximately $24 million for the flow of 85 m^3/s (3,000 cfs).

5. A multi-year pilot study on wetland restoration should be considered to determine the costs and benefits of wetland restoration on flood attenuation. The cost would be in the range of $10 to $20 million.

28.0 STATEMENT OF LIMITATIONS AND CONDITIONS

28.1 THIRD PARTY USE OF REPORT

This report has been prepared for Manitoba Infrastructure & Transportation to whom this report has been addressed and any use a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. KGS Group accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions undertaken based on this report.

28.2 GEO-ENVIRONMENTAL STATEMENT OF LIMITATIONS

KGS Group prepared the geo-environmental conclusions and recommendations for this report in a professional manner using the degree of skill and care exercised for similar projects under similar conditions by reputable and competent environmental consultants. The information contained in this report is based on the information that was made available to KGS Group during the investigation and upon the services described, which were performed within the time and budgetary requirements of Manitoba Infrastructure & Transportation. As the report is based on the available information, some of its conclusions could be different if the information upon which it is based is determined to be false, inaccurate or contradicted by additional information. KGS Group makes no representation concerning the legal significance of its findings or the value of the property investigated.

28.3 CAPITAL COST ESTIMATE STATEMENT OF LIMITATIONS

The cost estimates included with this report have been prepared by KGS Group using its professional judgment and exercising due care consistent with the level of detail required for the stage of the project for which the estimate has been developed. These estimates represent KGS Group's opinion of the probable costs and are based on factors over which KGS Group has no control. These factors include, without limitation, site conditions, availability of qualified labour and materials, present workload of the Bidders at the time of tendering and overall market conditions. KGS Group does not assume any responsibility to Manitoba Infrastructure &

Transportation, in contract, tort or otherwise in connection with such estimates and shall not be liable to the Client if such estimates prove to be inaccurate or incorrect.

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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS
FLOOD MITIGATION STUDY

FINAL REPORT

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KGS Group 12-0300-011

January 2016

(*printed in November 2015*)

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APPENDIX A

LIST OF STUDY LIAISON COMMITTEE MEMBERS

Study Liaison Committee Members

Fisaha Unduche	(Manitoba Infrastructure and Transportation)
Eugene Kozera	(Manitoba Infrastructure and Transportation)
Jason Senyk	(Manitoba Infrastructure and Transportation)
Henry Daniels	(Manitoba Infrastructure and Transportation)
Dan Laychuk	(Manitoba Infrastructure and Transportation)
Herb Mahood	(Manitoba Infrastructure and Transportation)
Christopher Budiwski	(Manitoba Agriculture, Food and Rural Initiatives)
Elmer Kaskiw	(Manitoba Agriculture, Food and Rural Initiatives)
Paul Guyader	(Emergency Measures Organization)
Mike Gagne	(Emergency Measures Organization)
Wayne Hildebrand	(Conservation & Water Stewardship)
Geoff Reimer	(Conservation & Water Stewardship)
Grant Mohr	(City of Winnipeg)
Ted Snure	(City of Brandon)
Wesley Penner	(Manitoba Hydro)
Genevieve Ali	(University of Manitoba)
Ronald Stewart	(University of Manitoba)
Clinton Cleave	(Rural Municipality of Ochre River)
Rick Plaiser	(Rural Municipality of Sifton)
Stan Cochrane	(Rural Municipality of Sifton)
Trever King	(Rural Municipality of Woodlands)
Doug Oliver	(Rural Municipality of West Interlake)
Darryl Jackson	(Town of Souris)
Scott Greenlay	(Association of Lake Manitoba Stakeholders)
Jane Hook	(Association of Lake Manitoba Stakeholders)
Gerald McIvor	(Southern Chiefs Organization)
Michael Anderson	(Manitoba Keewatinowi Okimakanak)
Randi Thomas	(Manitoba Metis Federation)
Chris Watson	(Manitoba Metis Federation)
Melissa Hotain	(Assembly of Manitoba Chiefs)
Pascal Badiou	(Ducks Unlimited)
Greg Bruce	(Ducks Unlimited)
Alvin Zimmer	(Shellmouth Liaison Committee)
Lorne Henry	(Shellmouth Liaison Committee)
Chris Reynolds	(Whitemud Conservation District)

APPENDIX B

LIST AND SYNOPSIS OF REFERENCE REPORTS

APPENDIX B1
SUMMARY AND SYNOPSIS OF BACKGROUND REPORTS

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Review of Reference Material

Doc. Number	Date of Report	Title	Author(s)	Number of Pages	Synopsis of Contents	Reviewer Comments
03-16	2003	1999 Post-Flood Report for the Souris River	U.S. Army Corps of Engineers	103	Review of 1999 flood on Souris River in Sask, ND, MB. Section 7.3 includes approximate discharge capacity of Souris River channel in Manitoba. Has a few recommendations at end of report, most of them regarding forecasting.	reports states that cost of 1999 flooding in MB was approx $100 million, mostly agricultural.
-	2012	2011 Flood - Fairford River Flow Metering Report	KGS Group	17 + Appendices	Report summarizing hydrometric data collected on the Fairford River and methods used for collection.	
-	2012	2011 Flood Assistance - Bridge Sonar Survey	KGS Group	53 + Appendices	Report summarizing river bed scour effects at all bridge crossing on the Assiniboine River within Manitoba from Shellmouth to Winnipeg.	Contents include survey information at some bridges. Some Hydrometric Data also included.
-	2012	2011 Spring Flood - Assiniboine River Watershed Flow Metering Report	KGS Group	80 + Appendices	Report summarizing hydrometric data collected on the Assiniboine River and significant tributaries and methods used for collection. Includes Portage Diversion	Data collected at bridges crossings. Rating Curves were developed at some of the sites
-	2012	2011 Spring Flood - Souris River Flow Metering Report	KGS Group	29 + Appendices	Report summarizing hydrometric data collected on the Souris River and methods used for collection.	Data collected at bridges crossings. Rating Curves were developed at some of the sites
	2011	Analysis of Options for Emergency Reduction of Lake Manitoba and Lake St Martin Levels	KGS Group and AECOM	15	Executive summary document on options to bring the levels of Lake St. Martin and Lake Manitoba down to the desirable range on an emergency basis following the 2011 flood. Includes conceptual cost estimate for 7 Options. The report concludes that constructing the Lake St. Martin emergency channel and maximizing FRWCS flows provides for the most timely and cost-effective means of lowering Lake Manitoba water levels.	
81-52	1981	Assiniboine - La Salle River Diversion Preliminary Engineering Report	PFRA	87	This is an investigation into 5 different alternatives developed to provide more flow throughout the year in the La Salle River for agricultural purposes by diverting water from the Assiniboine by either gravity flow or pumping methods. Various locations were selected and studied, west of Portage for the Assiniboine South Hespler area.	Associated costs are provided.
83-21	1983-1987	Assiniboine - South Hespeler Study Social and Environmental Effects	PFRA Environmental Studies	56	Report summarizes results of a study to examine the feasibility of planning for Lower Assiniboine Reservoirs, regional water supply canal, and potential irrigation areas.	Options include: Pembina River Diversion, Lake Manitoba to Pembina River Triangle Diversion, Assiniboine River Diversion, multiple options for Holland Reservoir and Rathwell Dam.
-	2007	Assiniboine River - Brandon Manitoba-Hydraulic Assessment	Bruce Harding Consulting	205 Pages including extensive Appendices	Description and results of numerical modeling of the Assiniboine River in a reach of 30+ km, addressing estimated water levels for revised hydrological conditions (24% increase in 1:100 yr flood from previous flood risk mapping)	Useful document for estimating water levels in and around Brandon. Includes flood frequency curve for Assiniboine as of 2006, river rating curves at various points along the river in this reach, and accompanied by the HEC-RAS dataset
-	2006	Assiniboine River at Brandon Flood Risk - Computation of the 100 Year Flood Discharge	Manitoba Water Stewardship	22	Description of the analyses used to update the estimate of 1:100 year flood for the Assiniboine River at Brandon; includes the estimation of the effects of regulation if the Shellmouth had existed prior to 1970, as well as the inclusion of anecdotal floods	Substantial uptick in the flood estimate since the previous 1:100 value had been developed in 1981; new estimated 1:100 year flood at Brandon is reported here to be 784 cms
-	2011	Assiniboine River Basin Hydrologic Model - Climate Change Study	Stantec	102	This project was done to assess the potential effects of climate change on stream flow and soil moisture. MIKE-SHE was used to generate a hydrologic model for historical stream flow and moisture conditions and compared to 3 modeled future conditions. The future conditions were modeled by MIKE-SHE with input data from the Canadian Regional Climate Model.	Thorough description of model and assumptions
84-36	1984	Assiniboine River Degradation Investigation Downstream of the Shellmouth Dam	Manitoba Natural Resources - engineering and Construction Branch	37	Description of surveys of the Assiniboine River in the summer of 1982 to assess the progress of degradation since the previous review by Vic Galay in early 70's. Some erosion was noted nearest the structures at Shellmouth but only nominal changes had occurred downstream and it could not be concluded whether that was due to a degradation process, or just natural variations in channel geometry with time.	Limited value in this study except possibly some indication of river cross sections
-	2001	Assiniboine River Dike Reconstruction - Pre- and Post- Reconstruction - Cross Section River Surveys of 2000	North South Consultants	121 pages including extensive appendices	Description and results of surveys of the river at transects adjacent to sites where dikes ha d been built up in the late '90's and in 2000, between Portage la Prairie and Baie St. Paul	Detailed cross sections are shown, along with comments regarding effects on the channel cross section viz a viz effects on fish.
1 12 00920	1997	Assiniboine River Dikes Reconstruction - Pre-design Draft Report 1997-1999	IDG-Stanley	150 with drawings drill logs, specs	Preliminary design report on Assiniboine Dike Reconstruction including surveys, subsurface exploration, lab testing, assessment of erosion rates, design criteria, land acquisition, quantities, costs, drawings, draft specifications, and soil logs	
98-311-02	1999	Assiniboine River Diking Project - Baie St. Paul to Headingley	KGS Group	113	Feasibility report on dikes in this reach, including geotechnical, hydraulics, economics, as well as building survey data	Good overview of economic evaluation of dikes; only two of the reaches studied have B/C ratios exceeding 1

Review of Reference Material

Doc. Number	Date of Report	Title	Author(s)	Number of Pages	Synopsis of Contents	Reviewer Comments
-	1974	Assiniboine River Diking Project - Preliminary Land Control and Land Cost Report (Draft)	PFRA	34 pages	Status of land ownership along the Assiniboine Dikes, prepared in preparation for transfer of ownership from PFRA to the Province of Manitoba	Useful background, possibly of value in considering enhancement of dike system; interesting to note that this report was prepared in 1974 and the transfer of the dikes occurred in 1996
-	1995	Assiniboine River Flooding and Operation of the Shellmouth Dam	Manitoba Water Resources Branch	270	Documentation of unregulated river flows from 1971 to 1995 from Shellmouth to Portage as they would have been if the Shellmouth Dam had not been built. Studies used a combination of a SIMPAK routing model for the reservoir and a Muskingum routing model for eight river reaches between Shellmouth and Portage	Some information provided on river rating curves at various locations, as well as Muskingum routing coefficients and stage vs. flood areas.
92-7	1992	Assiniboine River Flow Enhancement - Shellmouth Reservoir Supply Study	Manitoba Regional Division - PFRA	118	Due to water supply concerns along the Assiniboine River during drought conditions a study was conducted regarding the feasibility of an increased water supply and storage capability behind the Shellmouth Dam. Two scenarios were simulated applying known water supply and usage. Also considered was Saskatchewan water usage of flows in the upper Assiniboine basin and removing the minimum flow requirement for the Brandon Thermal Generating Station. Hydrological flow regime changes were evaluated and subsequent environmental impacts.	
-	2012	Assiniboine River Hydrodynamic Modeling Study	Hatch	190	Report documenting the hydrodynamic model that was set-up and calibrated for the reach of the Assiniboine River between Shellmouth and Brandon to evaluate impacts that operation of the Shellmouth dam had and will have in the future. To simulate historical conditions a complete record of inflows was developed for all major tributaries within this reach between 1976 and 2011. MIKE-11 1D was used for the entire study reach along with MIKE-21 FM for specific areas of interest which were an 8km stretch near St. Lazare and a 30km stretch u/s of Brandon.	
66-16	1966	Assiniboine River Project Shellmouth Outlet Conduit Hydraulic Model Study	PFRA	112	This report contains detailed model results on the shell mouth dam and outlet conduit encompassing the design details to date (1966). The physical model was built to a scale ratio of 1:36:2.	Results are provided for the following areas: Upstream Transition, Upstream Transition pressures, Upstream Transition Loss Coefficients, Discharge Capacity Curves, Gates, Downstream Transition, Downstream waves.
60-19	1960	Assiniboine River Project Shellmouth Reservoir	PFRA (Principal Author Lowood, J.D.)	47	Report on preliminary design of Shellmouth Dam. Includes cost estimates and design of structures	not of value for current study as report outdated and Shellmouth Dam now in existence.
70-9	1970	Assiniboine River to Souris River Diversion	Province of Manitoba - Water Control and Conservation Branch	100	Documentation of studies of a diversion channel from the Assiniboine River near Oak Lake to the Souris River a few miles downstream of Hartney. The purpose was to be for water supply and capacities of less than 1000 cfs were contemplated. There was no consideration of flood control, and this was not seen as a significant benefit. There was no B/C analysis provided, although the cost estimates were reasonably detailed.	This does not seem to be an attractive option for flood control in the Assiniboine Basin.
07-2	2007	Assiniboine River Water Management Model Documentation	Manitoba Water Stewardship	58	This report documents the fundamentals of the model that was developed to study the water supply and flood management issues in the Assiniboine River Valley from Shellmouth Dam to Headingly. It also addresses the analyses done to assess the scheme of gate installation at the Shellmouth Dam. By Manitoba water Stewardship, 2007.	
53-1	1955	Backwater Study The Pas - Cedar Lake Reach Saskatchewan River	Manitoba Water Resources Branch	82	This is a brief report documenting the slope-discharge relationship between control points and the Saskatchewan river between Cedar lake and The Pas. Rating curves were developed at each of the gauges in this reach as well as backwater effects at upstream points of controlled levels on Cedar Lake	
58-14	1958	Benefit Cost Analysis Lake Manitoba Regulation (1958)	Province of Man - Dept of Public Works	18+figures	Follow-up study to Lakes Winnipeg and Manitoba Board Studies, to refine B/C of Lake Man Regulation	B/C estimated to be 1.8. Interesting to note that the damages are 100% to farm interests

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61-2	1961	Benefit-Cost Analysis Assiniboine River Flood Control & Water Conservation Projects (1961)	Prof Ed Kuiper, University of Manitoba for Manitoba Water Control and Conservation Branch	69	Economic analysis of various combinations of Portage Diversion, Shellmouth reservoir and Holland Dam to identify the best alternatives(s) that should accompany construction of the Red River Floodway. The result was a recommendation of the Shellmouth dam and Portage Diversion with design capacities essentially as was constructed and exist today. The methodology builds on techniques and basic information developed by the Royal Commission on Flood Cost Benefit of 1958.	Useful to see the dependence on the data gathered for the Royal Commission and the possible incorporation of the methods and data available therein.
83-5	1983	Canada - Manitoba Flood Damage Reduction Program - City of Brandon and Rural Municipality of Cornwallis - Flood Damage Reduction Study	Manitoba Water Resources Branch	46+ appendices	Assesses flood mitigation options for Brandon, recommends upgrading of Brandon Flats Dike and implementation of restrictions in land development and usage in flood prone districts	Deserves more detailed consideration in planning of mitigation options
83-5	1983	City of Brandon and R.M. of Cornwallis Flood Damage Reduction Study	Manitoba Water Resources Branch	124	This study examines measures to reduce flood damages & guide floodplain management in the Brandon Flats area within the city of Brandon. It identifies flood risk areas, zoning, and ownership, estimates flood damages from previous events, proposes alternatives, and evaluates the economic benefit of alternatives (in 1981 dollars). Alternatives included structural (dams, reservoirs, dykes, diversions) and non-structural (relocation, flood proofing, regulatory).	Some of the analysis may be useful, but is likely out of date. The report also includes City of Brandon Dike Upgrading Geotechnical Report as Appendix B.
81-23	1981	City of Brandon and R.M. of Cornwallis Flood Risk Mapping Study	Manitoba Natural resources - Water Resources	129	Includes maps of dikes and zoning in Brandon, and stage-discharge frequency relationships.	
93-13	1994-1997	Dauphin Lake Basin Stream Rehabilitation Study	Natural Direction Env. Consultants	118	This study produced a stream rehabilitation program for 7 tributaries of Dauphin Lake. The study classified the present condition of stream channels, banks, riparian zones and fish habitat. The priority for stream rehabilitation was established and initial rehabilitation plans were developed that included: a description of the area, survey results, rehab options and rehab costs.	
73-31	1973	Dauphin Lake Regulation - Volume 1	Manitoba Water Commission	37+ figures	Study of changing regulation range of water levels on Dauphin Lake	Not of much relevance, but there is reference to flood control benefits, as measured exclusively by agricultural production
84-35	1984	Dauphin Lake Regulation and Siltation Study	Manitoba Water Commission	76+appendices	Many recommendations to address sediment issues and lake level regulation. For example dredging of the Mossy River - not clear what if any recommendations were acted upon.	Deserves more detailed consideration in planning of mitigation options
90-7	1990	Dauphin Lake Sediment Monitoring	Manitoba Water Resources Branch	114	This report documents the findings of a field monitoring program of sediment in tributaries of Dauphin Lake. The program involved two separate methods of measurement of sediment concentration on 7 waterways. The runoff in 1990 was less than normal.	
92-2	1992	Dauphin Lake Sediment Sourcing	Manitoba Water Resources Branch	168	This report documents the findings of a suspended sediment measurement program for tributaries of Dauphin Lake in 1991. Four southern tributaries were selected from the previous program in 1990. These were estimated to supply 86% of the sediment to Dauphin Lake. They included the Turtle River (highest contributor to sediment), Vermilion River, Edwards Creek, and Ochre River.	
90-2	1990	Dauphin Lake Sedimentation Study 1990 Report	Manitoba Water Resources Branch	168	This report summarizes findings from review of suspended sediment measurements in tributaries of Dauphin Lake for the years from 1957 to 1986 inclusive. Average annual sediment inflow to Dauphin Lake was estimated through this study to be 319,000 tonnes per year, with over 95% of that being deposited in the lake. It was estimated by the study that this only equated to about 6.3 cm of depth on the lake bottom over a period of 100 years if projected into the future.	
79-9	1979	Dauphin Lake Study Proposal	Dauphin Lake Working Group	76	This appears to be a combination of documents that made up a proposal for a 3 to 5 year study to identify the optimal range of water levels on Dauphin Lake. It seems to be mixed with recommendations of the Manitoba Water Commission pertaining to optimal ranges. It is not clear from this document whether the study was ever commissioned as proposed therein.	
78-6	1978	Detailed Evaluation of Selected Flood Damage Reduction Options Along the Manitoba Portion of the Souris River	Souris River Basin Board	74+appendices and figures	Analyses a variety of flood mitigation options to reduce flood damages to agricultural and residential (Wawanesa, Melita, Souris) interests.	Deserves more detailed consideration in planning of mitigation options
58-26	1958	Drought and Flood in the Qu'Appelle Watershed	PFRA	77	Studies of Drought and Flood have been carried out. The report concludes that land use zoning could be of value, but that construction of reservoirs to control floods in the Qu'Appelle Valley is not feasible.	

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-	2000	Fairford FN Flood Study Phase One - Volume I Engineering Report (2000)	Reid Crowther & Partners Ltd.	163	Diverse topics covered in this report ranging from identification of 1:100 year water levels along the Fairford/Lake St. Martin system, evaluation of housing quality and numbers lying within the 1:100 year flood zone, as well as consideration of groundwater conditions.	This may have some value in quantifying residential assets that could be damaged by flood waters.
	2013	Fairford River Water Control Structure Infrastructure Damage Assessment – Phase 2	KGS Group	69	The objective of the Damage Assessment was to determine, to the degree possible, the validity of damage claims by First Nations communities, their causal connection to the operation of the FRWCS, and to quantify the value of consequential damages. Only the approach and results of the Infrastructure Damage Assessment are documented in this report.	
99-9	1999	Feasibility Report for Improved Flood Protection for the Town of Melita	Manitoba Conservation	5+attachments	Diking study for town of Melita. B/C ratios barely over 1.0	Needs to be considered further at mitigation stage, appears to have reasonably up to date data.
70-16	1970	Feasibility Study & Cost Estimate of the Lake Manitoba - Garrison Reservoir Diversion	Masters Thesis	242	This is a feasibility and cost estimate study indicating that indeed it is feasible to deploy pumping stations in conjunction with a canal and a series of dams/reservoirs on the Assiniboine and Souris Rivers to move water from Lake Manitoba to the Garrison Reservoir. Capital and Annual costs are provided.	Effects of implementation are covered as well as a list of some deemed important but not studied in this report.
99-7	1999	Feasibility Study Report for Flood Protection Dyke for Saskatchewan River Ralls Island Area - The Pas, Manitoba	Cochrane Engineering Ltd. for Manitoba Dept of Natural resources	18 + attachments	Addresses flood issues on Saskatchewan River near The Pas	Likely no relevance to the study of Assiniboine / Lake Manitoba
77-32	1977	Flood Damage Assessment Along the Manitoba Portion of the Souris River	Souris River Basin Study Board (Principal author, Bob Harrison)	83+appendices and figures	Good treatise on flood damages to agricultural and residential interests; includes quantified estimates and other useful information on rating curves flood frequencies etc.	This is a valuable source of information - further assessment needed as planning evolves in current study
83-12	1983	Flood Frequency Analysis Little Saskatchewan River at the Town of Minnedosa - APP H	Manitoba Natural Resources - Water Resources	13	The portion of a report predicts the 1 in 100 year flood discharge based on the frequency curve of the annual maximum daily discharge in the Little Saskatchewan River Watershed. The dam was excluded from the analysis since it is operated for recreation and was assumed to have no appreciable effect on peak flows.	
83-8	1983	Flood Frequency Analysis Whitemud River and Neepawa Creek at the Town of Neepawa Appendix H	Manitoba Natural resources - Water Resources	20	The 2 rivers were split into 2 reaches for analysis and flood flow frequency curves were developed for each using stream flow records in the watershed in order to produce the 1 in 100 yr discharge.	Explanation is concise and data is provided along with the result.
)223129.40	2012	Flood Mitigation Study for First Nations along Fairford River, Lake St. Martin, and Lake Pineimuta	AECOM	76	Review of Flood Mitigation Options for LSM/Dauphin River. Includes options to upgrade LSMEOC and diking	Hydraulic design of LSMEOC options will be of value during the hydraulic assessment portion of our study. Options also include cost estimate.
-	2011	Flood Protection Levels and Designated Flood Area Legislation	Province of Manitoba	26	Power Point Presentation on flood protection legislation, design criteria for flood protection measures….	Possibly of value during review of diking alternatives or during review of development of controls/ land use changes…
-	2013	flood_review_task_force_appendices	Flood Review Task Force - David Farlinger	184	Appendices include public open house results, hydrologic and hydraulic information, information on 2011 forecasting, paper on wetland drainage, as well as an environmental report from Hatch.	
-	2013	flood_review_task_force_report	Flood Review Task Force - David Farlinger	192	Report that examines many aspects of the 2011 flood including operation of provincial flood control works, flood mitigation measures, accuracy and timeliness of forecasting, level of flood preparedness, adequacy of existing flood protection infrastructure, environmental and social impacts, land use policies and zoning criteria, adequacy of communications & impacts on road networks. 126 recommendations were included in the report.	
57-20	1958	Floods and Flooding Problems in the Qu'Appelle Valley	PFRA	109	Seems to be a parallel report, and probably a precursor to Report 58-26. No new information on floods, however, there are river rating curves and flood frequency curves presented for the Qu'Appelle River that may of some value later in KGS Group's studies.	
09-22	2009	From the Mountains to the Sea - The State of the Saskatchwan River Basin	Saskatchewan River Basin	173	The document provides a broad understanding of the functionality of the entire river basin through hydrology, water use, water quality and biodiversity. Its intent is to provide a snapshot of existing conditions (2008). Vulnerabilities and threats to the basin as a whole are discussed.	There is a chapter on Lake Winnipeg as a brief overview of conditions. It discusses current challenges facing the lake's overall quality and health and outlines a few detrimental activities. Specific relevance is unknown.

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-	2004	FRWCS Interlake Flood - Final Report (2004)	UMA Engineering	73	The objective was to evaluate the impact of operation of the FRWCS and the Portage Diversion (PD) on downstream water body levels. A computer model of pre-dam conditions was created and used to simulate water flows and levels that would have occurred if the FRWCS and PD had not been built. The impact was determined by comparing observed conditions to the modeled "natural" conditions. The analysis determined that the operations have increased the frequency and severity of both high and low water levels along the Fairford River, Pineimuta Lake and Lake St. Martin with a direct impact on the Pinaymootang, Little Saskatchewan and Lake St. Martin First Nations associated with flooding and impacts on ranching, fishing and trapping.	
	2012	Geotechnical and Hydraulic Assessment of Flood Protection Options for the Town of Souris	KGS Group	80	The report reviews the condition of the temporary dikes along the Souris R. and Plum Cr. In the Town of Souris and whether they are capable to function as permanent flood protection to a flood of magnitude comparable to that of 2011. Seepage and slope stability evaluations are summarized and subsequent alternatives to consider are provided.	Part 1 of 3: Souris, Wawanesa, Melita
	2012	Geotechnical Emergency Engineering Services Assiniboine River Dikes – 2011 Flood Report	KGS Group	1021	This draft report (as per March 2013) consists of a full documentation of the 2011 flood activities for the Assiniboine River Dikes, an analysis and assessment of the effectiveness of 2011 activities (lessons learned), post-flood geotechnical investigations and analyses of the current geotechnical condition of the critical sites of the 2011 dikes, and recommendations for both short term repairs to the dikes as well as recommendations for long term management of the diking system.	
75-42	1975	Impact on Waterfowl Breeding Success of Flooding in Pineimuta Lake Due to Regulation of Water Levels on Lake Manitoba, Including Suggestions for Management	Canadian Wildlife Service	56	Fluctuations of water levels on Lake Pineimuta have hampered the production of breeding ground for many migratory birds in the area. Recommendations were provided as to how the lake should be operated although there is no hydrologic or hydraulic evidence to support these.	Relevance is unknown.
02-25	2002	In-stream Flow Needs Modeling for Four Reaches of the Assiniboine River	Golder Associates	161	This report documents the modeling process developed to gain a better understanding of water-use practices along the Assiniboine River and how it affects fish habitat. The analysis was split into 4 reaches: Brandon, Holland, Lido Plage and St. Lazare. The hydraulic model was developed using River2D. Unregulated and regulated flow conditions were applied to each reach to generate weekly Weighted Usable Area curves for each fish species of interest.	Some explanation, but primarily modeling results
64-7	1964	Interim Report on Measures to Supplement Low Flows on the Souris River in Manitoba	Water Control and Conservation	73	Compares several water supply projects and recommends the best - the Coulter Dam on the Antler River, with a B/C of 2.5 and 26,500 acre-ft of storage. Storage on Gainsborough Creek is also considered economically feasible (16,400 acre-ft). Other options were also addressed on Graham Creek, Jackson Creek, Blind Souris, and the Main Stem of the Souris River. Detailed costs are provided along with a limited extent if drawings of the schemes studies.	Figure 18 shows a cost vs. storage relationship generated by the study. This could be of value in the current study despite its base in 1963 dollars.
54-7	1954	Investigation of Means of Lowering Lake Manitoba - Report on Auxiliary Channel Scheme No. 2 on Fairford River	J.S. Verner - Water Control and Conservation	6 + figures	Report on surveys of channel improvement schemes to supplement outflow from Lake Manitoba	Clearly only input to a broader study.
-	1971	Lake Dauphin Regulation Study	Manitoba Water Resources Branch	222	Report addresses flooding and damages, past and revised regulation, hydraulic information of outlet in order to determine a practical range of regulation for the outlet to mitigate immediate and future flooding. Looks at Outlet improvements, additional mediation and benefits.	Logically walks through steps taken to determine mitigation efforts. Contains historical inflows of Dauphin Lake and outlet rating curves.
-	2013	Lake Manitoba - Lake St. Martin Regulation Review Draft Report	Lake Manitoba/lake St. Martin Regulation Review Committee - Westdal	154	Appendices include a 70 page document on Land Use Planning by McKay Finnigan & Associates as well as public engagement results	Land Use Planning report will be of value.
-	2013	Lake Manitoba - Lake St. Martin Regulation Review Draft Report	Lake Manitoba/lake St. Martin Regulation Review Committee - Westdal	144	This report is a presentation of recommendations with a range of proposed regulations for Lake Manitoba and lake St Martin as well as recommendations regarding control structures, land use planning and zoning. Ideas were developed using an approach in 3 areas including: Hydrology, Public Engagement, other studies and Research. Propositions included additional water control works, practical range of regulation for both lakes, land use policies and zoning criteria around water bodies vulnerable to flooding.	

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82-1	1972-1984	Lake Manitoba Regulation	Unknown	243	A collection of memo's and submissions regarding various methods for regulating water levels on Lake Manitoba. Letters were submitted between 1972 and 1984 and are accompanied by drawings, specs and associated costs for the various schemes discussed.	
73-38	1973	Lake Manitoba Regulation - Volume 1	Manitoba Water Commission	44+ figures	Addresses flood levels on lake Manitoba and identifies some mitigation measures, principally lower target control level. Seems to have a broad identification of benefits and disbenefits to all interests. Strongly concludes that a lower target water level on lake Man cannot reduce periodic high flood conditions. Recommends leave target at 812.17 ft.	Good description of the history of the Lake Manitoba and the evolution of the control on the Fairford River; merits a more detailed review and understanding of the extensive facts and issues identified.
73-38A	1973	Lake Manitoba Regulation - Volume 2	Manitoba Water Commission	131	Appendices to Volume I - TOR of review, plus transcripts of public meetings carried out	Limited value now, but may be worth reviewing when our public hearings are planned.
61-11	1961	Lake Manitoba Regulation Operating Rules of the Fairford River Dam (1961)	Manitoba Water Control and Conservation	36	Report explains the hydraulic methods and operation of the Fairford Dam in compliance with the water level regulation imposed on Lake Manitoba. Uses 3 different operating rules to accommodate seasonal flows and storage.	Minimal text but gives brief description of factors involved in deciding upon operating the dam. Rating curves for the dam are attached.
68-17	1968	Lake Manitoba Supply Canal - Preliminary Report	Manitoba Water Resources Branch	53+ figures	Study of a diversion canal from Lake Manitoba to Assiniboine River through Long Lake to supplement low flows in Winnipeg to deal with sewage pollution in drought conditions.	Not much relevance to current considerations pertaining to floods, but worthwhile to note the potential side benefit of a channel of this type if built for flood relief.
71-26	1971	Lake Manitoba to Pembina Triangle Diversion	Manitoba Water Resources Branch	303	Report provides a preliminary design and cost estimate for the 4 components of the diversion which are: diversion to the Assiniboine, Hood Dam and Pump St, Rathwell Pump St, Assiniboine River to Pembina Triangle Canal. Involves pumping water from the Portage Diversion and moving it to by means of pumping until gravity can carry it to the Pembina Triangle. Requires slight modifications to the Portage Diversion. Reviews level of supply at 200 cfs, 1000 cfs, and 2000 cfs.	Detailed design and drawings provided in Appendices
71-18	1971	Lake of the Woods to Pembina Triangle Diversion Scheme	Manitoba Water Resources Branch	231	This is a documentation of studies of a water supply diversion of up to 1000 cfs from Lake of the Woods to the Pembina River watershed.	This does not appear to have merits in flood protection for the AR&LMB. Further consideration is not needed.
78-46	1978	Lake St. Martin & Pineimuta Lake Regulation	Manitoba Water Commission	38	Summarizes findings of a study of options to mitigate swings in water level on these lakes; concludes that none of the options are economically justifiable	Good background and description of the various interests around these lakes, as well as a list of mitigation measures considered (but not much detail on their composition and performance)
77-21	1977	Lower Whitemud River & Big Grass Marsh Flood Control Study - Phase I & II (1977)	Manitoba Water Resources Branch	113	Report is an evaluation of several flood mitigation options deemed most reasonable, presented in other reports for the Lower Whitemud River and Big Grass Marsh. A cost benefit ratio was used for comparison. A combination of options was chosen as most suitable with the Gopher Creek Diversion and a controlled release of the Big Grass Marsh.	Reviews 9 options
79-15	1979	Lower Whitemud River and Big Grass Marsh Flood Control Study Phase II	Water Control and Conservation	493	This study describes the technical and economical feasibility of providing flood control on the Whitemud R. and around Big Grass Marsh. 6 control schemes were developed involving combinations of various sizes that would divert water from the Whitemud R. around Gladstone to the marsh via Gopher Cr. Diversion. Water is to be held in the marsh by dikes/control structure . The capacity of the Whitemud R. d/s of the marsh would be increased. Project reduces flood damage for Gladstone, Woodside and Westbourne. Benefit/cost ratios provided.	
79-15B	1979	Lower Whitemud River and Big Grass Marsh Flood Control Study Phase II Appendix E – Project Costing	Water Control and Conservation	96	Provides cost estimates for each scheme developed in Phase II. Involves purchasing land to the 1% flood level around the marsh. 4 alternative compensation methods are applied.	
79-15A	1979	Lower Whitemud River and Big Grass Marsh Flood Control Study Phase II Appendix E (Tables only)	Water Control and Conservation	24	detailed Capital Construction Cost Estimates	
77-50	1977	Manitoba Northlands Community Planning Geotechnical Land-Use Study Waterhen	Manitoba Water Resources Branch	67	This report defines the stratigraphy and soil conditions of the area, shoreline conditions and erosion, an evaluation of groundwater and construction materials for the watermen area. This information is basic to community planning.	Relevance is unknown.
	1984	Modern Sedimentology and Hydrology of Lake Manitoba	William M. Last, U of M	14	This research based article maps out the sedimentology of Lake Manitoba and discusses 3 lacustrine processes influencing the lake: depth, morphology, water chemistry. Discuses composition and increased rate of sedimentation resulting in 0.3m of material on the lake bottom int he last 50 years in the south basin due to increased drainage and discharge from the portage diversion. Portage Diversion is also responsible for elevated levels of trace metals in the south basin.	

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	2010	Natural and unnatural complexities- flood control along Manitoba's	Shannon Sunden Bower	11	This paper provides an interesting and factual history of the development flood control measures along the Assiniboine River, and in particular the evolution of the diking system. The history of the jurisdictional issues is also of interest and explains the various government bodies that have been involved, in some cases with conflicting interests, in the flood protection in the Assiniboine Valley. There are some useful maps that show the locations of dikes and cutoffs that were developed over the years.	
71-23	1971	Other Storage in the Assiniboine River Basin (SNBB)	Manitoba Water Resources Branch	400	Study of the use of additional storage reservoirs to supplement low flows in the Assiniboine. Existing reservoirs at Shellmouth and Rivers were studied in combination with 7 other reservoirs in the region. A system numerical model was used to assess the benefits, and used mean monthly flows.	Good starting point for consideration of dam sites and reservoirs, but there was zero focus in this study on flood control
63-3	1963	Planning Study Report - Souris River Basin in Manitoba - Appendix A - Agricultural Report	Water Control and Conservation	35+figures	Agricultural Report describes soils, land use, water supplies and irrigation potential of 17 physiographic regions	No mention of flood risks, and probably out of date for current study needs
34929.04.	2011	Portage Diversion Design Basis (Hatch 2011)	Hatch	247	Summary of the basis of design for all major components of the Portage Diversion, including components designed by Acres as well as the channel and structures designed by the Province. There are extensive drawings included.	This is a valuable source of specific information on performance of the PD. It will be useful in KGS Group's studies of possible expansion of the works.
-	2011	Portage Diversion Flood Control Infrastructure Assessment - Pre-2011 Flood Final Summary Report	Hatch	136	Wide ranging report on risks, capabilities, vulnerabilities pertaining to the PD as well as the dikes between Portage and Headingley.	Risks should be reviewed and added to vulnerability lists. Detailed river and channel hydraulic models need to be requested from MIT.
64-24	1964	Portage Diversion Model Study Interim Report No. 1 Construction and Preliminary Testing of Typical Channel	Clayton, G.L.	30	Discusses decision regarding construction of specific components of the model. Explains testing methods and provides results of the testing done on the model.	Relevance is unknown.
64-12	1964	Portage Diversion Outlet Structure Model Studies Report No. 1 Wind Conditions, Wind Setup & Wave Characteristics Over the South Portion of Lake Manitoba	Manitoba Water Control and Conservation	24	Report summarizes the hydraulic design of the portage diversion outlet structure with regards to wind and wave action and how the model will be used to assess this. Discusses what assumptions were made to develop the appropriate loading conditions for these cases.	Limited information. Attached data includes a frequency analysis on wind conditions.
76-46	1976	Portage Diversion Proposed Modification to Reach 1A (1976)	Manitoba Water Resources Branch	46	Documentation of studies of modifying the "failsafe" section of the west dike along the Portage Diversion. Looked at raising dike (no overflow allowed), containing the spillage from a a field failsafe section, purchasing the affected land, or do nothing. Costs are defined	It is not clear whether there was any consideration of the effect on the Outlet Structure if the failsafe cannot fail. Also, it is not clear what, if anything, was done as a follow-up to the study.
65-24	1965	Portage Diversion Report on the Soil Mechanics Aspects Involved in the Test Dyke Near Lake Manitoba	Manitoba Water Control and Conservation	40	Review of test dike built on Reach 1A of the Portage Diversion with regards to performance and stability of high fill sitting on soft marshland. Also, review of the economic practicality of such a project in order to make judgments for future work.	Primarily a geotechnical study
99-34	1999	Pre-Design Report - St. Lazare Flood Protection Infrastructure	StanTec for Water Resources branch of Manitoba	10 pages + attachments	Assessment of alternative dike options for the Town of St. Lazare.	May have merit in consideration of mitigation measures. Not clear whether the least expensive of the alternatives was actually built.
77-2	1977	Preliminary Evaluation of Flood Damage Reduction Option in the Manitoba Portion of the Souris River (1977)	Souris River Basin Study Board (Principal author, Manfred Samp)	42+figures	Considers a wide range of water supply options to supplement low flows in droughts.	No consideration of flood issues directly, but the options should be kept on the radar screen for future consideration in our study.
94-5	1994	Preliminary Report of the Assiniboine River Management Advisory Board to the Minister of Manitoba Natural Resources	Assiniboine River Management Advisory Board	17	Outlines the establishment and goals of the Assiniboine River Management Advisory Board. This board is primarily concerned with water demand, supply, and allocation In the Assiniboine River Basin (Including Qu'Appelle and Souris Rivers).	This does not address flooding, and will not be of value to this study.
58-9	1958	Preliminary Report Souris - Pembina Diversion	Manitoba Water Resources Branch	17	Preliminary report on studies of a diversion channel carrying a design flow of between 1000 and 3000 cfs. No B/C evaluation, sketchy delineation of the project. Objective of the project was primarily water supply. This report appears to be of limited value as an option for flood control.	
75-57G	1975	Primary Production in Churchill and Reindeer River Lakes	J.R. Lawrence	27	This is the study of how the construction of a hydro-electric dam will affect the production of phytoplankton in the Churchill and Reindeer Lakes due to inundation of the Churchill and Reindeer Rivers. Factors studied that affect primary production include: light penetration, nutrient availability and flushing rates.	Unsure of the full relevance

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91-9	1991	Probable Maximum Flood for the Shellmouth Dam	PFRA	27	Report intended to investigate the feasibility of increasing the storage of the Shellmouth reservoir. This report supersedes a previous estimation determined simplistic in calculation of moisture input, lack of sub basins and absence of computer modeling.	Used HEC-1 to model the event.
90-5	1990	Probable Maximum Rainfall and Snowmelt for the Assiniboine River Watershed Upstream of Shellmouth Dam	Manitoba Water Resources Branch	39	Report on PMP upstream of Shellmouth Dam. Provides hourly moisture input for PMF event.	Nothing of value as PMF not considered in current study .
80-26	1980	Proposal to Provide an Overview Assessment of Design and Operation of the Red River Floodway, Portage Diversion and Shellmouth Reservoir	Charles Howard and Associates Ltd	77	A proposal document to conduct work. Relevance unknown. Questions with associated answers were raised with topics that included the Red River Floodway, Portage Diversion and Shellmouth Reservoir.	I don't see the relevance of this proposal to the study
69-1	1969	Proposal To Supply Potable Water to the Village of St. Claude	Water Control and Conservation	15	No hydrologic or hydraulic data or analysis. Purely costs associated with distribution of water to st claude	relevance unknown
61-15	1961	Proposed Shellmouth and Holland Reservoirs and Portage Diversion	PFRA	50	A review of 3 major water control structures along the Assiniboine River. It outlines the key decisions used for reviewing these 3 options and makes a comparison of each measure. Dollar values are associated using values appropriate to 1960.	Basic review. Minimal detail. Large portion outlining assumed costs.
55-27	1955	Qu'Appelle Basin Floods of 1955	PFRA	70	Explains magnitude and distribution of 1955 flood with regards to precipitation, major areas of runoff and consequences. Reactions in tributaries of the Qu-Appelle basin and adjacent waterways was also reviewed.	Not a lot of depth but a good overview of how the Qu-Appelle basin reacted to an unusually high degree of runoff.
56-31	1956	Qu'Appelle Basin Floods of 1956	PFRA	90	Builds off of previous report with more detail since the 1955 flood encouraged extensive hydrologic and hydraulic data acquisition. No attempt to interpret the flood was made at the time the report was created.	Same format and approach as previous report "Qu'Appelle Basin Floods of 1955"
89-03	1989	Reconnaissance Study of Potential Storage Development on Assiniboine River Tributaries	PFRA	187	Study of enhancement of low flows in Assiniboine with 11 previously studied reservoirs the Shell, Birdtail, Little Sask, and Souris Rivers, in combination with new options for reservoirs on small tributaries;	The dams were addressed individually, and no combined effects were cited. Also, there was no focus on flood control.
2003-05	2003	Regulation of Water Levels on Lake Manitoba and Along the Fairford River, Pineimuta Lake, Lake St. Martin, and Dauphin River and Related Issues	Lake Manitoba Regulation Review Advisory Committee	Summary Report - 43 pages Main Report-101 pages	Gives a variety of recommendations on operation of the Fairford Dam and control of water levels along this system; the objective is to achieve a range of El 810.5 to 812.5 ft on Lake Manitoba, while also reducing the variability's of water levels on the system downstream to lake Winnipeg	These two reports contain a wide variety of factual information on the water resources of the region, including lists of infrastructure and land usage.
-	1978	Rehabilitation of Assiniboine River Dikes 1978 Cost Estimates	PFRA	12	Cost Estimate to rehabilitate dikes at 10 damaged locations. Two alternatives were investigated: Alt 1 was to maintain as much as possible current alignment and Alt 2 was to set back the dikes 50-100m to avoid endangerment from river erosion for 50 year.	May be of value during review of dike alternatives and cost estimates
	1952	Report Conservation and Flood Control Assiniboine River	PFRA	73	This report is an extension of a 1944 investigation with a purpose to reduce flood hazards along the Assiniboine River considering flood effects in WPG and regulating water levels for water supply.	Reviewed various mediation measures singularly and in combination in order to give a proper recommendation of the most successful solution. Cost/benefit included in comparison.
52-8	1952	Report Conservation and Flood Control Assiniboine River - Diversion at Portage la Prairie Appendix 4	PFRA	136	This appendix reviews flood control methods to protect the area between Portage la Prairie and WPG, including WPG. 5 diversion routes were investigated both hydraulically and regarding their estimated cost over various discharge capacities to determine feasibility.	Section with hydraulic design. Section regarding effects on Lake Manitoba.
52-6	1952	Report Conservation and Flood Control Assiniboine River - Headwater Storage, Channel Appendix 2	PFRA	92	This appendix reviews the mitigation method of headwater storage by comparing large reservoir storage to numerous small scale detention ponds. A dollar value was calculated for both options relating to acre-feet.	Detailed design of the Russel reservoir is attached
97-40	1997	Report of the 1997 Flood Monitoring Program Red River Floodway and Portage Diversion	Acres International Ltd	62	Review of deficiencies at Floodway and at Portage Diversion, pertaining to problems during and after 1997 flood. Recommendations to improve are included.	Mostly maintenance issues identified. A few big-picture points pertaining to ice jamming at Outlet Structure of PD. Should be reviewed with MWS / MIT to see what, if anything, was done for follow-up.
	1958	Report of the Royal Commission on Flood Cost Benefit	Royal Commission on Flood Cost Benefit	129	Summary report on assessment of the optimal flood protection works for southern Manitoba, including Winnipeg and the Assiniboine Valley.	Techniques and data presented may be of value in the current study of the Assiniboine Valley.
74-15	1974	Report on a Hydrologic Study of Headwater Storage in the Riding Mountain Escarpment of the Whitemud River Watershed	Manitoba Water Resources Branch (Bob Harrison, principal author)	11 + figures	Hydrological characteristics of flood control schemes in the upper headwaters of the Whitemud River	Very small storage reservoirs, not likely candidates for significant results either in Whitemud watershed or downstream

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58-16	1958	Report on Assiniboine River Diking and Cutoffs	PFRA	7+ figures	Summarizes work done between portage and Headingley 1950 to 1958. History of diking program described, identifying weak sections of dike. Dikes estimated to be capable of handling a minimum of 18,000 cfs with up to 22,500 cfs in some areas	Interesting, but likely out of date for current purposes.
55-10	1955	Report on Assiniboine River Flooding	Manitoba Water Resources Branch	133	Report provides an overview of flooding history in the valley prior to 1955, a description of factors influencing the 1955 flood and possible flooding in the future. Details 12 floods prior to 1955, existing flood control measures and forecasting.	Basic report regarding "cause and effect" of historical floods in the Assiniboine River valley. Scarce detail.
11203060	1999	Report on Geotechnical and Structural Alternative Solutions for Rehabilitation of Sites 15, 17, and 18 of the Assiniboine River Dike Project	Stantec Consulting Ltd.	23	Evaluation of alternative means to rehabilitate three dike reaches varying from 220 m to 410 m in length; structural solutions were found cost prohibitive; earth fill dikes with slopes from 1:4 to 1:5 were recommended	Might be of use in considering dike enhancements in current study
58-20	1958	Report on Measures for the Control of the Waters of Lake Winnipeg and Manitoba Supplementary Volume I	Lakes Winnipeg and Manitoba Board	281	Interesting description of history and resources of the area and how floods have adversely affected them. Some data on hydrology too, but this is missing the Main Report that must discuss the schemes developed.	This document describes the interesting event in the spring of 1882 when the Assiniboine River reportedly overflowed to Lake Manitoba via the area now occupied by the Portage Diversion, and Lake Manitoba peaked at El 818 ft, causing overflow back to the Assiniboine river about 30 km downstream of Portage.
58-21	1958	Report on Measures for the Control of the Waters of Lake Winnipeg and Manitoba Supplementary Volume II	Lakes Winnipeg and Manitoba Board	215	Appendix 5: Lake Manitoba Regulation for Flood Control - includes option for Diking, land purchase, Fairford Dam and channel improvements, Water Supply Channel to Assiniboine River Appendix 7: Lower Assiniboine River Regulation - includes option for Diking, land purchase, Diversion Channels, Headwater Storage, dredging (other appendices not relevant to this study)	Lake Manitoba options and Lower Assiniboine options should be reviewed during assessment of mitigation options. Multiple alternatives may be of interest. Includes cost estimate and benefits
58-17	1958	Report on Measures for the Control of the Waters of Lakes Winnipeg and Manitoba (1958) Extracts	Lakes Winnipeg and Manitoba Board	148	Extract of several different reports and appendices of "Report on Measures for the Control of the Waters of Lakes Winnipeg and Manitoba" Also includes extract on 1954-55 Report on Saskatchewan River	Not useful as only partial copies of reports included.
58-21A	1958	Report on Measures for the Control of the Waters of Lakes Winnipeg and Manitoba (1958) Main report	Lakes Winnipeg and Manitoba Board	80	report on options to control water on Lake Winnipeg, Lake Manitoba, Lower Assiniboine, Sask. River, Nelson River.	More valuable is Appendix 5 and 7 which discuss in more details options on LMB and Lower Assiniboine. Refer to Doc no. 58-21
-	2006	Report on Shellmouth Reservoir Operation and Assiniboine River Flooding - Winter 2005 to Spring 2006	Manitoba Water Stewardship	124	Overview of the events related to the flood of 2006. Identifies weaknesses in dike heights Shellmouth to Millwood, and the conflicts between operating to minimize peak as opposed to extended duration of above natural flows post-flood peak.	Interesting remark that flood of 2006 was so large that regardless of how Shellmouth was operated it could not have prevented flooding even if operated with perfect foreknowledge.
4809	1915	Report on the Lowering of Lake Manitoba	Dominion Power Water Branch	53		Unable to read.
84-33	1984	Review of Portage Diversion	Acres International Ltd	62	Almost exclusively hydraulic in nature. A variety of issues were addressed, including proven discharge capacity of the channel, difficulties with control due to ice conditions, channel degradation and its potential effects. Remaining vulnerabilities were identified. A variety of additional studies were recommended.	Useful to review in more detail in future as part of assessment of improvements and/or expansion of the Portage Diversion
80-17	1980	Review of the Red River Floodway, Portage Diversion and Shellmouth Reservoir (1980)	Manitoba Water Commission	253	Review and analysis of resulting adverse effects of which each system creates for surrounding areas after all 3 have been in operation together for 10+ years. Review of operating procedures and response to issues addressed by Manitoba Water Commission.	Not a lot of technical support or calculations nut rather engineering knowledge and judgment
65-32	1965	Review Survey of Souris River, North Dakota for Flood Control	U.S. Army Corps of Engineers	65-32	Studies of flood protection in the us portion of the Souris river catchment. Flood vulnerability is primarily at Minot. Two servo or schemes were recommended, one near Burlington, North Dakota, and another on the des lacs river above Kenmare North Dakota. Extensive drawings and engineering data provided on the proposed reservoirs, as well as B/C analysis. B/C s of 1.9 to 2.1 were estimated. Not clear if these reservoirs were ever developed.	Warrants consideration as alternatives to assist the Manitoba side of the watershed.
69-7	1969	Sask.Nelson Basin Board Alexander Dam on Assiniboine River	Manitoba Water Control and Conservation	139	Design and cost estimate of Alexander Dam option.	Hydraulic design info will be of value during hydraulic assessment. Cost estimate needs to be updated but good reference as a starting point.
65-31	1965	Saskatchewan Reclamation Project – Pasquia Area	PFRA	87	This report explains the establishment of the Pasquia area and its relevance as a test region for the larger Sipanok Area. Construction plans and details are explained discussing how exterior water was held outside, internal water was removed and how the drains, dams and diversions were constructed. Cost estimates are provided for all components.	Pasquia Area - The Pas

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53-21	1953	Saskatchewan River Reclamation Project - Degree of Protection	PFRA	28	This is a report that discusses the appropriate design flood magnitude for the Saskatchewan River Reclamation Project. It discusses the three key factors in selection of the design flood - general engineering considerations, economic optimization, and protection of human life. Each aspect is discussed and addressed with respect to the Saskatchewan River Reclamation Project. The recommendation provided is a design criterion of a 1:1000 year flood, which is governed by general engineering principles.	Classic description of key factors to be considered in selecting degree of protection for dikes.
78-8	1978	Selected Agricultural Flood Damage Reduction Programs Along the Souris River in Manitoba	Manitoba Water Resources Branch	25	Studies of diking, river channel enlargement, channel realignment, land acquisition, alternative cropping practices, and income stabilization; none were considered to be acceptable and effective long-term solution to agricultural flood problems	Details of options studied and estimation of benefits are limited.
78-9	1978	Selected Urban Flood Damage Reduction Programs Along the Souris River in Manitoba	Souris River Basin Board	15 + figures	Analysis of flood protection options for the towns of Wawanesa, Souris and Melita	Deserves more detailed consideration in planning of mitigation options
-	2002	Shoal Lakes Water Management Options - Preliminary Investigation	Ad Hoc Study Group on Shoal Lakes Water Management	33	Summary of studies of three options to relive the flooding problems. All three were diversion channels. One was ruled out due to a low B/.C ratio, the others were recommended for more detailed study.	Precursor to a follow-up study by KGS Group
08-0311-01	2010	Shoal Lakes Watershed Study Report	KGS Group	187	Report is an evaluation of the flooding issues within the Shoal Lakes Watershed. 3 mitigation options were designed to a feasibility level, reviewed and assessed. Each option was accompanied by an environmental screening level assessment.	Contains: Inflow-Available-for-Outflow water balance model, Hydraulic model of diversion channels and cost benefit analysis
71-44	1971-1972	Some Interrelationships of Vegetation, Wildlife and Settlement of the Whitemud River Watershed	Department of Mines, Resources and Environmental Management	34	This document was prepared with the intention to describe pre-settlement flora/fauna, pattern of pre-settlement and impact of this on landscape of the whitemud river watershed. Information was gathered through land surveyors reports/maps, published observations of pre-settlement and published histories.	Unsure of relevance.
00-64	2000	Souris River Basin 1999 Post Flood Report	U.S. Army Corps of Engineers	65	Summary review of the events of the 1999 flood, estimated to be about a 1 in 10 year event. Refers to flood damages in Manitoba to be several hundreds of millions of dollars in hay losses.	First recommendation was for the International Souris River Basin Board to expand its mandate from one of overseeing apportionment to include coordination of flood forecasting, the determination of flood operation, and the issuance of operational guidance. Also recommends the development of a flood forecasting model that addresses the entire drainage basin comprehensively.
78-37	1978	Souris River Basin Study - Flood Damage Reduction	Souris River Basin Study Board	196	Overview of the study area, its physiography, hydrology, and other aspects. Good description of the highest profile problem (flooding), as well as identification of a wide variety of options considered in the study	Good reference - should be reviewed in more detail when developing mitigation options. Interesting to note that none of the flood reductions schemes identified and studied showed viable economic return on investment. The only options of value appeared to be zoning, land use regulations and a limited number of flood proofing schemes.
78-38	1978	Souris River Basin Study Supplement 2 Flood Damage Reduction	Manitoba Water Resources Branch	403	flood damage assessment along Souris River in Manitoba. Preliminary and detailed assessment of flood reduction options. Includes cost-damage relationships, stage discharge relationship, frequency curves.	Multiple flood alternatives investigated, and should be looked in more detailed during development of mitigation alternatives. Most relationships & curves are likely outdated and cannot be used for our study. However should be reviewed again during hydraulic assessment & cost of damages
-	1984	Souris River Study	Manitoba Water Commission	53	Broad review of flooding concerns in MB on Souris River. Includes table showing year by year agricultural damages ($) due to flooding.	reports states that structural alternatives considered in 1978 Souris River Study to alleviate flooding upstream of Harney are still not economically justifiable. First recommendation is the purchase of flood prone lands. Other recommendations focus on policies and guidelines regarding future drainage practices in basin
69-10	1969	Storage on the Souris	Manitoba Water Control and Conservation	25_figures	Study of existing and potential storage sites to supply water for irrigation in the Souris River region.	Appears to have limited value for current study of flood mitigation
73-43	1973	Summary of Flood Control Studies in the Whitemud River Watershed	Manitoba Water Resources Branch	17 + figures	Addresses a wide variety of flood protection schemes in the Whitemud River watershed. No scheme was found to have economic merits.	Good summary of schemes addressed, but appear to have no direct pertinence to flood control in Lake Manitoba

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65-9	1965	Summary Report Lake Winnipegosis Regulation Study	Manitoba Water Control and Conservation Branch	22	Studies of control of Lake Winnipegosis - mostly directed at avoiding low levels.	Not clear what the effect on flood levels on Lake Manitoba would be.
69-11	1969	Supplementary Study to Whitemud River Basin Planning Study Report Part I	Water Control and Conservation	138	This study updates the assessment provided in 1968 with results of the discharges seen in the 1969 spring flood. Peak Q=2950 cfs. With the recent results the damage and cost estimates were restructured and projected for a 50 year period. It was concluded that the most economical solution was to protect individual property rather than construct major mitigation measures.	
81-1	1981	Surface Water Management Plan for the Upper Whitemud River Watershed	Manitoba Natural Resources - Water Resources	47	This report describes the hydrology of the watershed in an attempt to better understand a more efficient method to move water and alleviate associated problems for agriculture. Amongst improvements to 1st order drainage, improvements to Boggy Creek and Brookdale drain were investigated. Drainage works had an associated cost provided in the appendix.	
77-31	1977	The Assiniboine River Degradation Investigation Downstream of the Portage Diversion	Manitoba Water Resources Branch	37	Intention of report is to determine if degradations occurring downstream of the diversion as a result of sediment being trapped upstream. In review are: 15 cross sections, bed material samples, bore home info, water surface profiles and rating curves.	Minimal discussion. Mainly a collection of data and a brief conclusion.
	2002	The Role of the Assiniboine River in the 1826 and 1852 Red River Floods	William F. Rannie, U of W	23	The paper attempts to relate the 1997 flood on the Red to the 1826 and 1852 floods. Arguments are made to the effect that the Assiniboine contributed to the 1826 and 1852 flood, although no definitive evidence is available, while it had minimal impact in 1997. Using historical meteorological data, an attempt is made to conclude that although a simultaneous peak on the Red and Assiniboine could be catastrophic, it is likely to never happen due to the weather systems that must cover each region.	
96-4	1996	Towards a Sustainable Water Use Strategy for the Assiniboine River Basin (1996)	Assiniboine River Management Advisory Board	67	The report attempts to discuss various water use sectors and allocation, while maintaining quality. These points are logically broken down and explained but useful recommendations and conclusions are not provided.	Unsure how useful this report is.
00-22	2000	Towards a Sustainable Water Use Strategy for the Assiniboine River Basin (2000)	Assiniboine River Management Advisory Board	25	Report primarily describes how the Review Board was created and what its purpose is.	unsure of its relevance
82-8	1982	Town of Souris - Flood Damage Reduction Study	Manitoba Water Resources Branch	58	Reports on studies of flood reduction alternatives. Structural options do not have adequate B/C ratios to justify, except diking of water treatment plant. Recommends building restrictions and zoning.	
98-46	1998	Upgrading of the Existing Dike Along the Carrot River and a Flood Protection Dike for the Saskatchewan Rivers, Ralls Island Area	Cochrane Engineering Ltd.	64	Seems like an RFP with limited information.	Not sure if this is useful at all.
00-50E	2000	Upper Assiniboine Basin Study - App E - GIS and Data Standards	Sask/Man/Feds	16+appendices	Recommendations on GIS	Not relevant to task of flood mitigation, but maybe affects assessments later in KGS study
00-50F	2000	Upper Assiniboine Basin Study - App F - Public Involvement Report	Clark Windsor Mills and Associates	79+figures	Report on public programs associated with study	Should be reviewed by our public involvement folks
77-13	1977	Water Demand in the Manitoba Portion of the Souris River Basin	Souris River Basin Study	123	This report provides an estimate of water use in MB portion of Souris River Basin based on 1974 survey of surface and groundwater use, future trends/demands, and several different water supply options. Within report is reference to High Souris Dam, Nesbitt Dam, Coulter Dam and Patterson Dam.	Contains cost values for each option.
55-27A	1966	Water Supply and Water Use in the Assiniboine & Qu'Appelle Basins	Prairie Provinces Water Board	90	This is an appendix to a report that addresses a variety of options for reservoirs primarily for water supply. It was written in 1966. The reservoirs described in this Appendix include Shellmouth and 10 other dam sites in the area. No b/c studies were done.	
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 1	Sask-Nelson Basin Board		This appendix provides a historical and geographical background of the Saskatchewan Nelson Basin. It describes settlement and the formation of the Water Board. In relation to 1970, it explains municipal, industrial, irrigational, hydro power and other water uses.	Provides basic logic behind the boards and other commissions set up throughout the basin.
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 2	Sask-Nelson Basin Board		This appendix provides guidelines and standards to apply to various aspects of project investigations including: Investigation Standards and Costs, Project Design Floods, Guidelines for Cost Estimating and Cost of Pumping Power. Guidelines were developed with the intention of having them applied to study, design and cost any project regardless of location.	Provides standards for hydrology calculations as well as unit prices for construction materials and methods.

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-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 3	Sask-Nelson Basin Board		Project catalogue	
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 4 Volume 1	Sask-Nelson Basin Board		This appendix is split into 3 sections: Stream flows, Evaporation, Stream flow Generation. Section A, Strream flows, explains how the estimation of existing hydro and irrigation use was combined with raw data to produce monthly stream flows at 145 locations between 192-1967. Section B explains methods used to define evaporation from 17 locations in the basin and how to interpolate between each. Stream flow Generation describes how synthetic stream flow info was produced. Generated Data in the SNBB was not used for this section.	Informative text and applicable information
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 4 Volume 2	Sask-Nelson Basin Board		Contains stream flow data assembled and computed by the SNBB. Most are monthly stream flow either recorded at gauging stations or estimates of natural flows at gauge stations or project sites.	Additional tables included which provide statistics on development of data. Entire volume is the collection of data explained in V1
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 5	Sask-Nelson Basin Board		Appendix describes methods used to perform water supply studies. Section B appears useful, containing applications and methods for conducting Multi Reservoir Analysis. Provides detailed explanations of various analysis tools.	Contains discussion on: High Speed Models, Synthetic Sequence Models, Network Analysis Models, Linear Programming Models.
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 6	Sask-Nelson Basin Board		Within the study there were 55 possible dams and 23 diversions determined and 244 combinations of each were studied. This appendix describes how the regulated flows were computed and provides the results.	Operating rules for each reservoir, amount of water withdrawn and point in the basin where regulation would be measured were determined. Assumptions for these processes are provided.
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 7	Sask-Nelson Basin Board		Environmental Considerations	
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Appendix 8	Sask-Nelson Basin Board		Contains 2 reports on the possible effects of a diversion on donor basins, particularly the Mackenzie River System and the Churchill River. Some combinations of projects saw water transferred to the Sask-Nelson basin from the Athabasca-Mackenzie River basin and Part A of this appendix addresses effects seen as a result of water being drawn from the donor basin. Part B discusses the physical analysis, economical analysis and qualitative analysis of diverting water from the Churchill River to the Nelson river.	
-	1972	Water Supply for the Saskatchewan-Nelson Basin - Main Report	Sask-Nelson Basin Board	Main Report - 210 pages	Assessment of alternative means to supplement water supplies in the Sask-Nelson basin. A variety of dams/reservoirs in our study areas were identified and analyzed for benefits from water supply.	No focus on flood control. The schemes identified will, however, be of value in identifying storage alternatives for relieving flood conditions.
58-18	1958	Water Yields and Flood Potential in the Whitemud River Basin	PFRA	34	This report provides an estimate of various tributary flows in the Whitemud River basin and inflow hydrographs at 5 proposed dam sites in the Riding Mountain Area to determine effectiveness for water supply or flood control. It was assumed that the reservoirs are used for water supply OR flood control. Complete live storage was used for both. Also provides annual discharge at various tributary locations in the Whitemud River Basin.	
	2012	Wawanesa Existing Dike Assessment	KGS Group	101	The report reviews the performance of the existing dikes within Wawanesa with respect to the 2011 flood event and whether the temporary dikes are capable to function as permanent flood protection. Seepage and slope stability evaluations are summarized and subsequent alternatives to consider are provided.	Part 2 of 3: Souris, Wawanesa, Melita
57-14	1957	Whitemud River Basin Investigation Gladstone Area	PFRA	38	This report is an attempt to alleviate flooding on the portion of the whitemud river, above Gladstone, by diverting 1/2 the flood water through a diversion into the Dead Lake Area. 5 options were reviewed.	
68-2	1968	Whitemud River Basin Planning Study Report Part 1	Water Control and Conservation	68	This report delivers a plan for flood control through a series of diversions and reservoirs to mitigate instances on the main stem of the Whitemud River. It also provides an investigation of future water requirements regarding domestic and municipal water supply. Annual cost/benefits were calculated for all and due to all results having a benefit cost ratio less than unity.	Costs are provided in the appendix

APPENDIX B2
SYNOPSIS OF REPORTS ON ROLE OF PRAIRIE WETLAND IN FLOOD ATTENUATION

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REFERENCE MATERIAL SUMMARY

This appendix contains excerpts from the various articles reviewed on the topic of wetland drainage specific to addressing the hydrologic questions “what are the benefits of wetlands in reducing flood damages and how do the benefits vary for different flood magnitudes?” To assess the benefits of wetlands, their role in flood attenuation must be understood. There have been a number of studies on peak flow reductions. Those relevant to the North American prairie wetlands are cited. These include understanding the hydrologic functions of wetland flood storage, i.e., conductivity of wetlands and the effect of storage on the contributing drainage area and storm flow modification.

Shaw, D.A, Vanderkamp, G., Conly, M., Pietroniro, A., and Matz, L., 2012, The Fill-Spill Hydrology of Prairie Wetland Complexes during Drought and Deluge, Hydrologic Processes.

Shaw et al. (2012) looked at water level data for the St. Denis National Wildlife Area, Saskatchewan, to examine the connectivity between wetlands and contributing drainage areas for runoff. St. Denis is a 3.6 sq km undrained area with more than 100 wetlands. It was observed that in dry years the downstream pond only receives local runoff, however, after successive wet years when upstream storage has been satisfied the runoff volume increased by a factor of 20. The study concluded that basin conditions such as antecedent surface storage effects the conductivity between wetlands and the surface water conductivity of wetlands that determines the percentage of the basin that can contribute to the outlet.

The study shows that after successive wet years much of the surface storage will be filled. Such antecedent basin storage conditions may result in high frequency precipitation events producing water volumes similar to a low frequency, high volume runoff event.

Shook, K. R. and Pomeroy, J. W., 2011, Memory Effects of Digressional Storage in Northern Prairie Hydrology, Centre for Hydrology, University of Saskatchewan.

Shook and Pomeroy (2011) researched the behaviour of prairie basins dominated by wetlands through two sets of simulations. The behaviour of the basin was simulated by applying precipitation and allowing it to runoff. Only direct evaporation from the water surface was computed. The first model was based on LiDAR data. The second simple model represented the wetlands based on using the frequency distribution and connectivity from the LiDAR data. Knowing the depth of water uniformly applied, and the volume running off and remaining, the fraction of the basin contributing flow was determined.

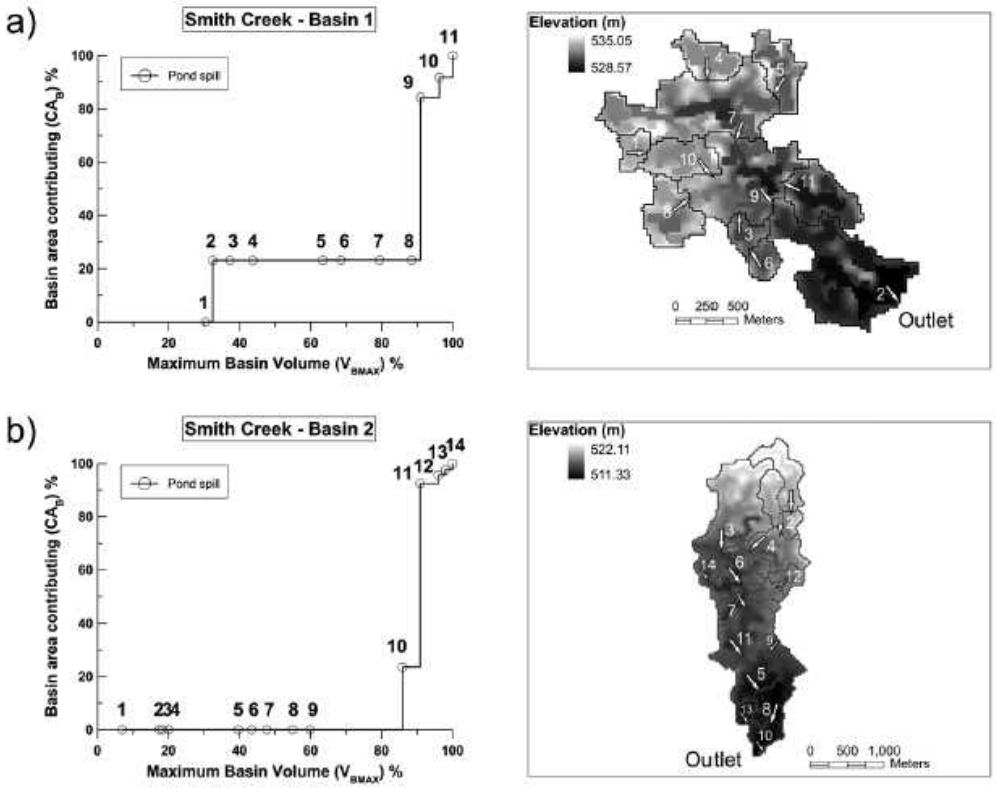
The modelling indicated the existence of hysteretic relationships between the fractional water-covered area, the fractional contributing area of a basin and the fractional water storage. The existence of hysteresis (memory of storage) is that evaporation is taken from the water surface only while runoff is delivered from the drainage area of each wetland. This memory explains that the sudden changes in contributing drainage are during low frequency events. The existence of the hysteresis indicates a hydrologic model using a single transfer function for basins affected by wetland storage does not accurately estimate the dynamic contributing drainage area. Therefore, it is nearly impossible to build a model of wetland prairie basins.

Shaw, D.A., Piteronro, A. and Martz, L.W., 2012, Topographic analysis for the prairie pothole region of Western Canada, Hydrologic Process 27.

Shaw et al. (2012) developed and tested conceptual curves for the hypothesized relationship between decreasing potential storage and contributing area. The curves reflect the fill and spill process of wetlands. The conceptual curves represent the relationship between surface storage of wetlands and contributing area in the test basins well. The benefit of this theoretical research is that it advances the idea of the variable or dynamic contributing drainage area that has been largely ignored in prairie hydrology literature.

The conceptual curves for the relationship between pre-threshold storage and contributing area illustrated the complexity of runoff in the response from a prairie pothole region. The curves reflect the fill-spill of storage wetlands. For the test basin, once 80 to 90% of the basin storage volume is filled the contributing drainage increases dramatically.

TOPOGRAPHIC ANALYSIS FOR THE PRAIRIE POTHOLE REGION OF WESTERN CANADA 3



Bullock, A. and Acreman, M., 2003, The Role of Wetlands in the Hydrologic Cycle, Hydrology and Earth System Sciences.

The paper presents 439 published statements of functions from 169 studies. From the published hydrologic evidence, there is a “difference from the long-standing generalization that wetlands always reduce floods”. Most of the studies indicate that floodplain wetlands reduce or delay floods. However, for headwater wetlands the same influence is also seen but less conclusively. For those headwater wetlands that increase the response to rainfall, it is because they tend to

be saturated and convey rainfall rapidly to the river, thus they are a principal mechanism for generation of floods. The conclusion is that, “Scientific evidence gives only limited support to the generalized model of flood control…”

Taylor, C. and Pierson, D.C., 2010, The Effect of a Small Wetland on Runoff Response during a Spring Snowmelt, Atmosphere-Ocean.

The small catchment in south central Ontario was divided into two zones: 1) the upstream wetland drainage area; and 2) the downstream side slope area to evaluate the effect of a small wetland runoff response during spring runoff in 1980. The Telford Catchment (47.5) is primarily a forested area (60%) with a perched regional water table. It was found that the majority of the runoff occurs during the spring when the water-table is high enough to produce extensive saturated surfaces. Wetlands have a regulatory effect on runoff, but they vary with antecedent conditions and storm magnitude. When wetlands are elevated water levels producing extensive saturated zones and there is little storage capacity in the wetland, it has a minimal effect on the runoff hydrograph; but the effect increases greatly when the system drains and more storage becomes available in the wetland. The study illustrates the influence that the available storage capacity of wetlands has on flood reduction ability. However, the benefits are not quantified.

Trochlell, P. and Bernthal, T., 1998, Small Wetlands and the Cumulative Impacts of Small Wetland Losses: A synopsis of the Literature, Wisconsin Department of Natural Resources Publication.

This paper discusses the values of small wetlands and the effect of their cumulative losses. Societal costs include degraded water quality, increased flood problems, etc. The loss of small wetlands should be evaluated from a cumulative impact perspective. Reference is made to a USACE study of the Charles Basin that protecting 8,400 acres of wetlands would prevent $17 million in damages (Thibodeau and Ostro, 1987). A 1981 study into streamflow changes in the Red River valley is mentioned that indicated drainage may play a role (Brun, 1981) as “One analysis shows that an increase in wetlands loss would result in about a 50% increase in predicted mean flow, a 36% increase in predicted maximum daily flow, and a 70% increase in the predicted mean during spring flow”.

Blann, K. A., Anderson, J. L., Sands, G, R. and Vondracek, B., 2009, Effects of Agricultural Drainage on Aquatic Ecosystems: A Review, 2009, University of Minnesota, St. Paul, Minnesota, USA.

This paper presents a comprehensive review of research findings for agricultural drainage on aquatic ecosystems, with specific reference to subsurface drainage. As land conversion, wetland drainage, installation of surface and subsurface drainage networks, and other modifications of terrestrial and aquatic ecosystems on behalf of agriculture tend to occur in tandem, it is difficult to separate subsurface drainage effects from these other changes. Drainage systems have reduced surface storage, increased conveyance, and increased the effective drainage area altering the dynamics of and generally increasing flows in larger streams and rivers. Cumulatively, these changes in hydrology, geomorphology, nutrient cycling, and sediment dynamics have had profound implications for aquatic ecosystems and biodiversity.

Extensive artificial drainage can significantly alter the natural hydrology of agricultural watersheds where it is used extensively. The increased frequency and intensity of flooding are

attributed drainage but are not quantified. These hydrologic changes typically occur in conjunction with other land use changes that also play a significant role.

The effects of subsurface drainage have begun to emerge but despite improved understanding, the effects of subsurface drainage are complex and vary in response to many local factors. The magnitude and direction of effects on peak flows and total runoff depends on a number of factors, including the timing and amount of precipitation, antecedent soil moisture, soil type, depth to water table, topography, and configuration of catchment drainage networks, as well as management factors such as tillage practices and drainage system design. Subsurface drainage can also create increased temporary storage capacity in the upper layer of soil, allowing water to infiltrate and spread through the soil over a longer period.

The paper concludes that aquatic ecosystems have been substantially modified and impacted by land use, and the hydrologic and water quality changes associated with the extensive development of agricultural surface and subsurface drainage.

Babbar-Sebensa, M., Barrb, R.C., Tedescoc, L.P., and Anderson, D.M., 2013, Spatial Identification and Optimization of Upland Wetlands in Agricultural Watersheds, Ecological Engineering, 52.

The study developed a watershed-scale methodology for identifying potential sites for wetlands in a tile-drained landscape and for optimizing the spatial distribution of these wetlands for reductions in peak runoff flows. The study area was Eagle Creek Watershed (ECW) located 10 miles northwest of Indianapolis, IN. Results show that a large number of potential sites could be identified (e.g., 2953 sites in ECW), and with a choice of effective wetland design parameters and with spatial optimization of their areas, locations, and drainage areas, it was possible to achieve significant peak flow reductions with fewer sites and smaller wetlands. The potential sites (e.g., 2953 sites) cover only 1.5% of the entire watershed area, but capture runoff from 29% (almost a third) of the watershed area. Saturated hydraulic conductivities of wetlands can have a significant impact on the peak flow reductions. Peak flow reduction ranged from 6.5% to 14.6% for saturated hydraulic conductivities of 0.5 mm/h to 50 mm/h.

Carter, V., 1986, An Overview of the Hydrologic Concerns Related to Wetlands in the United States. Canadian Journal of Botany, v. 64, no. 2, p. 364-374.

The paper discusses some of the hydrologic considerations related to wetlands, as well as the perceived hydrologic functions of wetlands and summarizes research needs in wetland hydrology.

Carter V., undated, Technical Aspects of Wetlands, Wetland Hydrology, Water Quality, and Associated Functions, U.S. Geological Survey.

The author notes a strong correlation exists between the size of flood peaks and basin storage (percentage of basin area occupied by lakes and wetlands) in many drainage basins throughout the United States (Tice, 1968; Hains, 1973; Novitzki, 1979, 1989; Leibowitz and others, 1992). Novitzki (1979, 1989) found that basins with 30% or more areal coverage by lakes and wetlands have flood peaks that are 60 to 80% lower than the peaks in basins with no lake or wetland area. Wetlands can provide cost-effective flood control; and in some instances, their protection has been recognized as less costly than flood-control measures such as reservoirs or dikes (Carter and others, 1979).

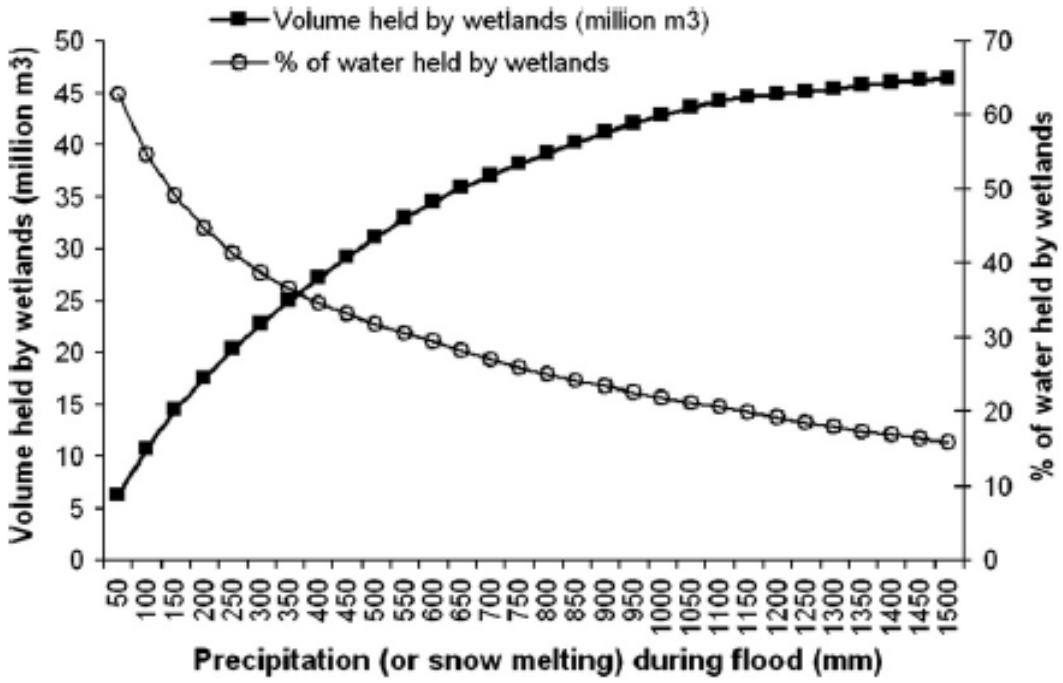
Not all wetlands are able to store floodwaters or modify stormflow; some, in fact, add to runoff. Downstream wetlands, such as those along the middle and lower reaches of the Mississippi River and its tributaries, are more effective at reducing downstream flooding than are headwater wetlands, largely as a result of larger storage capacities (Ogawa and Male, 1986). Runoff from wetlands is strongly influenced by season, available storage capacity, and soil permeability. Wetlands in basin headwaters are commonly sources of runoff because they are groundwater discharge areas.

Huang, S., Young, C., Feng, M., Heidemann, K., Cushing, W.M., Mushet, D.M., and Liu, S., 2011,Demonstration of a Conceptual Model for using LiDAR to Improve the Estimation of Floodwater Mitigation Potential of Prairie Pothole Region Wetlands: Journal of Hydrology, v. 405, no. 3-4,p. 417-426.
(Also available online at: http://dx.doi.org/10.1016/j.jhydrol.2011.05.040.)

High-resolution LiDAR data was used to estimate the flood storage capacity of wetlands within a 196 km^2 area of North Dakota. While wetlands are often viewed as isolated (closed) basins, many may connect to one another during wet conditions through the ‘‘fill and spill’’ mechanism (Winter and LaBaugh, 2003; van der Kamp and Hayashi, 2009). In this study, wetland catchments were delineated and their spill points determined. The maximum water storage capacity of individual basins was calculated and the connectivity among these basins was modelled to determine the total volume at the entire landscape level.

In total 2,245 unique basins of which 1,375 contained water were identified. The surface area of the largest water body was 849,750 m^2 and the total water surface area was around 15.03 million m^2. The existing water bodies contained approximately 32.65 million m^3 of water. The total potential flood storage of the 2,245 basins was estimated at 52.44 million m^3 which includes the existing water. When basin connectivity was considered, 413 catchment polygons were merged and the total volume increased to 80.24 million m3. The 80.24 million m^3 volume is a quantification of the maximum storage capacity of these basins and is not a quantification of how much of this maximum capacity would realistically be filled during a major flood event.

From the basin storage volume, a “Basin Floodwater Regulation Index” shown on the following figure was developed. It demonstrates the effect of wetland storage for flood events of different severities. As shown on the figure, for the basin storage to increase from 6 to 46 million m^3 requires the precipitation to increase from 50 to 1,500 mm. Above 1,000 mm, the water volume increases very slightly in response to precipitation. Thus, if the basin was extremely wet, i.e., 60% full, not a lot of precipitation is required to fill and spill the wetlands. The study illustrates the conductivity between prairie wetland and the need to simultaneously consider actual floodwater storage and the potential catchment contribution in modelling the flood reduction potential of wetlands. Modelling to date has not considered the conductivity of wetlands and may have underestimated wetland storage effects. Also, the effect of antecedent conditions on the ability of wetlands to fill and spill; the wetter the conditions, the more readily wetlands spill.



Van der Kamp, G., Stolte, W.J., and Clark, R.G., 1999, Drying out of small prairie wetlands after conversion of their catchments from cultivation to permanent Brome grass. Journal of Hydrologic Sciences, Vol 44 (3), pp 387-397.
(Available on line at: http://itia.ntua.gr/hsj/44/hysj_44_03_0387.pdf).

The study examined the effect that adopting farming practices, which represented a land use approaching 'natural or original' prairie conditions, has the potential to reduce runoff. That study looked at ten small wetlands located in the St. Denis Wildlife Area near Saskatoon. Wetland water levels had been recorded there for 30 years beginning in 1968. In the early 1980s, the uplands of some of the wetlands were converted to permanent grass cover as nesting cover for wildlife. In the initial years after conversion, higher runoff rates were observed than in the adjacent unconverted watersheds. However, after a few years, runoff rates from the converted watersheds were substantially less than those in the unconverted watersheds. This had profound effects on the hydrologic balance of the wetlands. From 1987 onwards, wetlands in the grassed areas became much drier. In fact, some wetlands dried up and remained dry even in years of heavy precipitation.

Van Der Kamp et al. were not able to identify the specific hydrologic processes that resulted in the profound change in runoff from the watershed, which had been converted to permanent cover; however, they do speculate that increased infiltration of snowmelt may be one factor. The Upper Assiniboine River Basin Study Drainage and Flood Control Committee concurred that this may indeed have been a factor, in that, after several years, a macro pore structure may have developed in the converted watersheds, which would lead to higher infiltration rates even under high snowmelt runoff conditions.

An Taisce, The National Trust for Ireland, 2012, The Use of Wetlands for Flood Attenuation, Summary Document, February 2012.
(Available on line at:
http://www.antaisce.org/Portals/5/Final%20Wetland%20Flood%20Attenuation%20report%202012.pdf).

Taisce commissioned the Aquatic Services Unit (ASU) of UCC to carry out a review of the role of wetlands in flood attenuation in Ireland entitled "The Use of Wetlands for Flood Attenuation". The report's finding were that wetlands as providing a potential ecosystem service for flood attenuation is welcomed; but it was also often exaggerated, as many wetlands only played a weak role, if at all, in attenuating floods. This is due to the diverse nature of wetlands in terms of location within a catchment, hydrological function and physical dimension. Key excerpts from the report are as follows:

The ability of a wetland to attenuate flooding depends on two critical factors: wetland storage capacity, and the wetland storage-outflow relationship (Potter, 1994). These, in turn, are affected by an array of local factors, such as climate, terrain, soil type, inflow source (surface water, groundwater, and precipitation), drainage features and not least, management of the storage-outflow relationship within the wetland. It is the water storage function of wetlands that allows for flood attenuation. The critical factor influencing whether wetlands have an impact in reducing peak flood stages is the available storage capacity at a particular time (Schultz & Leitch, 2001), plus the rate at which excess water within the wetland is shed to either a downstream surface waterbody or via subsurface drainage.

The value of such wetlands to attenuate large flood events may, however, be overstated. In a case study on the flood attenuation value of wetlands within the Red River Basin, Manitoba, Canada, Simonovic and Juliano (2001) showed that, although an increase in wetland area could potentially reduce total flood volume, the capacity of wetlands to attenuate large flood events was limited. Similarly, the results of simulation modelling in a US study found that Prairie Pothole wetlands reduced flooding for an annual event by 9-23%, compared with only 5-10% for a 100-year event (SAST, 1994, cited in Leibowitz, 2003). Bengtson and Padmanabahn (1999; cited Schultz & Leitch, 2001) found that restoration of 2,700 ha of upland depression wetlands in the North Dakota's Maple River Watershed (US) resulted in flood peak reductions of 3.8% for small events, 2.2% for medium events, 1.7% for large events and 1.6% for very large events, with the assumption of 1 foot of storage ('Bounce' refers to the available storage potential of a wetland. Wetlands can be at full capacity after heavy and/or prolonged rainfall, at which stage they would have no 'bounce'). When storage 'bounce' was increased to 2 feet the figures increased to 5.4%, 3.2%, 2.4% and 2.4%, respectively.

The ability of a wetland to attenuate floods fluctuates depending on the amount of rainfall and the flood event. As a general rule, the ability of the wetlands to attenuate floods is highest for high frequency, low to medium rainfall events that occur when there is a large capacity for water storage ('hydrological floods'). Flood attenuation ability is lowest for large events, particularly following a long period of high rainfall where the capacity for water storage is low ('economic floods', i.e., floods that potentially cause economic damage). Wetlands are more likely to attenuate hydrological floods, but less likely to attenuate economic floods. Flood height is the critical determinant of the economic cost of a flood; therefore, management of alluvial floodplains upstream of sensitive areas so as to maximize flood storage and thus reduce flood height is likely to be the most cost effective use of wetlands for flood attenuation.

Anthropogenic drainage of wetlands, unsurprisingly, can also markedly alter their storage-outflow relationship. Haan and Johnson (1968, cited in Leibowitz, 2003) found that increased drainage produced greater peak flows during long duration, low intensity rain events, but not for large volume, high intensity events. Similarly, Miller (1999; cited Shultz & Leitch, 2001) found that drainage of wetlands increased annual peak flood discharge by up to 57% during high-frequency (small) flood events but had little effect on low-frequency (large) flood events. Both drained and undrained wetlands have the capacity to store water; but because an undrained wetland empties much more slowly, it tends to store more water in a given storm event despite the potentially higher storage capacity of drained soils. This slowly-draining nature of a natural wetland also means that all of its potential storage may not be available at the time of a subsequent flood. This is especially important for large regional floods, such as those encountered in Ireland during late 2009, whereby elevated flood waters accumulated over a period of days and weeks. Analysis of the hydrological conditions that have previously given rise to severe flooding in the Tolka River, North County Dublin, showed that the conditions for such flooding occurred during winter, when heavy rain in previous days and weeks led to saturated conditions and were then followed by a sustained severe rainstorm event of around 48 hours duration (OPW, 2005). Under such conditions, most of the soils of catchment and wetland areas are fully saturated. When the volume of wetland storage is too small compared with the volume of flood entering, the peak discharge remains unaffected, i.e., when wetlands are “full”, there is little or no attenuation effect.

Much of the confusion about the value of wetlands for flood attenuation is probably owing to their highly variable nature. Not only do they consist of several major and multiple minor types of habitat, based on their hydrology, vegetation and location, but there is great physical and hydrologic variation among individual wetlands of the same type (Cole et al., 2003). The major divisions are related to their hydrological properties and broadly distinguish between: (1) wetlands in the lower part of a catchment dependent on water from parent water bodies such as rivers, lakes and coastal waters; and, (2) wetlands in the upper headwater parts of a catchment, which are independent of any water body (Keddy, 2000; Regan & Johnston, 2011).

Individual wetlands probably have quite limited storage and attenuation potential on their own, but to illustrate the potential impact that isolated wetlands in a landscape may collectively have on flood attenuation, three examples are sited:

(1) The increased severity and frequency of flooding at Devils Lake, North Dakota, prompted an investigation into the level of flood storage that could be gained by restoring the wetland resource within the lake’s various sub-catchments. The region is characterized by many isolated groundwater wetlands, many of which have been drained for conversion to agriculture. Modelling estimated the storage potential in: (1) possibly intact/undrained wetlands; and, (2) possibly drained wetlands. A number of scenarios were modelled, such as gains in storage capacity derived from restoring 25%, 50%, 75% or 100% of drained wetlands. The runoff reduction estimates based on possible storage capacity increases indicated that depression wetland restoration could reduce the volume of runoff entering Devils Lake and contribute to reduction of flood risk (Doeing & Forman, 2001). Despite limitations within the model, the study shows the potential for a collective, isolated-wetland resource to contribute to flood risk management.

(2) Lane & D’Amico (2010) used remotely-sensed Light Detection and Ranging (LiDAR) data to estimate potential water storage capacity of isolated wetlands in north central

Florida. They also modelled the water storage potential of what they highlighted as >8500 polygons identified as isolated wetlands and found that, collectively, they stored 1619 m^3/ha on average, with a median value of 876 m^3/ha. They discussed the difficulty of quantifying wetland ecosystem services, especially the difficulty of scaling the loss of numerous individual wetlands to effects at the catchment scale. Their results could be used to improve watershed models that incorporate isolated wetland recharge, discharge, and flow-through hydrodynamic processes, which may enable estimation of the potential attenuation role of such wetlands.

(3) Schultz and Leitch (2001) simulated restoration of depression wetlands in the Red River Catchment, North Dakota. They found that wetland restoration could improve flood attenuation at a catchment scale, but that wetland restoration was not a cost effective solution given the scale of the flooding problem (see Section 8.3 for cost effectiveness in relation to this example).

Conclusions:

- The review has led us to make a distinction between 'hydrological' floods (high frequency, low to medium rainfall events that occur commonly without economic damage) and 'economic' floods (low frequency events following high intensity rainfall, potentially causing economic damage). Wetlands may readily attenuate 'hydrological' floods but may fail to attenuate 'economic' floods. The literature does not distinguish between these events and tends to group 'floods' into a single category.

- In general, the influence of wetlands in reducing flood peaks is greatest for high frequency, low to medium rainfall events that occur when wetlands have a large capacity for storage. It is least for large events when soil and wetland storage are saturated in advance.

- Most evidence of flood plain attenuation effects are derived from modelling rather than empirical studies. Factors influencing the flood attenuation properties of floodplains include: surface water levels, contribution of hillslope flow, soil moisture deficit, topography (particularly surface depressions), antecedent rainfall conditions, surface vegetation characteristics, drainage characteristics and the degree of connectivity between channel and floodplain.

Recommendations:

- Experimental studies that measure attenuation effects for different wetland types are required in Ireland, especially on peatlands. The OPW Flood Studies Update (FSU) is investigating floodplain attenuation effects through hydrological simulation, but little is known about runoff generation and attenuation on Irish peatlands.

- It may be useful to study full hydrometric datasets in combination with land use change statistics across different catchment types in Ireland to help determine the impact of catchment land use change on flooding. This may help target where land management changes can influence water retention in the future.

- Given the highly variable nature of wetlands, even within each wetland type, considerably more study is needed to develop a rapid and cost-effective method to estimate the attenuation potential of individual wetlands.

Manitoba Conservation, Environment Canada, Sask Water, 2000, Upper Assiniboine River Basin Study, Main Report.
(Available on line at:
http://www.gov.mb.ca/waterstewardship/reports/planning_development/uarb_report.pdf).

The upper Assiniboine River Study was undertaken because of concerns about the alleged contribution of agricultural drainage to the 1995 flood. The 1995 flood was estimated as a one in 100-year event. Of particular interest was the drainage of wetlands associated with agricultural development and practices in the basin.

The Semi-distributed Land Use-based Runoff Processes (SLURP) hydrologic model was successfully applied in modelling flows for larger watersheds of the upper Assiniboine River Basin with 1995 to 1997 land cover. However, it was not possible to determine the effect on basin hydrology of changes to land use including artificial drainage using the SLURP model. However, the study concluded that the high flows in 1995 could be attributed to a combination of meteorological events. Runoff from high precipitation in the fall of 1994 had nearly filled many of the natural wetlands and soil moisture levels were high, reducing the natural storage available for spring snowmelt runoff. There was an above-normal snowpack followed by a mid-April rainstorm, which contributed additional runoff water and caused rapid melting of the snowpack. This rapid melt across much of the basin resulted in an unusual hydrologic condition, the nearly coincident peak flows on the Assiniboine and Whitesand rivers at Kamsack, as well as on smaller local tributaries between Kamsack and the Shellmouth Dam. To compound the situation locally, high levels in the receiving streams reduced or prevented flow from many drainage ditches. It was felt that “drainage works in the basin did not contribute significantly to the observed 1995 flood peaks and volumes. These can be attributed to natural conditions”.

Although the effects of wetland drainage could not be modelled, the principal study conclusions relevant to the hydrologic impact of agricultural wetland drainage were:

- Climatic processes dominate the generation of major flood event.
- The greatest relative impact on the drainage of wetlands is during moderate runoff events, with the effect diminishing for increasingly larger flood events.
- Drainage of wetlands has little effect on major flood events.
- Effects of land cover changes and wetland drainage in the basin could not be quantified, but a trend of increasing runoff volume was detected.
- Increased runoff volumes into lakes gives rise to management issues and reduces operational flexibility of reservoirs.

Yang, W., Wang, X. Gabor, S., Boychuk, L. and Badious, P., 2008, Water Quantity and Quality Benefits from Wetland Conservation and Restoration in the Broughton’s Creek Watershed. (Available on line at: http://www.ducks.ca/assets/2012/06/broughtons.pdf).

This study uses the ‘‘hydrologic equivalent wetland’’ concept in the Soil and Water Assessment Tool to examine the effects of various wetland restoration scenarios on stream flow and sediment at a watershed scale. The modeling system was applied to the 25,139 ha Broughton’s Creek watershed in western Manitoba. Canada. As a representative prairie watershed, the Broughton’s Creek watershed experienced historic wetland losses from 2,998 ha in 1968 to

2,379 ha in 2005. Modelling showed that, if wetlands in the Broughton's Creek watershed could be restored to the 1968 level, the peak discharge and volume could be reduced by 23.4% (Yang, 2010) and 62% at the watershed outlet.

(**http://www.ducks.ca/search-results/?cx=003672755779210886391%3Aq025ggogluc+&cof=FORID%3A11&q=broughton**)

The modelling supports that wetland drainage has an impact on low or moderate frequency flood events.

Ludden, A. P., Frink, D.L. and Johnson, D.H., 1983, Water Storage Capacity of Natural Wetland Depressions in the Devils Lake Basin of North Dakota, Journal of Soil and Water Conservation, 38.1.

Photogrammetric mapping techniques were used to derive the water storage capacities of natural wetland depressions other than lakes in the Devils Lake Basin of North Dakota. Results from sample quarter-section areas were expanded to the entire basin. Depressions in the Devils Lake Basin have a maximum storage capacity of nearly 811,000 cubic decameters (657,000 acre-feet). The depressions store about 72% of the total runoff volume from a 2-year frequency runoff and about 41% of the total runoff volume from a 100-year frequency runoff. The study did not examine the role of evapotranspiration, climate, and antecedent moisture in the annual available storage.

The actual amount of wetland storage available in the Devils Lake Basin has been a matter of some debate in recent years. A brief summary of the studies conducted by various agencies or teams of agencies examining the acreage, and storage potential of wetlands in the Devils Lake basin (available on line at: **http://www.swc.nd.gov/4dlink9/4dcgi/GetSubContentPDF/PB-2059/UpperBasinStorageStudies.pdf**).

Ogawa, H., and Male, J.W., 1986, Simulating of Flood Mitigation Role of Wetlands: Journal of Water Resources Planning and Management, v. 112, no. 1, p. 114-127

A simulation methodology was developed for evaluating the flood mitigation potential of inland wetlands. The methodology accommodates widely used computer models with generally available input data. Being a watershed simulation approach, the methodology can assess the potential for downstream flooding resulting from a reduction in upstream wetland storage capacities. The wetland encroachment scheme resembles the well-tested method of flood insurance studies and allows partial filling of a wetland. Simulations were performed for different antecedent moisture conditions, rainfall intensities and degrees of wetland encroachment. Results showed that encroachment on less than 25% of a wetland area would have only minimal impact on peak flows. In addition, results showed that downstream main-stem wetlands are more effective in reducing downstream flooding than upstream wetlands.

Downstream wetlands, such as those along the middle and lower reaches of the Mississippi River and its tributaries, are more effective at reducing downstream flooding than are headwater wetlands, largely as a result of larger storage capacities.

Doeing, B.J. and Forman, S.M., 2001, Final Report: Devils Lake Upper Basin Storage Evaluation, Prepared by West Consultants Inc., for United States Army Corps of Engineers.
(Available on line at: http://www.swc.state.nd.us/4dlink9/4dcgi/GetContentPDF/PB-517/Wet_Storage_Final.PD).

The study investigated the impacts of upper basin storage restoration alternatives on the inflows to Devils Lake. The study focused on the change in annual runoff volumes and did not consider changes in annual peak flow rates. The results of this study indicate that depression restoration can reduce the volume of runoff entering Devils Lake from the 2,616 square mile upper basin watershed.

Because of these limitations and the difficulties of HEC-HMS, a custom hydrologic model, the Pothole-River Networked Watershed Model (PRINET) was developed to simulate the depression storage, soil storage, and runoff in the Devils Lake Basin. Different levels of restoration 25%, 50%, 75%, and 100% by volume were analyzed. A total of 13,464 wetlands having a total surface area of 79,762 acres and a total volume of 127,835 acre feet were considered.

The modelling indicated that the annual flow reductions resulting from depression restoration vary significantly for individual water years. In dry years, the percent of flow reduction is larger than in wet years. The average annual runoff reduction is less than the restored volume. The average annual volume captured was consistently about 25% of the restored volume across the different restoration scenarios. (For example, for the 25% restoration level the average runoff reduction is 6,826 acre-ft. Since 19,472 acres were restored, this yielded 6,826 acre-ft / 19,472 acres = 0.3506 feet = 4.21 inches.)

The results of this study indicate that depression restoration will reduce the volume of runoff entering the lake and reservoirs. The runoff reduction values reported in this study were considered to be conservative.

Bengtson, M.L. and Padmanabhan, G., 1999, Hydrologic Model for Assessing the Influence of Wetlands on Flood Hydrographs in the Red River Basin Development and Application, Department of Civil Engineering and Construction, North Dakota State University (Available on line at: http://www.ijc.org/rel/pdf/wetlandndsu.pdf).

Following the 1997 flood in the Red River Valley, there was a compelling need to investigate the impact of wetlands on flooding in the valley on a scientific basis. To quantitatively assess the effectiveness of wetlands on peak flow rates two watersheds, one on the North Dakota side; and the other on the Minnesota side, the Maple River and Wild Rice River watersheds were modelled using HEC-1.

The HEC-1 model was chosen for the study basically because it offers the most of the methods for simulating the major hydrologic processes occurring during a flood event without the complications of involving those hydrologic processes, which may not have significant effects during floods. The flood storage capacity of wetlands was modelled using diversions. The total amount of diversion was set to equal the storage capacity of the wetlands. Once the wetland storage capacity was satisfied, all remaining flow was routed downstream.

Each watershed has a drainage area of approximately 1,600 square miles. In the Maple River watershed there are approximately 2,187 drained wetlands with an estimated storage volume of 2,500 to 3,000 ac-ft. In the Wild Rice River watershed, there are approximately 4,145 drained wetlands with a storage volume of 14,100 to 28,100 ac-ft. The drained wetlands represent about 0.25% of the Maple River watershed by area, but 1.3% in the Wild Rice River watershed.

The study noted that the average bounce or depth of the wetlands was about 1.1 feet. Therefore, this volume was available at the beginning of the simulated flood event. It was felt that this was a generous assumption as for spring snowmelt floods; all this storage might not be available if the preceding fall was extremely wet. The results of modelling showed that the maximum reduction in flood stage was about 3% for a 10-year storm. For a 100-year event, the reduction was less than 0.4%. For a 4 foot bounce, the flood stage was reduced by about 10% for the 10-year event, but only 1.26% for the 100-year event. Changing the timing of diversions did not significantly affect the results. The reductions were expressed in the reduction of stage as it was felt simply comparing the change in peak flow rate would be misleading and it was more meaningful to compare changes in stage due to reductions in peak flow. For the 100-year event, the reduction in flow was 2.7%. The percentage reduction in stage is likely smaller than the 100-year event flood where the flow is spread out on the floodplain.

To investigate the effect of much greater wetland storage, the model was run with 1%, 2.5% and 5% restored wetlands. The best results were for the higher-frequency events. With 1% restored wetlands by area, the flood stage was reduced by nearly 19%. With 5% wetlands, the flood stage was reduced by 34%. For a low-frequency event such as the 100 year storm, storage from 1% restored wetlands reduced the flood stage by less than 0.9%. For 5% restored wetland, the flood stage was reduced by 8.4%. This case represents nearly 20 times the existing drained wetlands by area.

The authors concluded that “Based on the results of this study, restoring drained wetlands could not have significantly affected the flood stage for a flood in the order of the 1997 Red River Valley Flood in the Maple River watershed. Even increasing the additional wetland storage represented by restoring drained wetlands by four times did not result in a significant reduction in flood stage. Also, wetlands were assumed empty at the time of the flood event, which may not be the case. The findings from the Miller study (Miller, 1999) support these results. For flows in the order of magnitude of a 100-year event, even 40% wetlands by area did not have a significant effect. Restoring drained wetlands is highly unlikely to significantly affect the low-frequency flood events such as the Red River Valley flood of 1997”.

Miller, R.C., 1999. Hydrologic Effects of Wetland Drainage and Land Use Change in a Tributary Watershed of the Minnesota River Basin: A Modeling Approach. M.S. Thesis, University of Minnesota. U.S.A.

A study was performed to determine the hydrologic effects of wetland drainage and land use change in the Little Cobb River watershed which is tributary of the Minnesota River. This 130 sq mi watershed had undergone extensive drainage. The watershed was modelled with the Hydrologic Simulation Program-FORTRAN (HSPF), a continuous water-balance model, for a period of 43 years. Ten percent and 40% wetlands by area were assumed. Flow duration curves were developed for the daily flows from the simulations. These flood frequency curves converged for the large magnitude, low-frequency events even with 40% wetlands.

Simonovic, S.P. and Juliano, K.M., 2001, The Role of Wetlands during Low Frequency Flooding Events in the Red River Basin, Canadian Water Resources Journal, 26:3, 377-397. (Available on line at: http://www.ijc.org/rel/pdf/wetlandsman.pdf).

Through a case study, the role that wetlands play in low frequency floods, using the Rat River watershed in southeastern Manitoba was investigated. The study was a result of questions raised due to the 1997 Flood. The Rat River watershed with an area of approximately 1,550 km^2 is located approximately 30 km southeast of Winnipeg. The river flows into the Red near St. Agathe. The watershed wetlands compose 3% of the land area.

The hydrologic model HEC-HMS was used for the 1997 flood event. Three scenarios, with increasing wetland areas of 5%, 8%and 13% were considered. The results based on the 1997 flood were:

- The 2% increase in wetland area (46.04 km^2). Spring flow started on April 19, rather than April 18. The peak of the hydrograph remained at approximately 752 m^3/s on April 23 with an increase flow volume of 2.2%.
- The 5% increase in wetland area (73.66 km^2). Spring flow started on April 20, rather than April 18. The peak of the hydrograph remained at approximately 752 m^3/s on April 23, with an increase flow volume of 5.6%.
- The 10% increase in wetland area (119.704 km^2). Spring flow started on April 21, rather than April 18. The peak of the hydrograph remained at approximately 752 m^3/s on April 23, with an increase flow volume of 11.1%.

The results show that for a low frequency flood event, the reduction in water volume is minimal in comparison to the amount of land to be restored; and the reduction in the flood peak is non-existent. With the percent increase in wetland area, and the total flood volume reduction for 1997, the results seem to be proportional.

The authors stated that, “The results seen are for the Rat River watershed. Increasing wetlands by the percentages indicated in this report would likely not have any effect on the total flood volumes for the Red River during a low frequency flood. That is, given the highest percent increase in wetland area, and its corresponding total volume reduction, the decrease in damages experienced would be very modest if extrapolated to the entire Red River Basin. If wetlands were increased by 100% in all watersheds in the valley, a decrease in total flood volume of 11.1% during a low frequency flood event still is not significant”. It was noted that the “largest flood in the history of the Red River Valley occurred in1826, prior to basin development and prior to any drainage activities”>

Shultz, S.D., 1999, The Feasibility of Wetland Restoration to Reduce Flooding in the Red River Valley: A Case Study of the Maple (ND) and Wild Rice (MN) Water, Department of Agricultural Economics, North Dakota State University Agricultural Economics Report No. 432. (Available on line at: http://www.ijc.org/rel/pdf/wetlandsschultz.pdf).

The economic feasibility of wetland restoration and the use of impoundments to store water and reduce flood damage were evaluated in two Red River Valley watersheds: the Maple River Watershed in North Dakota, and the Wild Rice River Watershed in Minnesota. Benefits were based on the hydrologic modeling estimates of reductions in peak flood stage with alternative

storage scenarios and flood events in the Maple River Watershed (Bengtson and Padamanabahn, 1999).

Wetland restoration was not found to be economically feasible in reducing flood damage in either of the two watersheds. Reductions in peak flood stage and damage would need to be reduced from 3.4% to 10.2% for different wetland restoration options to break even.

The use of impoundments was also found not to be an economically feasible method in reducing flood damage. Future research is warranted with respect to the use of impoundment-storage for flood control on a site-specific basis and the calculation of the non-flood-related benefits of restored wetlands.

AECOM, 2009, Fisher River Study, Hydrodynamic Model and Economic Analysis Study for the Government of Manitoba, (Available on line at: http://www.gov.mb.ca/waterstewardship/reports/planning_development/rpt_f685_002_04_fisher_river_final_090605_elb.pdf).

The study examined the causes of flooding and estimated the contribution to flooding from historic land use changes and extension of the drainage network since the mid-1960s in the Fisher River Watershed. The 2,200 km^2 watershed located in the Interlake Region of Manitoba flows northward and eventually drains into Lake Winnipeg at Fisher Bay. Since the mid-1960s the watershed has experienced an extensive expansion of the drainage network in the upper reaches. The First Nations believe that drainage improvements and land clearing in the upstream reaches have contributed to their flooding problems.

HEC–HMS hydrologic and MIKE II hydrodynamic models were developed for the watershed. The hydrologic model generated runoff hydrographs from 148 catchments reflecting land use and drainage conditions existing in 1966 and 2002. Water bodies were found to reduce from 16.6% in 1966 to 14.0% in 2002. The loss in wetland area was accounted for and the aggregate CN computed for each catchment. Inflow hydrographs representing upstream and lateral inflows were routed through a network of streams that existed in 1960s land use.

The Mike 11 was used to physically route the flows through the complex system of main channel, tributaries, branches and drainage works. The hydrodynamic model included natural constrictions and man-made obstructions such as bridges, culverts and low level crossings. The pre-1966 model treated the recently developed drains as unimproved shallow swales with increased roughness consistent with floodplains. For the 2006 flood, about a one in 5-year event, the following impacts at the downstream end of watershed (Dallas gauge) were observed: a 5% increase in volume, a 2.9% increase in mean daily peak flow and a 1.4% increase in instantaneous peak flow due land use changes since the 1960s.

For the 100-year flood the following impacts at the Dallas gauge were observed: a 4% increase in volume, a 5.5% increase in mean daily peak flow and a 4.4% increase in instantaneous peak flow due to land use changes.

The impacts were much more pronounced in the upper and middle reaches of the Fisher River. Peak flows were found to increase in the range of 5 to 12%. This was because of timing of upland drainage reaching the downstream locations, which was after the downstream runoff and the attenuation of the flood peak because of the throttling effect of existing culverts and floodplain storage.

The study illustrates that the location of wetlands within a watershed affects their ability to control floods. The hydraulic capacity of the downstream receiving water courses and waterway crossings also needs to be considered.

Schottler, S.P., Ulrich, J., Belmont, P., Moore, R., Lauer, J.W, Engstrom, D.R. and Almendinger, J.E., 2013, Twentieth Century Agricultural Drainage Creates More Erosive Rivers, Hydrological Processes.
(Available on line at:
http://www.lakepepinlegacyalliance.org/wordpress/wp-content/uploads/2013/01/HP-Schottler-et-al-1.pdf).

The study compared changes in precipitation, crop conversion and extent of drained depressional area in 21 Minnesota watersheds over the past 70 years. A comparison of periods 1940 to 1974 and 1975 to 2009 indicated that more than one half of the watersheds had increases in water yields in the later period. The most significant increase was observed in the spring (May to June) with a much smaller increase in the fall (September to October). The seasonal and annual water yields and runoff were found to increase by > 50% since the 1940s on half the watersheds, with no statistical change in others. Climate and crop conversion explained less than one half of the increase in river flow. Artificial drainage was identified as the largest driver of the increase in flow, exceeding the effects of precipitation and crop conversions. The majority of the increase in flow was attributed to changes in water residence time on the landscape and subsequent reductions in evapotranspiration from the drainage of wetlands.

The study only focused on the effect of drainage on seasonal and annual runoff yields. No mention is made to the impact of wetland drainage in flood attention effects.

Schultz S.D, and Leitch J.A., 2001, The feasibility of restoring previously drained wetlands to reduce flood damage, Journal of Soil and Water Conservation.

The economic feasibility of restoring previously drained wetlands to reduce flood damage was evaluated in North Dakota's Maple River Watershed, a sub-watershed of the Red River Valley of the North. Benefit-cost ratios ranging from 0.2 to 0.7 were not adequate to make simple wetland restoration - based on plugging existing drains, restoration with outlet control devices, or complete restoration intended to provide a full range of wetland-based environmental services - economically feasible over a 20-year future period. Peak flood stage and flood damage would need to be reduced by 3.3% to 8% for wetland restoration options to break-even and the inclusion of likely non-flood related wetland benefits did not improve feasibility. It is not recommended that public funds be used for extensive wetland restoration projects throughout Maple River Watershed or across large areas of the Red River Valley for the purpose of reducing flood damage.

Pomeroy, J., Westbrook, C., Fang, X., Minke, A. and Guo, X., 2010, Prairie Hydrological Model Study Final Report University of Saskatchewan Centre for Hydrology, Saskatoon, (Available on line at: http://www.usask.ca/hydrology/Reports.php).

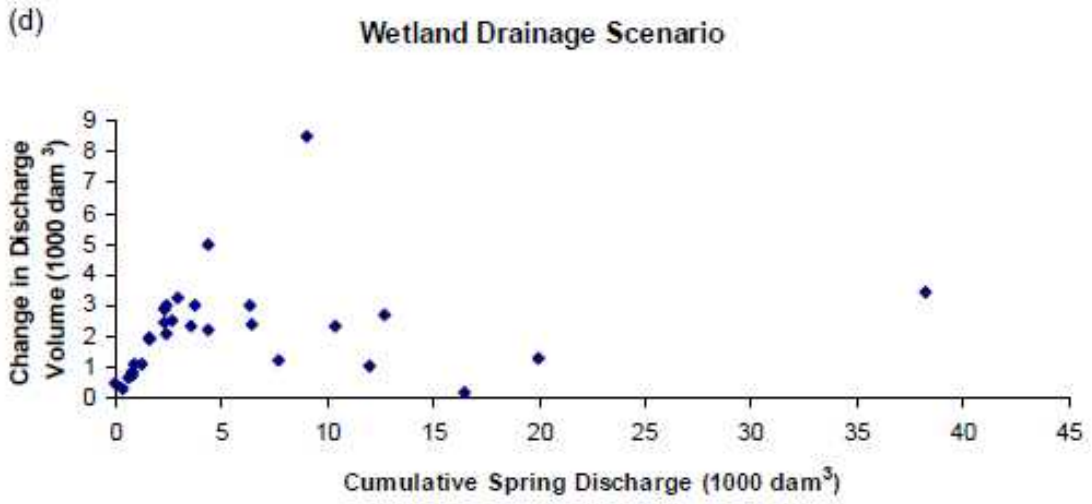
The Smith Creek watershed was studied to understand the effects of land use changes and drainage on eastern prairie hydrology, flooding and drought, and to develop a hydrological model for predicting these changes. The watershed located in eastern Saskatchewan is within the upper Assiniboine River Basin. The approximately 435 km^2 watershed has undergone rapid

changes over the past 50 years, transitioning from areas of extensive woodland and wetlands to grain and oilseed cultivation. There has been a reduction in wetland areas from 17% to 9% during 1958 to 2001.

In 2010, the University of Saskatchewan developed a Prairie Hydrological Model (PHM). The model was used to estimate the sensitivity of streamflow to changes in drainage and land use. The Prairie Hydrological Model incorporated all major prairie hydrological cycle components such as wetland storage and runoff generation mechanisms to be capable of addressing the influences of changing land use, wetland drainage and climate variability. A key component of the model was wetland modules that permitted open water evaporation and fill and spill from wetlands. In the model, hydrological processes were simulated on landscape units called hydrological response units (HRU). HRUs are landscape units, within which processes and states are represented by single sets of parameters. HRUs can be finely scaled (hillslope segment), or coarsely scaled (sub-basin). HRUs in the prairies typically correspond to agricultural fields (stubble or fallow fields), natural cover (grassland or forest woodland), and bodies of water (lake or pond).

Streamflow simulations were carried out to examine the sensitivity of the spring runoff volume to the changes in land use and drainage for the period 1965-82 and 1993-2005. Two types of scenarios were undertaken: a land use scenario and a drainage scenario. The land use examined a ‘forest cover’ scenario, which involved the conversion of fallow and stubble fields to tree woodland and an ‘agricultural land use’, which involved conversion of grassland and tree woodland to fallow and stubble. The drainage scenarios examined a ‘restoration’ scenario that converted fallow and stubble to wetland and open water at the 1958 level with the 1958 drainage network. The ‘wetland drainage’ scenario focused on the historic loss of wetlands and open water to increase fallow and stubble fields.

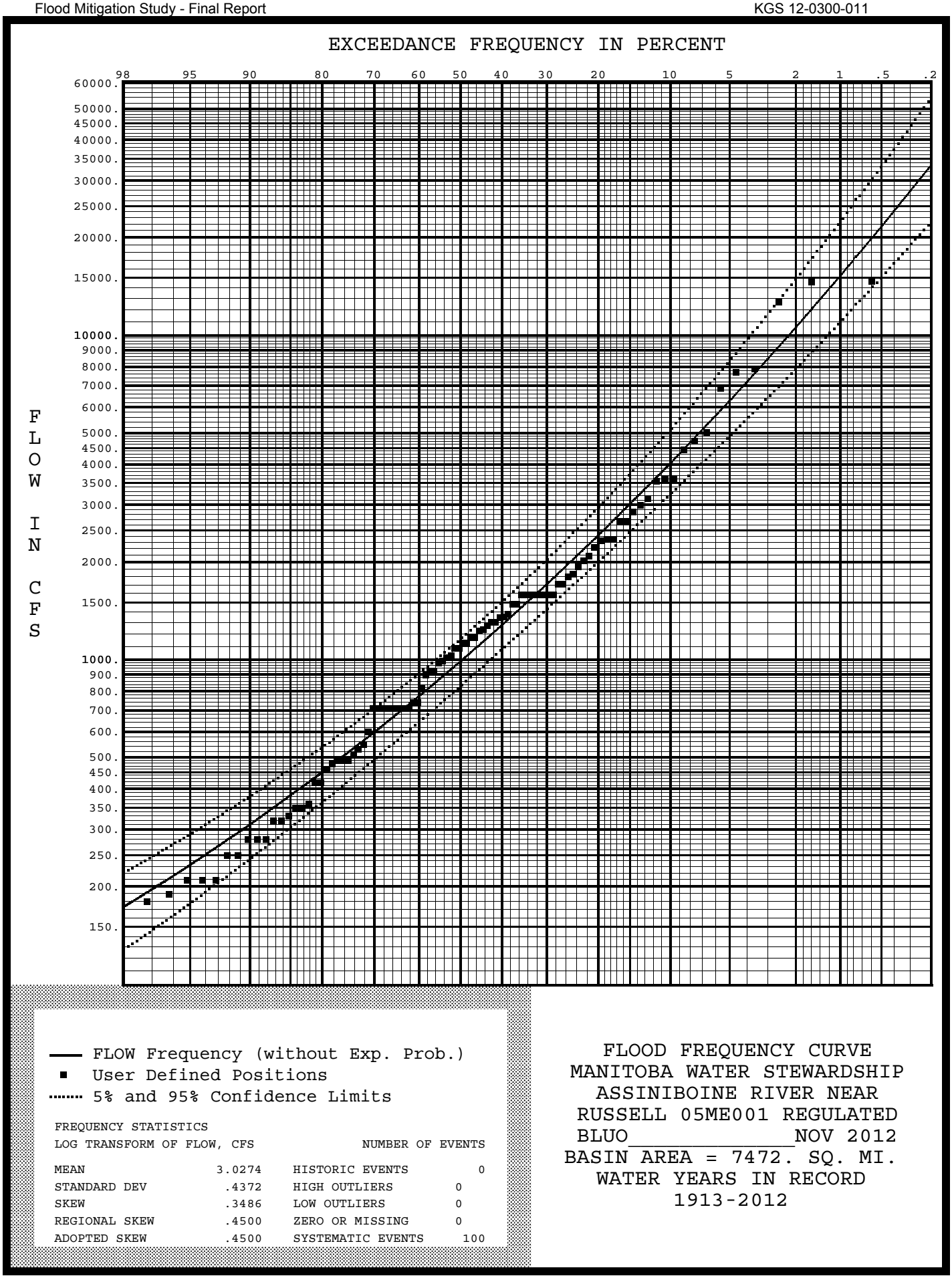
The drainage scenarios showed very dramatic changes in the spring discharge volumes. The spring volume decreased with wetland restoration and increased with wetland drainage. The greatest impact for both scenarios occurred for moderate spring volumes (around 10,000 dam^3). The volume was reduced by 6,000 to 8,000 dam^3 when wetland storage was restored, whereas it increased by 3,000 to 8,000 dam^3 when wetlands were drained. The changes were moderate for higher flows, likely due to the limitations of available storage changes in response to drainage changes. Overall, the long-term impact of drainage change during the 29-year period lead to a 45% drop in the total basin spring volume for wetland restoration and a 36% rise for wetland drainage. For comparison, converting cropland to forest had very small effect (an average 1% increase), while the effect of conversion of woodland to agricultural land use had a moderate increase (19% increase).



The simulations indicated that the peak daily discharge tended to increase for forest conversion, agricultural conversion, and the wetland drainage scenarios and decreased for the wetland restoration scenario. The reason for the increases in the peak discharge was the hydrograph under this conversion tends to become sharper and less attenuated with the increased volume. Conversely, wetland restoration resulted in reduced peak spring discharge due to the increased wetland capacity for storing runoff and subsequent decreasing of the spring volume. The effects of restoring wetland on decreasing the peak discharge were overwhelming for the moderate and high flows. Interestingly, removing wetland and increasing channel drainage reduced the peak daily discharge for the moderate flow. This was attributed to the fact that the length of the channel increased and, consequently, took a longer time for streamflow to travel to the basin outlet.

APPENDIX C

FREQUENCY ANALYSIS

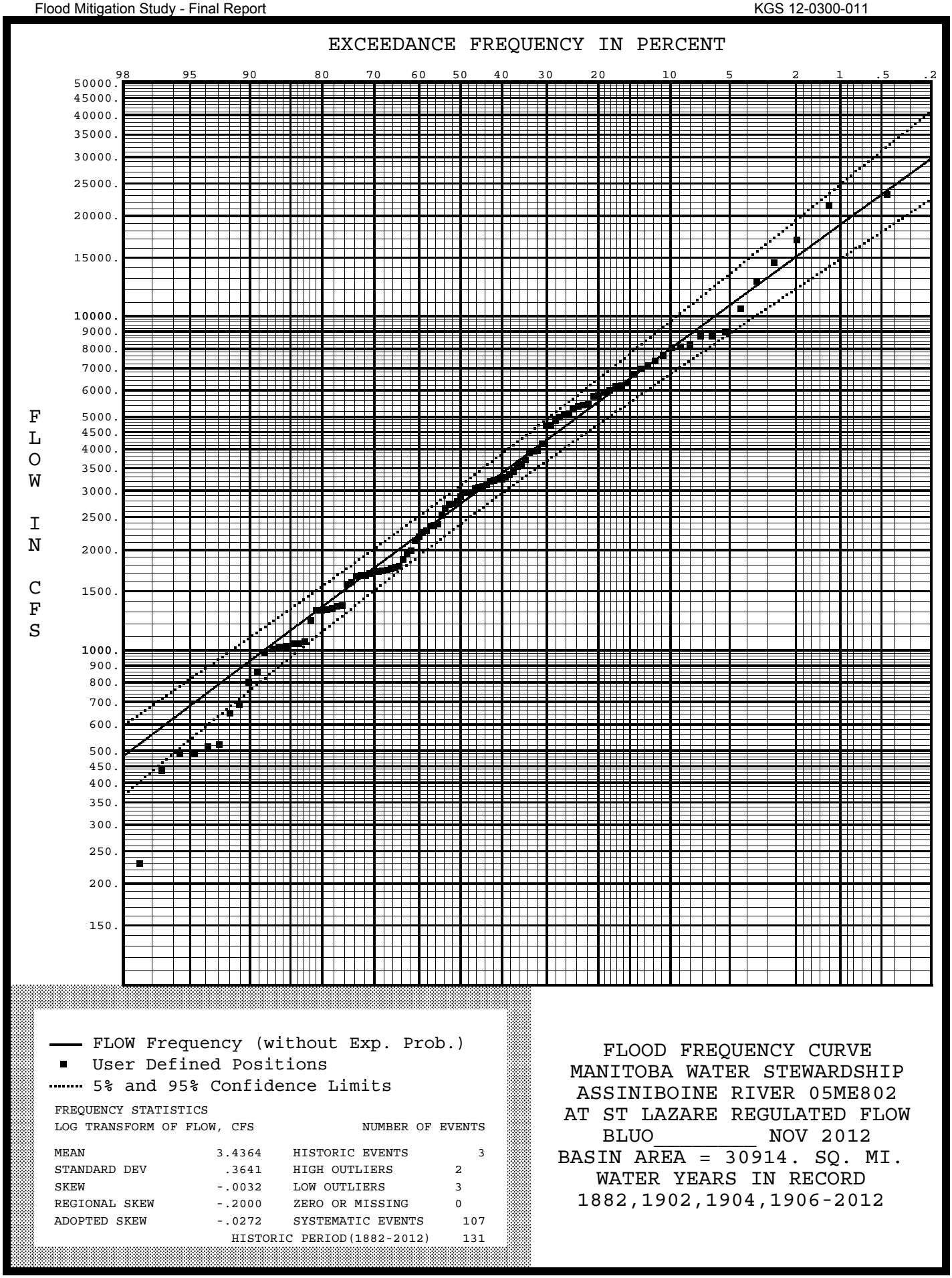


FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

MEAN	3.0274	HISTORIC EVENTS	0
STANDARD DEV	.4372	HIGH OUTLIERS	0
SKEW	.3486	LOW OUTLIERS	0
REGIONAL SKEW	.4500	ZERO OR MISSING	0
ADOPTED SKEW	.4500	SYSTEMATIC EVENTS	100

NUMBER OF EVENTS

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
ASSINIBOINE RIVER NEAR
RUSSELL 05ME001 REGULATED
BLUO_____NOV 2012
BASIN AREA = 7472. SQ. MI.
WATER YEARS IN RECORD
1913-2012



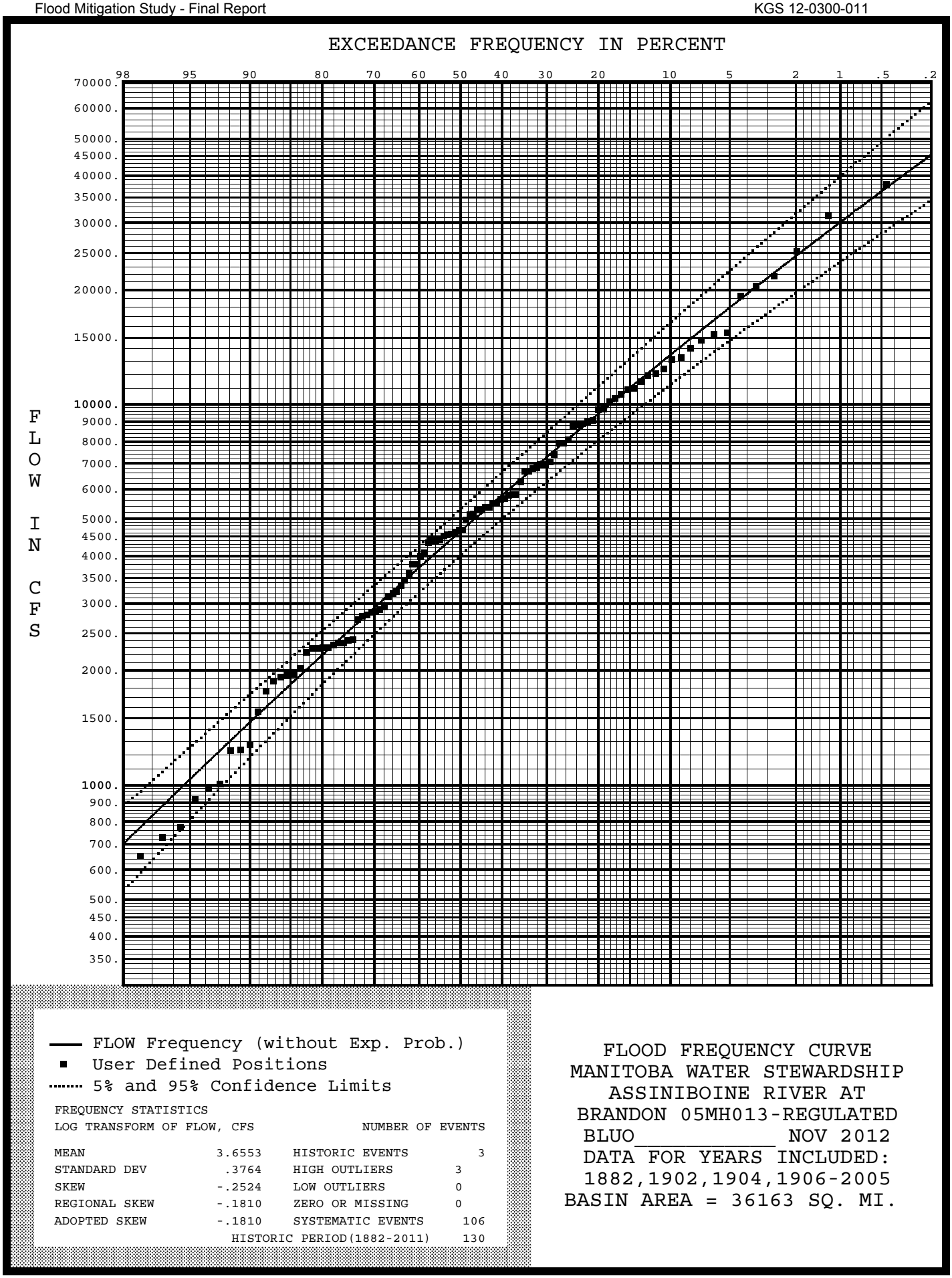
FLOW Frequency (without Exp. Prob.)
User Defined Positions
5% and 95% Confidence Limits

FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

MEAN	3.4364	HISTORIC EVENTS	3
STANDARD DEV	.3641	HIGH OUTLIERS	2
SKEW	-.0032	LOW OUTLIERS	3
REGIONAL SKEW	-.2000	ZERO OR MISSING	0
ADOPTED SKEW	-.0272	SYSTEMATIC EVENTS	107
		HISTORIC PERIOD(1882-2012)	131

NUMBER OF EVENTS

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
ASSINIBOINE RIVER 05ME802
AT ST LAZARE REGULATED FLOW
BLUO_____ NOV 2012
BASIN AREA = 30914. SQ. MI.
WATER YEARS IN RECORD
1882,1902,1904,1906-2012

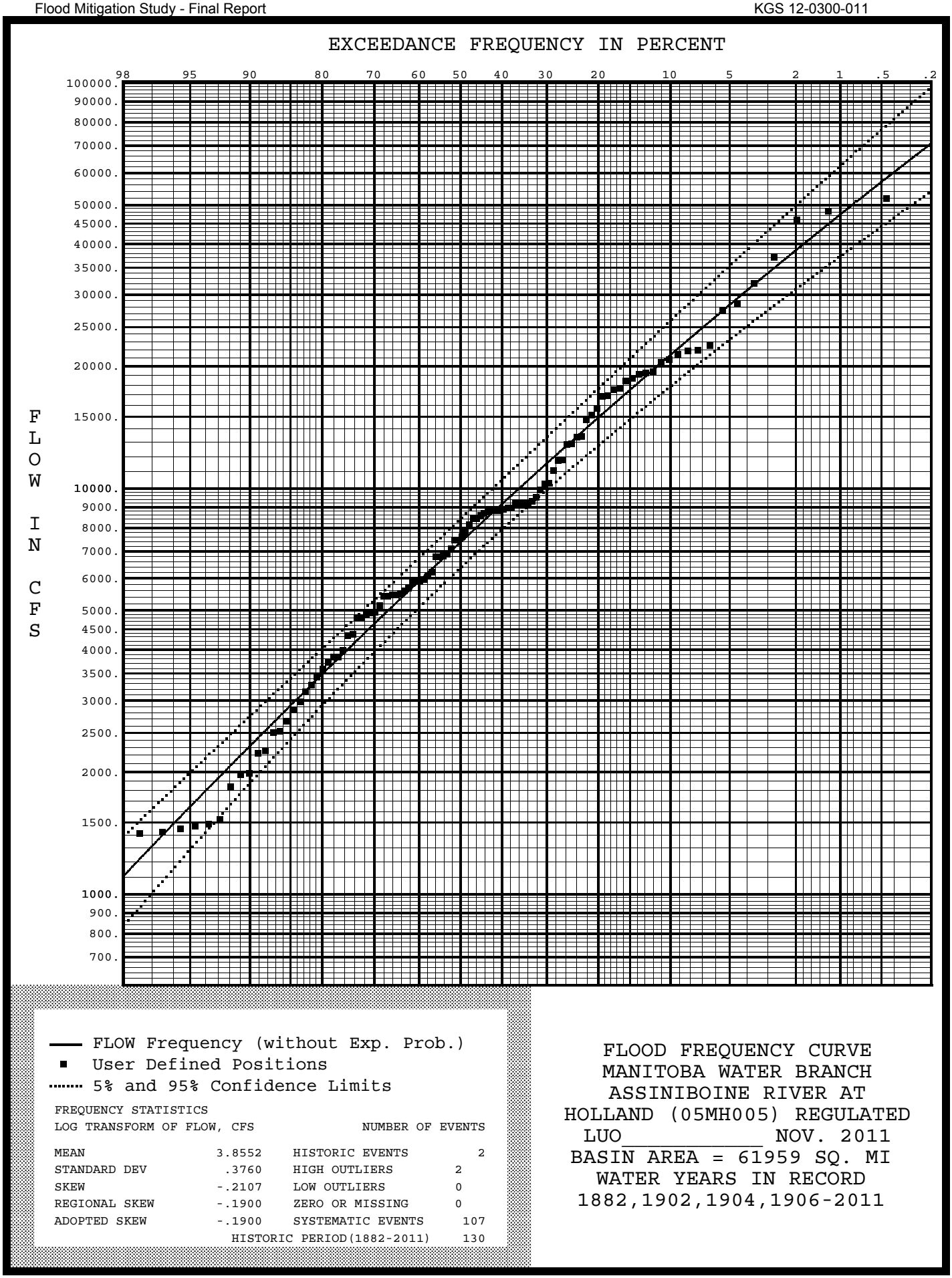


FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

MEAN	3.6553	HISTORIC EVENTS	3
STANDARD DEV	.3764	HIGH OUTLIERS	3
SKEW	-.2524	LOW OUTLIERS	0
REGIONAL SKEW	-.1810	ZERO OR MISSING	0
ADOPTED SKEW	-.1810	SYSTEMATIC EVENTS	106
		HISTORIC PERIOD(1882-2011)	130

NUMBER OF EVENTS

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
ASSINIBOINE RIVER AT
BRANDON 05MH013-REGULATED
BLUO_____ NOV 2012
DATA FOR YEARS INCLUDED:
1882,1902,1904,1906-2005
BASIN AREA = 36163 SQ. MI.



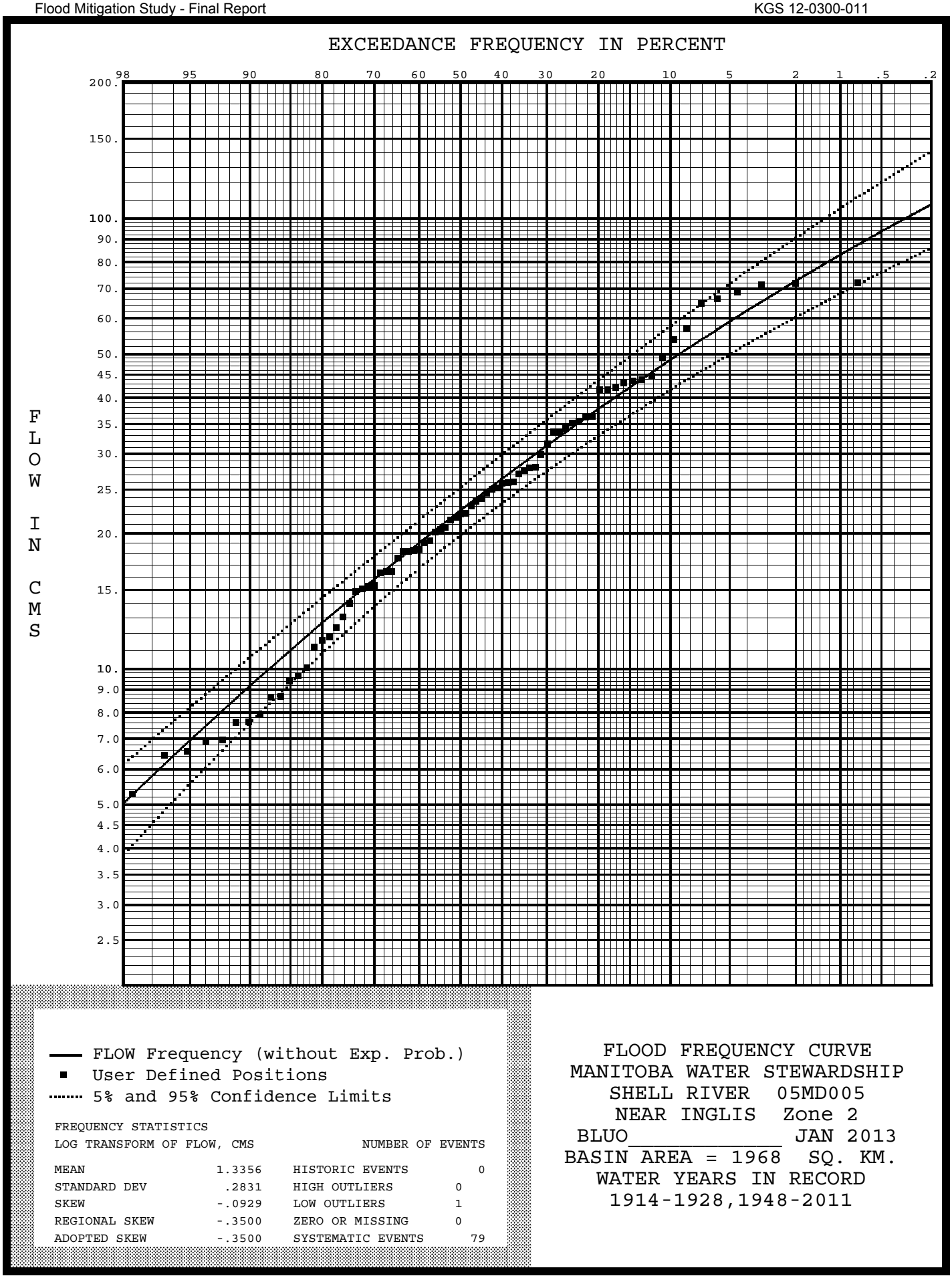
FLOW Frequency (without Exp. Prob.)
User Defined Positions
5% and 95% Confidence Limits

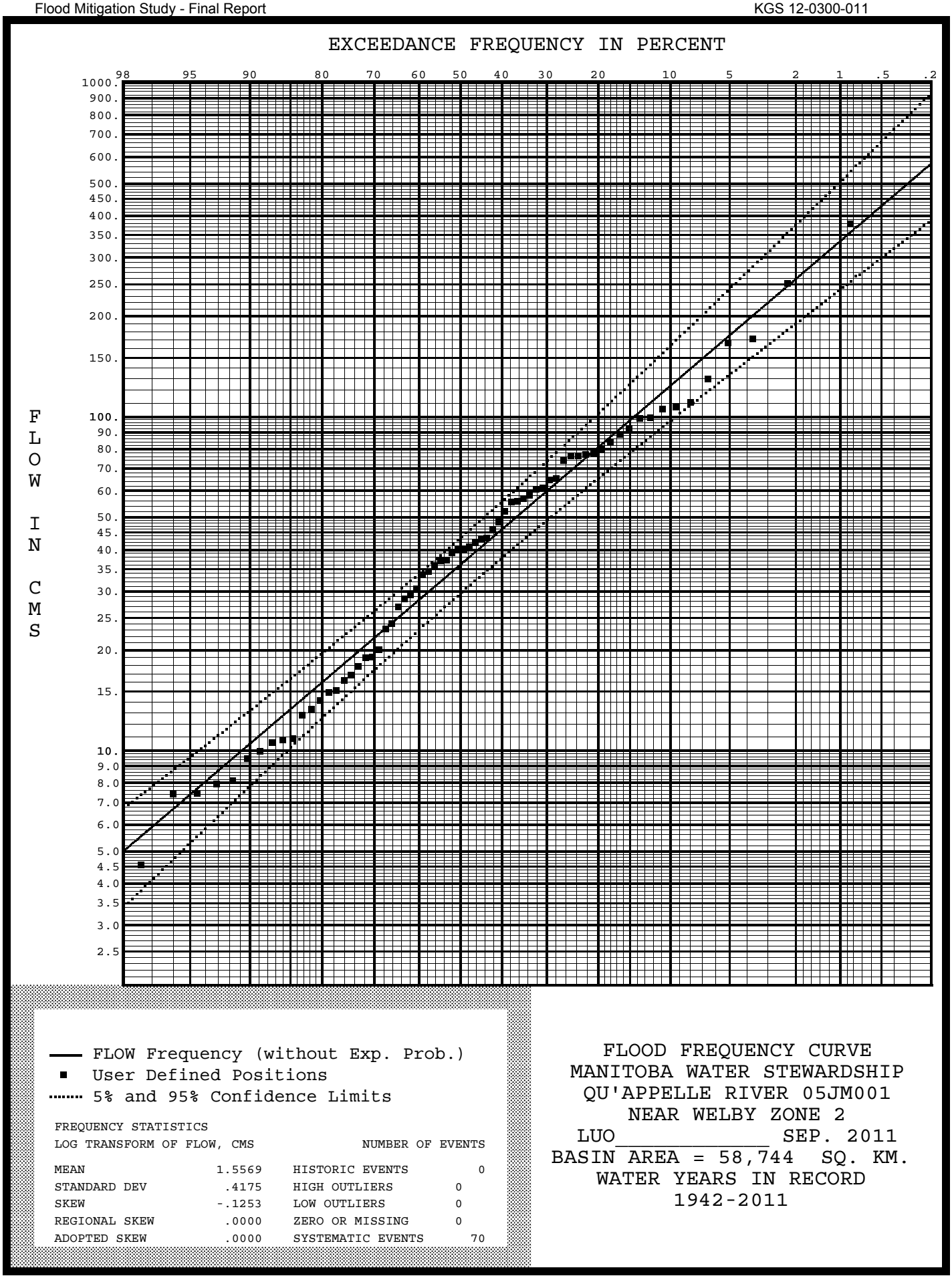
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CFS

MEAN	3.8552	HISTORIC EVENTS	2
STANDARD DEV	.3760	HIGH OUTLIERS	2
SKEW	-.2107	LOW OUTLIERS	0
REGIONAL SKEW	-.1900	ZERO OR MISSING	0
ADOPTED SKEW	-.1900	SYSTEMATIC EVENTS	107
		HISTORIC PERIOD(1882-2011)	130

NUMBER OF EVENTS

FLOOD FREQUENCY CURVE
MANITOBA WATER BRANCH
ASSINIBOINE RIVER AT
HOLLAND (05MH005) REGULATED
LUO_____ NOV. 2011
BASIN AREA = 61959 SQ. MI
WATER YEARS IN RECORD
1882,1902,1904,1906-2011


EXCEEDANCE FREQUENCY IN PERCENT
98 95 90 80 70 60 50 40 30 20 10 5 2 1 .5 .2
200. 150. 100. 90. 80. 70. 60. 50. 45. 40. 35. 30. 25. 20. 15. 10. 9.0 8.0 7.0 6.0 5.0 4.5 4.0 3.5 3.0 2.5
FLOW IN CMS
FLOW Frequency (without Exp. Prob.)
User Defined Positions
5% and 95% Confidence Limits
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS
MEAN 1.3356
STANDARD DEV .2831
SKEW -.0929
REGIONAL SKEW -.3500
ADOPTED SKEW -.3500
NUMBER OF EVENTS
HISTORIC EVENTS 0
HIGH OUTLIERS 0
LOW OUTLIERS 1
ZERO OR MISSING 0
SYSTEMATIC EVENTS 79
FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
SHELL RIVER 05MD005
NEAR INGLIS Zone 2
BLUO JAN 2013
BASIN AREA = 1968 SQ. KM.
WATER YEARS IN RECORD
1914-1928,1948-2011

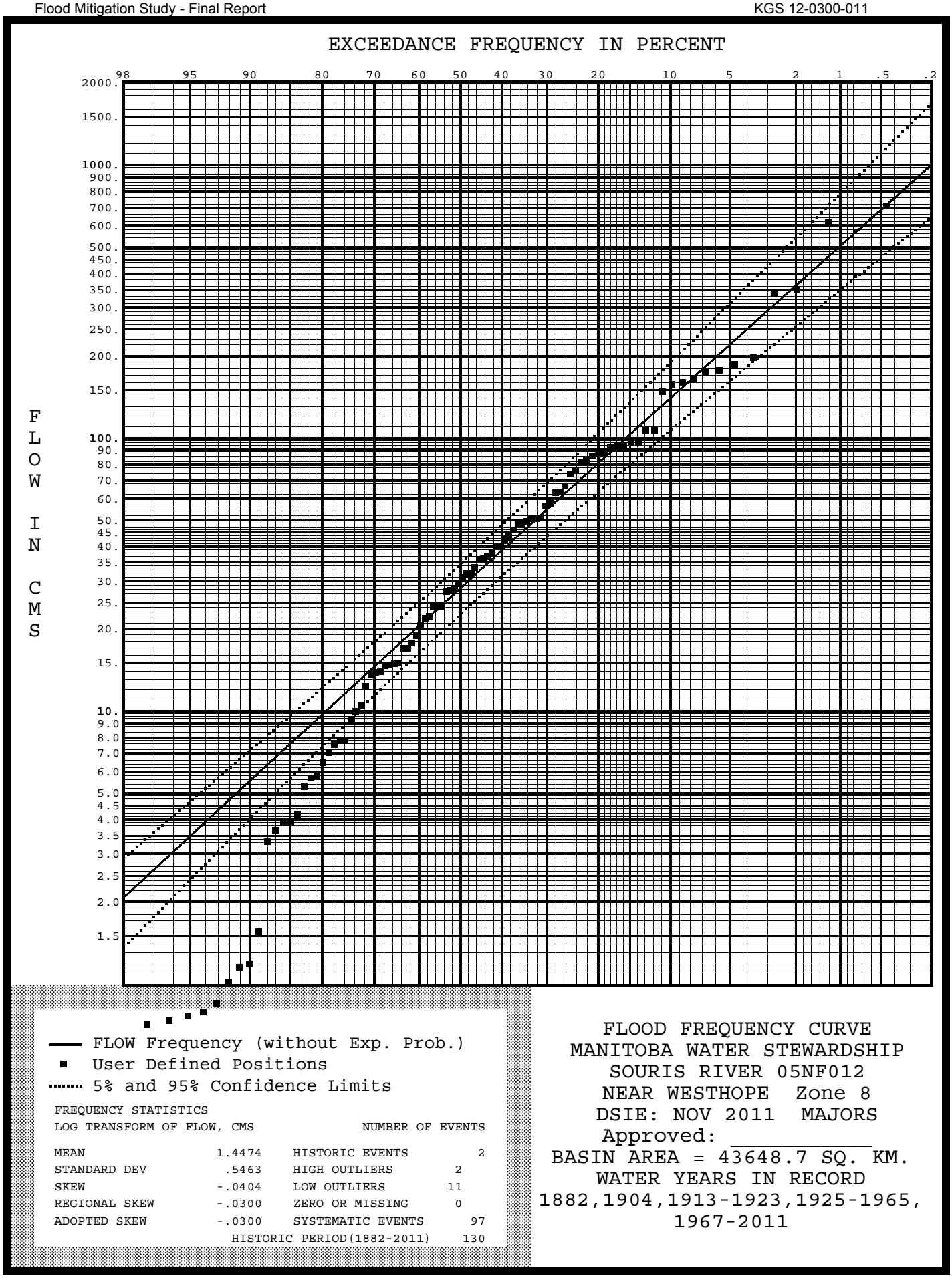


FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS

MEAN	1.5569	HISTORIC EVENTS	0
STANDARD DEV	.4175	HIGH OUTLIERS	0
SKEW	-.1253	LOW OUTLIERS	0
REGIONAL SKEW	.0000	ZERO OR MISSING	0
ADOPTED SKEW	.0000	SYSTEMATIC EVENTS	70

NUMBER OF EVENTS

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
QU'APPELLE RIVER 05JM001
NEAR WELBY ZONE 2
LUO_____ SEP. 2011
BASIN AREA = 58,744 SQ. KM.
WATER YEARS IN RECORD
1942-2011

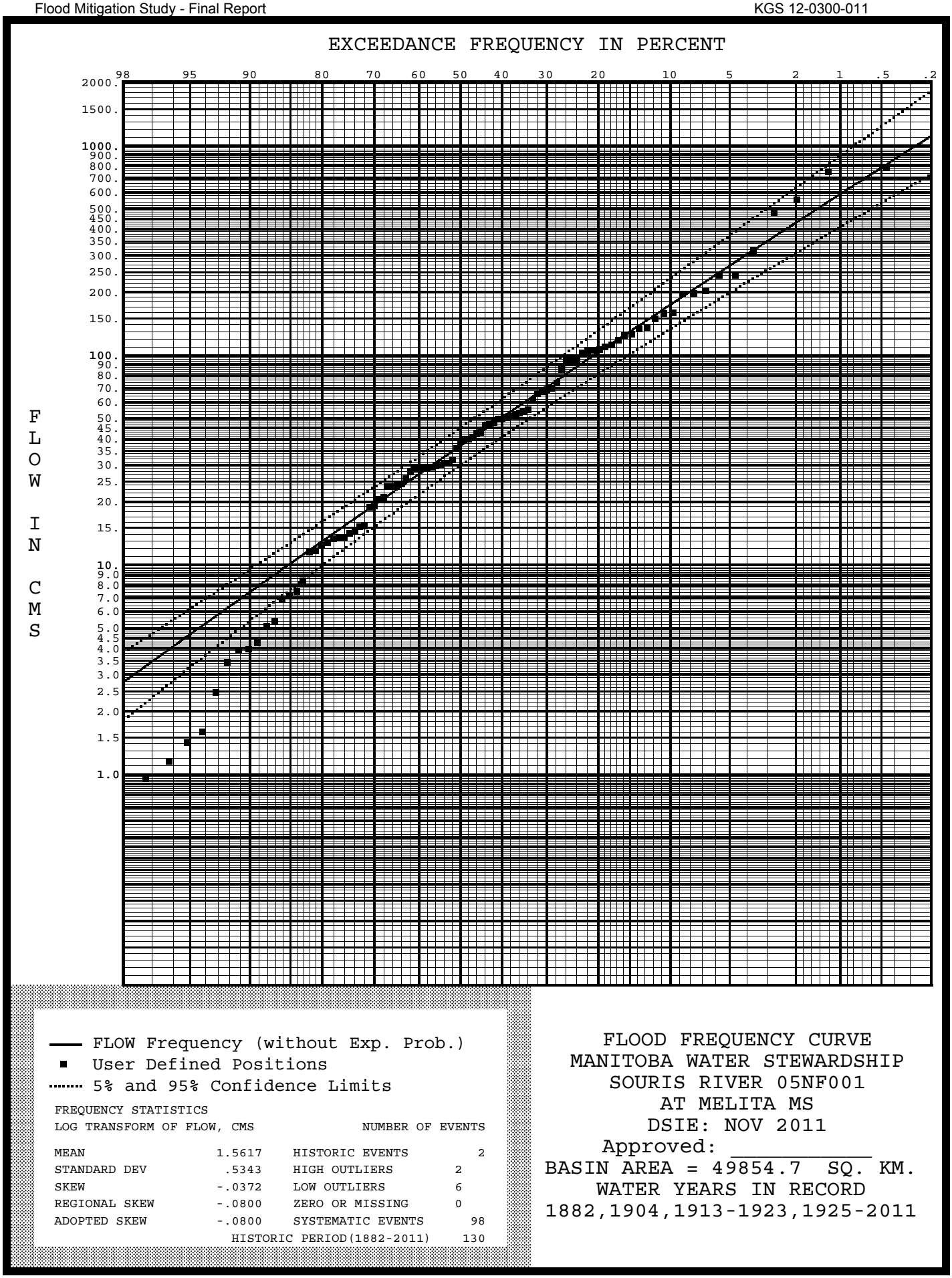


FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS

MEAN	1.4474	HISTORIC EVENTS	2
STANDARD DEV	.5463	HIGH OUTLIERS	2
SKEW	-.0404	LOW OUTLIERS	11
REGIONAL SKEW	-.0300	ZERO OR MISSING	0
ADOPTED SKEW	-.0300	SYSTEMATIC EVENTS	97
		HISTORIC PERIOD(1882-2011)	130

NUMBER OF EVENTS

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
SOURIS RIVER 05NF012
NEAR WESTHOPE Zone 8
DSIE: NOV 2011 MAJORS
Approved: _____
BASIN AREA = 43648.7 SQ. KM.
WATER YEARS IN RECORD
1882,1904,1913-1923,1925-1965,
1967-2011



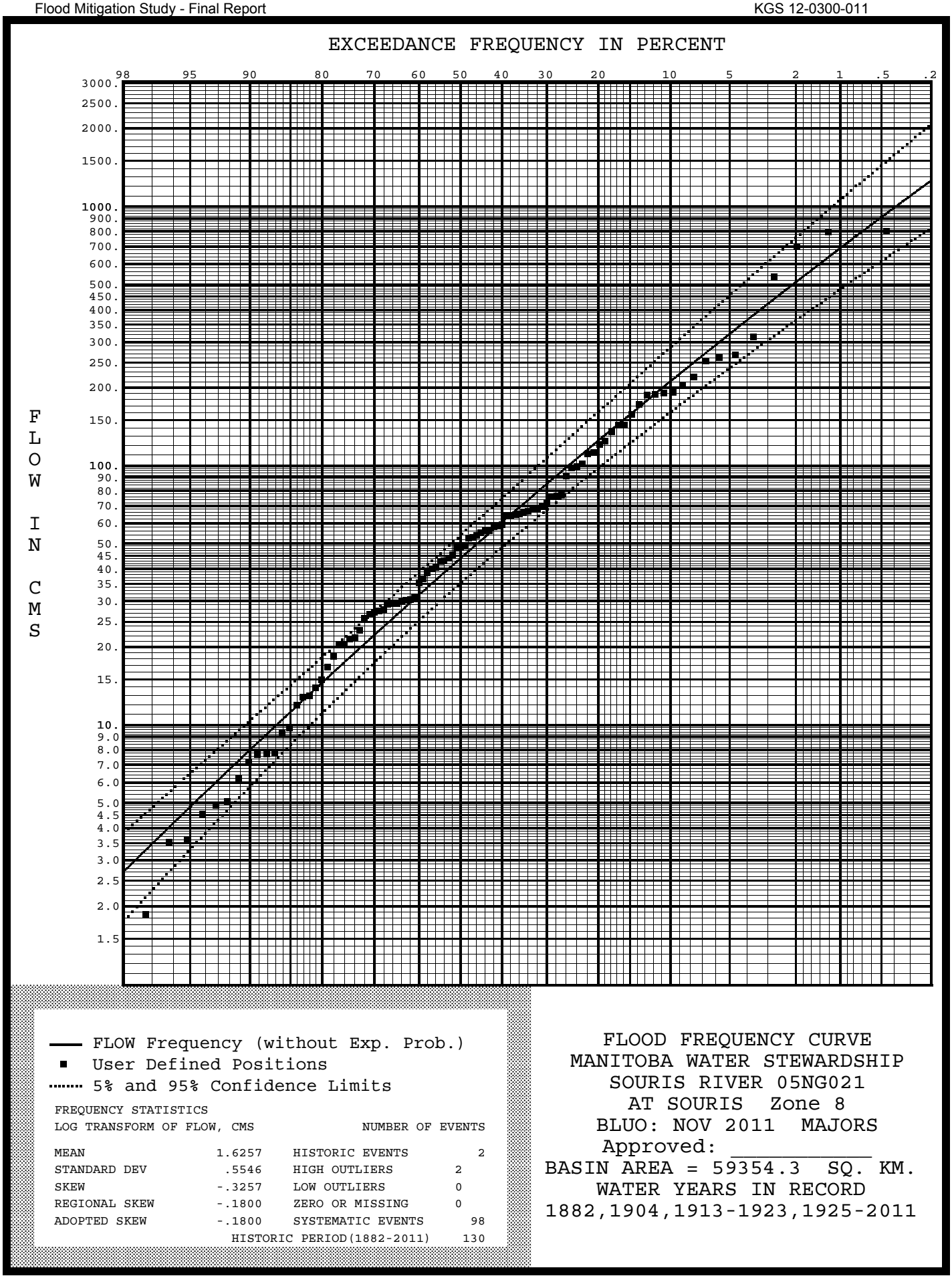
FLOW Frequency (without Exp. Prob.)
User Defined Positions
5% and 95% Confidence Limits

FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS

MEAN	1.5617	HISTORIC EVENTS	2
STANDARD DEV	.5343	HIGH OUTLIERS	2
SKEW	-.0372	LOW OUTLIERS	6
REGIONAL SKEW	-.0800	ZERO OR MISSING	0
ADOPTED SKEW	-.0800	SYSTEMATIC EVENTS	98
		HISTORIC PERIOD(1882-2011)	130

NUMBER OF EVENTS

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
SOURIS RIVER 05NF001
AT MELITA MS
DSIE: NOV 2011
Approved: _____
BASIN AREA = 49854.7 SQ. KM.
WATER YEARS IN RECORD
1882,1904,1913-1923,1925-2011

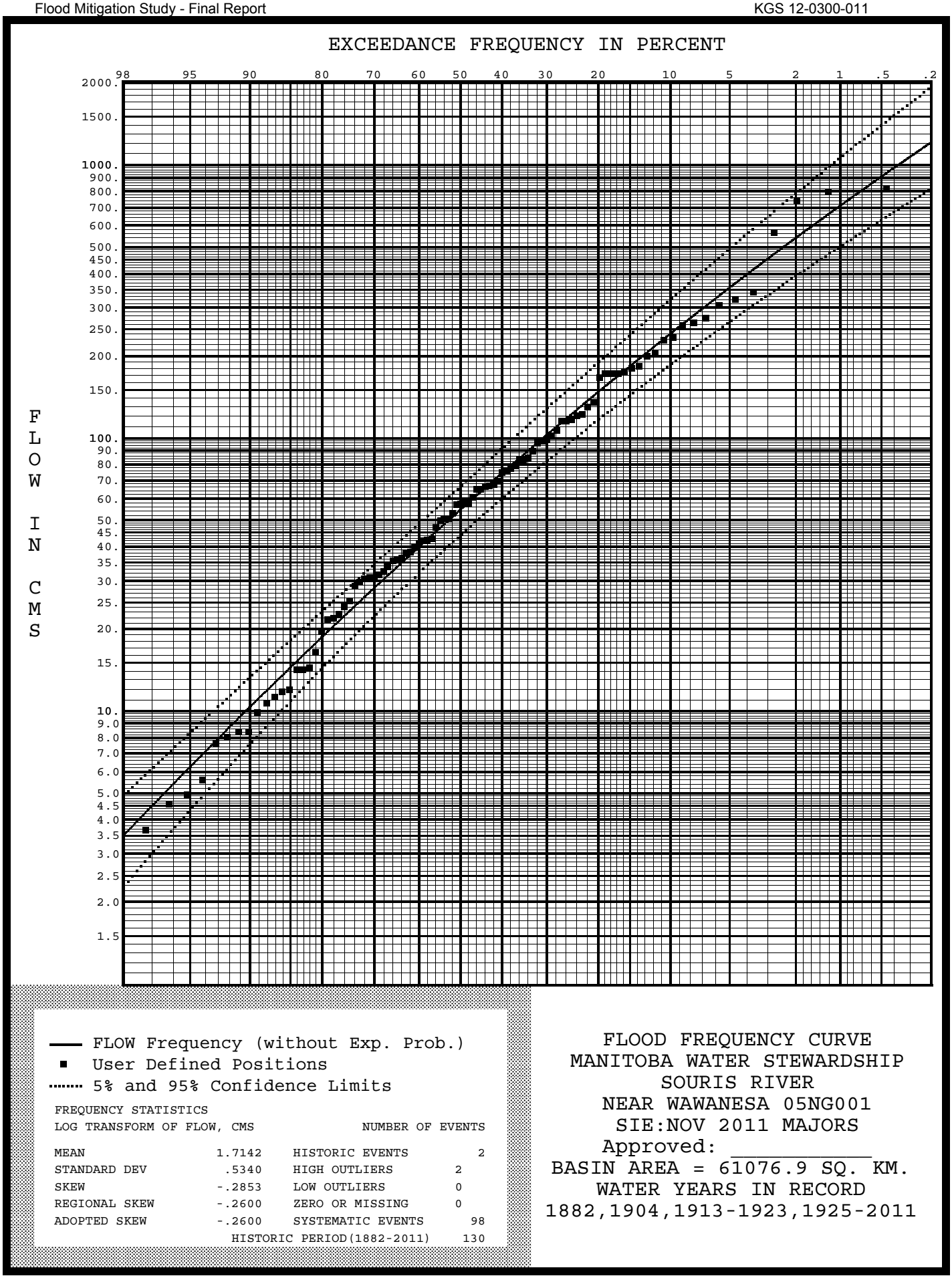


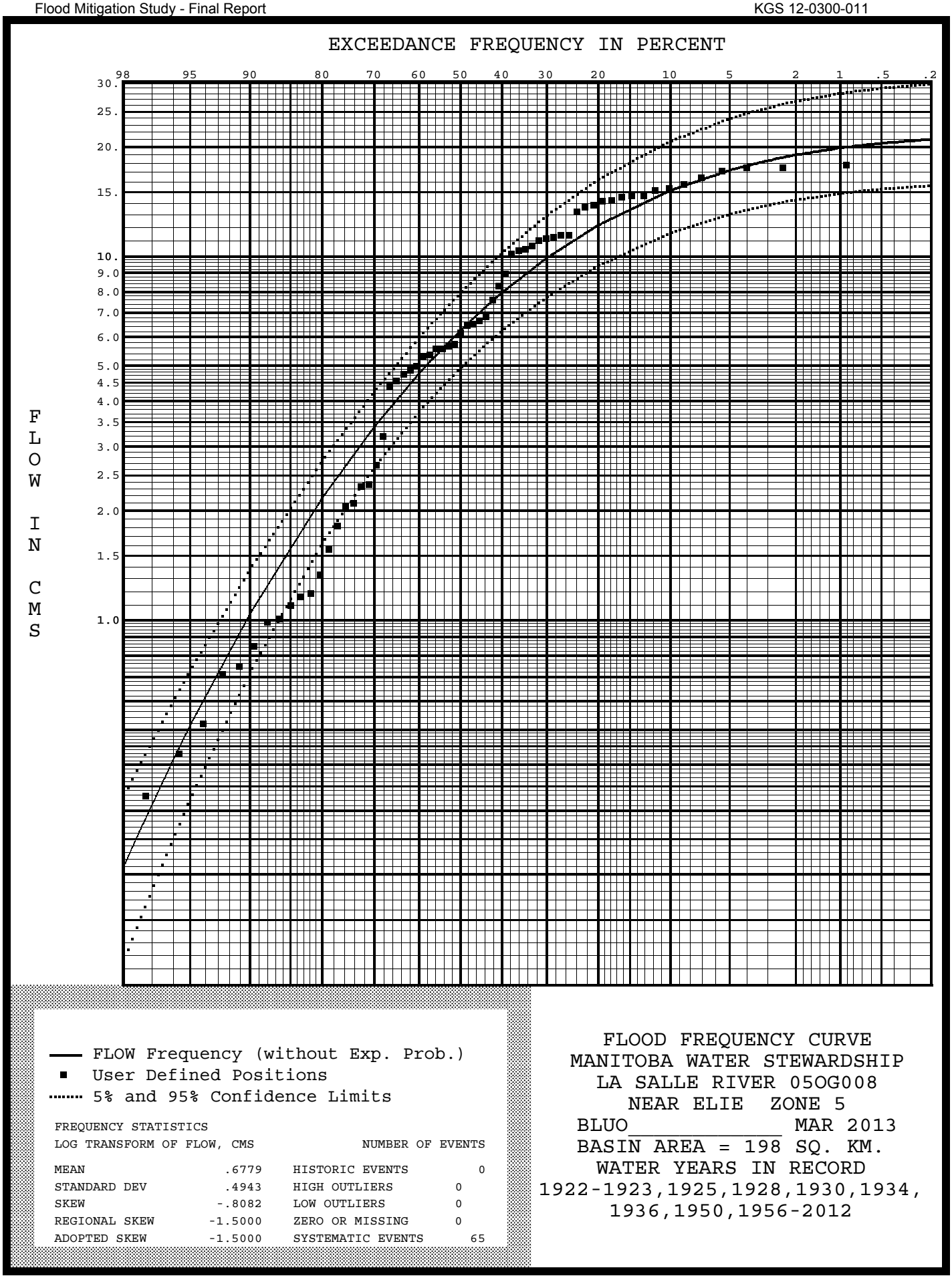
FLOW Frequency (without Exp. Prob.)
User Defined Positions
5% and 95% Confidence Limits

FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS

MEAN	1.6257	NUMBER OF EVENTS	
STANDARD DEV	.5546	HISTORIC EVENTS	2
SKEW	-.3257	HIGH OUTLIERS	2
REGIONAL SKEW	-.1800	LOW OUTLIERS	0
ADOPTED SKEW	-.1800	ZERO OR MISSING	0
		SYSTEMATIC EVENTS	98
		HISTORIC PERIOD(1882-2011)	130

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
SOURIS RIVER 05NG021
AT SOURIS Zone 8
BLUO: NOV 2011 MAJORS
Approved: _____
BASIN AREA = 59354.3 SQ. KM.
WATER YEARS IN RECORD
1882,1904,1913-1923,1925-2011


EXCEEDANCE FREQUENCY IN PERCENT
98 95 90 80 70 60 50 40 30 20 10 5 2 1 .5 .2
2000. 1500. 1000. 900. 800. 700. 600. 500. 450. 400. 350. 300. 250. 200. 150. 100. 90. 80. 70. 60. 50. 45. 40. 35. 30. 25. 20. 15. 10. 9.0 8.0 7.0 6.0 5.0 4.5 4.0 3.5 3.0 2.5 2.0 1.5
FLOW IN CMS
FLOW Frequency (without Exp. Prob.)
User Defined Positions
5% and 95% Confidence Limits
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS
MEAN 1.7142
STANDARD DEV .5340
SKEW -.2853
REGIONAL SKEW -.2600
ADOPTED SKEW -.2600
NUMBER OF EVENTS
HISTORIC EVENTS 2
HIGH OUTLIERS 2
LOW OUTLIERS 0
ZERO OR MISSING 0
SYSTEMATIC EVENTS 98
HISTORIC PERIOD(1882-2011) 130
FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
SOURIS RIVER
NEAR WAWANESA 05NG001
SIE:NOV 2011 MAJORS
Approved: _____
BASIN AREA = 61076.9 SQ. KM.
WATER YEARS IN RECORD
1882,1904,1913-1923,1925-2011

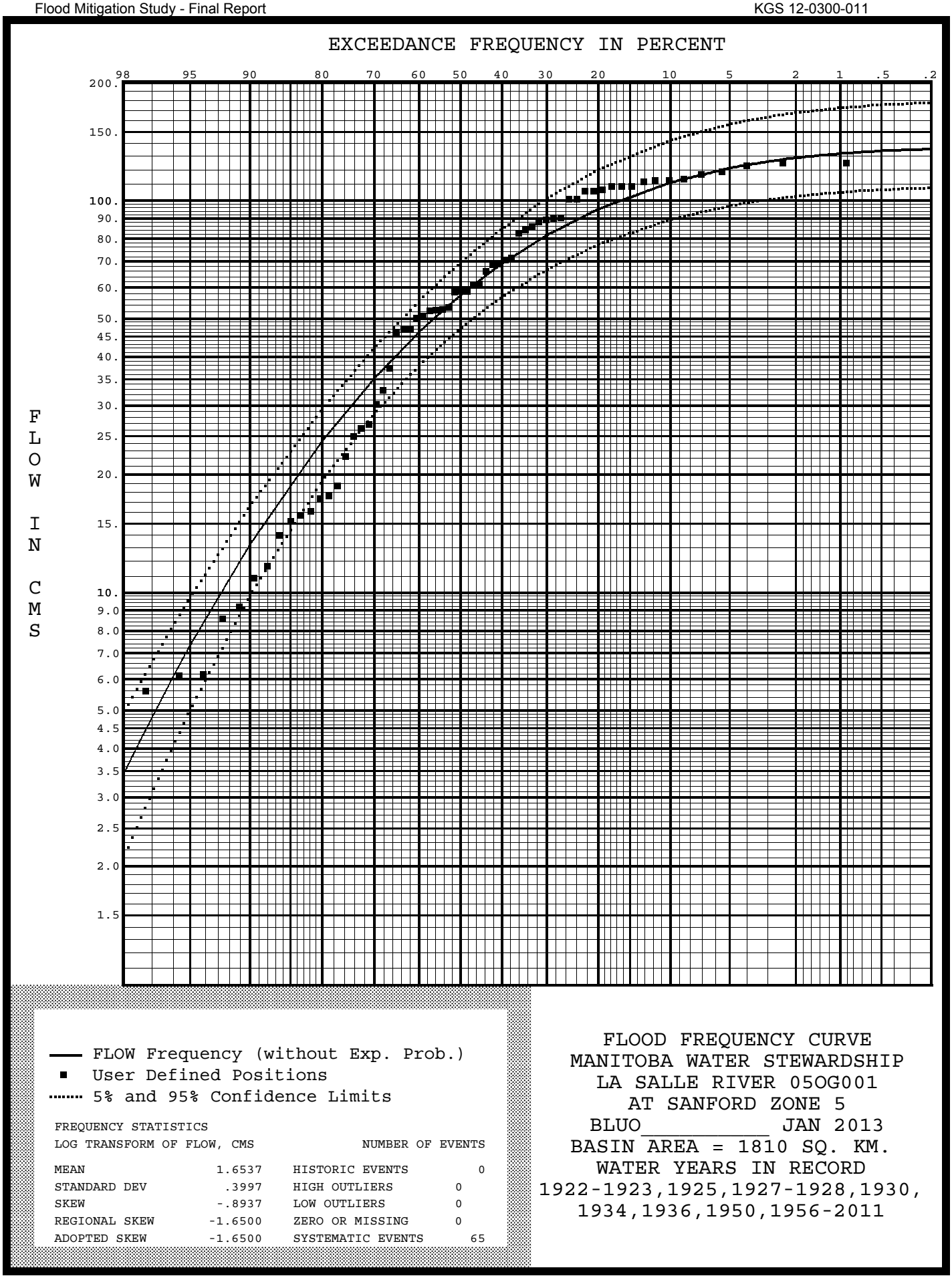


FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS

MEAN	.6779	HISTORIC EVENTS	0
STANDARD DEV	.4943	HIGH OUTLIERS	0
SKEW	-.8082	LOW OUTLIERS	0
REGIONAL SKEW	-1.5000	ZERO OR MISSING	0
ADOPTED SKEW	-1.5000	SYSTEMATIC EVENTS	65

NUMBER OF EVENTS

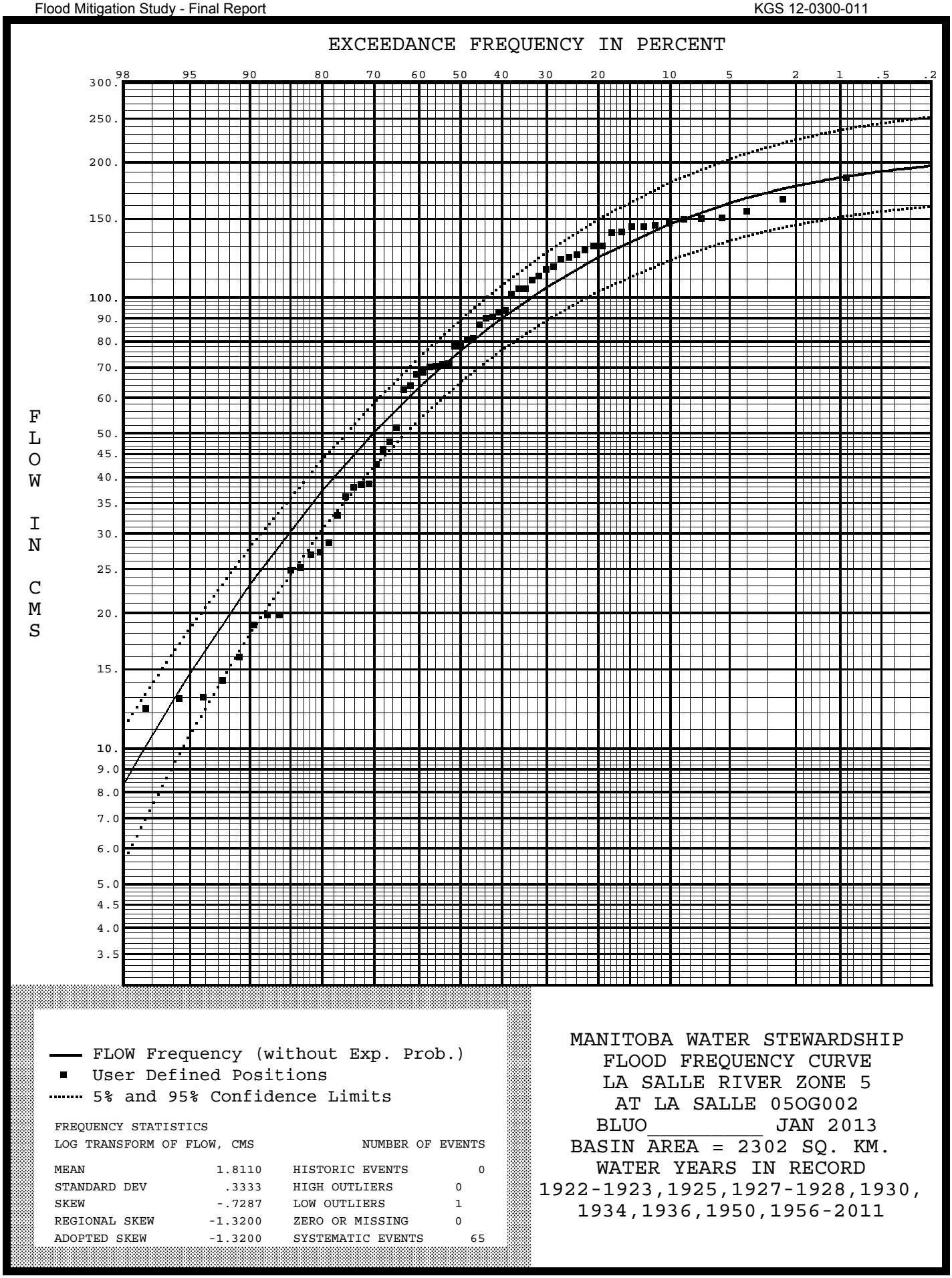
FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
LA SALLE RIVER 05OG008
NEAR ELIE ZONE 5
BLUO_____ MAR 2013
BASIN AREA = 198 SQ. KM.
WATER YEARS IN RECORD
1922-1923,1925,1928,1930,1934,
1936,1950,1956-2012

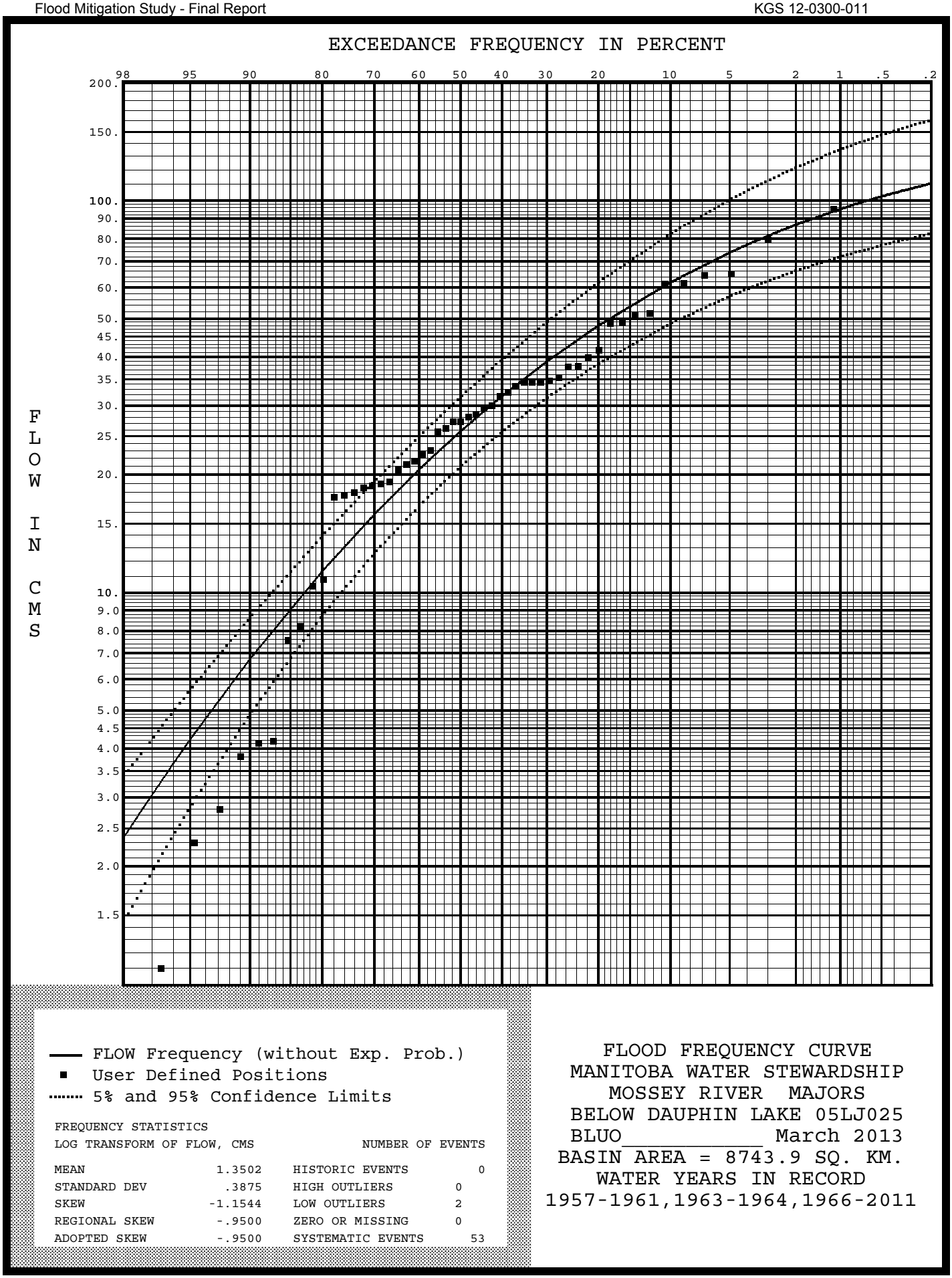


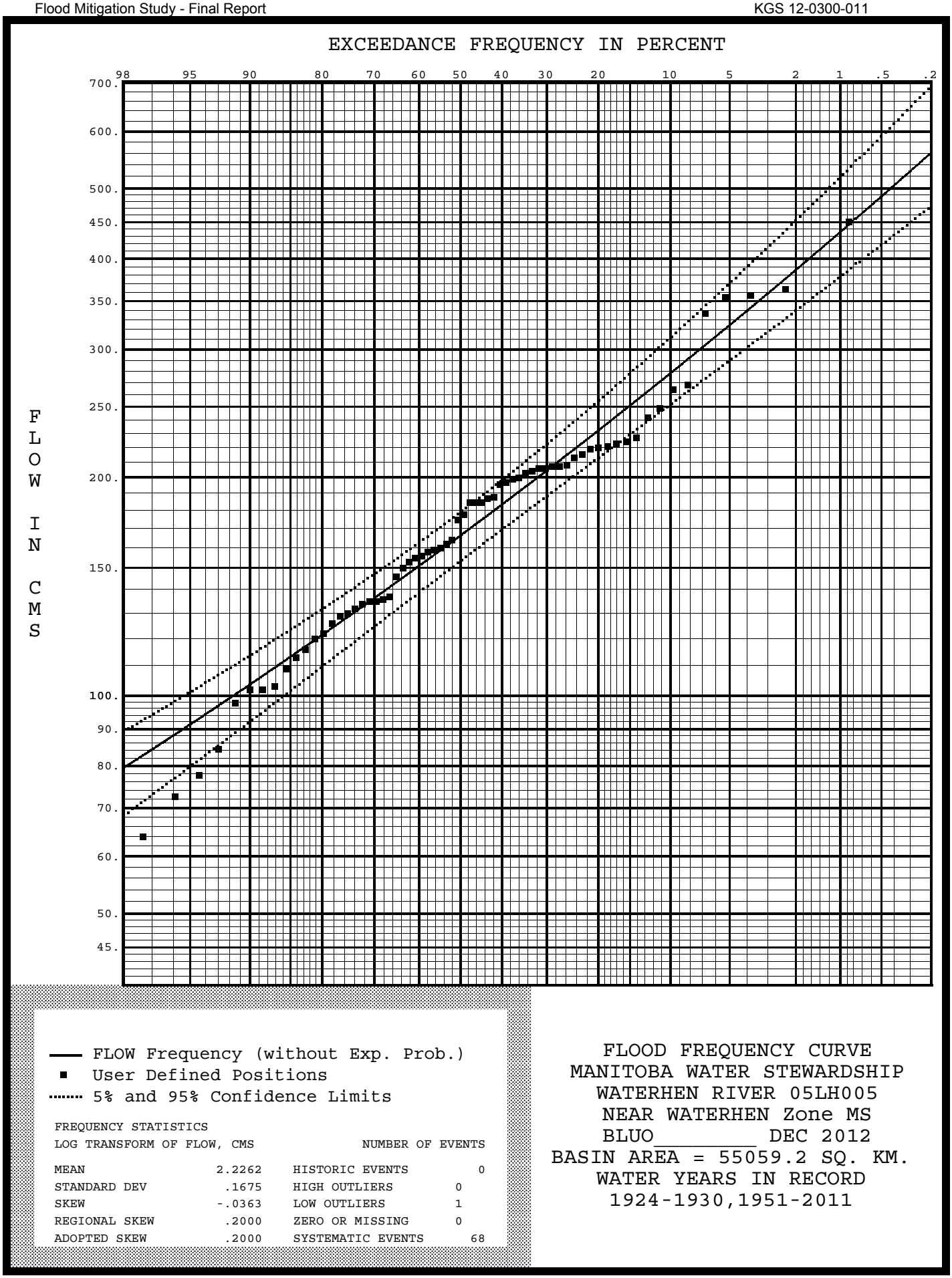
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS

MEAN	1.6537	NUMBER OF EVENTS	
STANDARD DEV	.3997	HISTORIC EVENTS	0
SKEW	-.8937	HIGH OUTLIERS	0
REGIONAL SKEW	-1.6500	LOW OUTLIERS	0
ADOPTED SKEW	-1.6500	ZERO OR MISSING	0
		SYSTEMATIC EVENTS	65

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
LA SALLE RIVER 05OG001
AT SANFORD ZONE 5
BLUO_____ JAN 2013
BASIN AREA = 1810 SQ. KM.
WATER YEARS IN RECORD
1922-1923,1925,1927-1928,1930,
1934,1936,1950,1956-2011


EXCEEDANCE FREQUENCY IN PERCENT
98 95 90 80 70 60 50 40 30 20 10 5 2 1 .5 .2
300. 250. 200. 150. 100. 90. 80. 70. 60. 50. 45. 40. 35. 30. 25. 20. 15. 10. 9.0 8.0 7.0 6.0 5.0 4.5 4.0 3.5
FLOW IN CMS
FLOW Frequency (without Exp. Prob.)
User Defined Positions
5% and 95% Confidence Limits
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS
MEAN 1.8110
STANDARD DEV .3333
SKEW -.7287
REGIONAL SKEW -1.3200
ADOPTED SKEW -1.3200
NUMBER OF EVENTS
HISTORIC EVENTS 0
HIGH OUTLIERS 0
LOW OUTLIERS 1
ZERO OR MISSING 0
SYSTEMATIC EVENTS 65
MANITOBA WATER STEWARDSHIP
FLOOD FREQUENCY CURVE
LA SALLE RIVER ZONE 5
AT LA SALLE 05OG002
BLUO_____ JAN 2013
BASIN AREA = 2302 SQ. KM.
WATER YEARS IN RECORD
1922-1923,1925,1927-1928,1930,
1934,1936,1950,1956-2011


EXCEEDANCE FREQUENCY IN PERCENT
98 95 90 80 70 60 50 40 30 20 10 5 2 1 .5 .2
200. 150. 100. 90. 80. 70. 60. 50. 45. 40. 35. 30. 25. 20. 15. 10. 9.0 8.0 7.0 6.0 5.0 4.5 4.0 3.5 3.0 2.5 2.0 1.5
FLOW IN CMS
FLOW Frequency (without Exp. Prob.)
User Defined Positions
5% and 95% Confidence Limits
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS
MEAN 1.3502
STANDARD DEV .3875
SKEW -1.1544
REGIONAL SKEW -.9500
ADOPTED SKEW -.9500
NUMBER OF EVENTS
HISTORIC EVENTS 0
HIGH OUTLIERS 0
LOW OUTLIERS 2
ZERO OR MISSING 0
SYSTEMATIC EVENTS 53
FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
MOSSEY RIVER MAJORS
BELOW DAUPHIN LAKE 05LJ025
BLUO March 2013
BASIN AREA = 8743.9 SQ. KM.
WATER YEARS IN RECORD
1957-1961,1963-1964,1966-2011

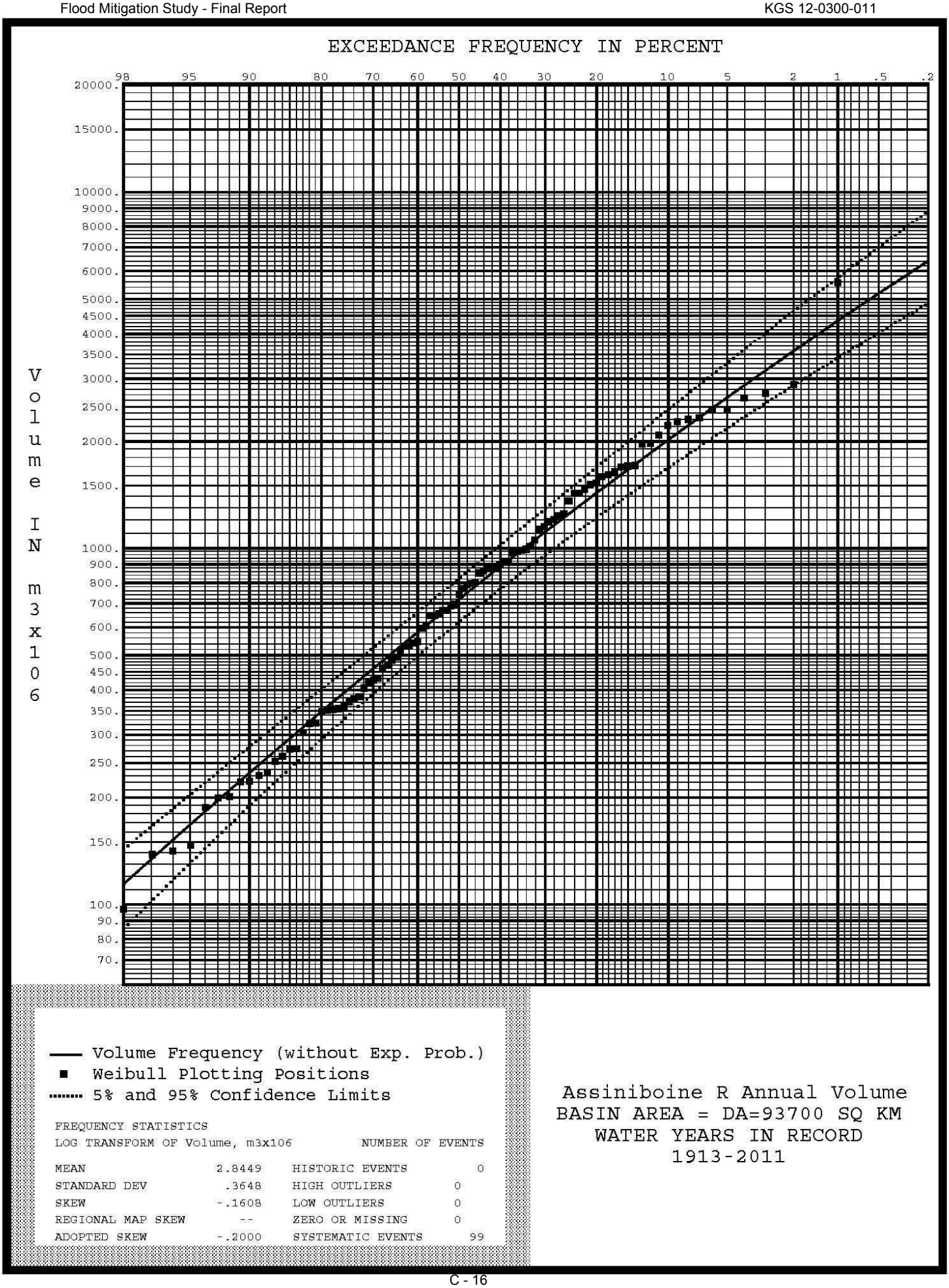


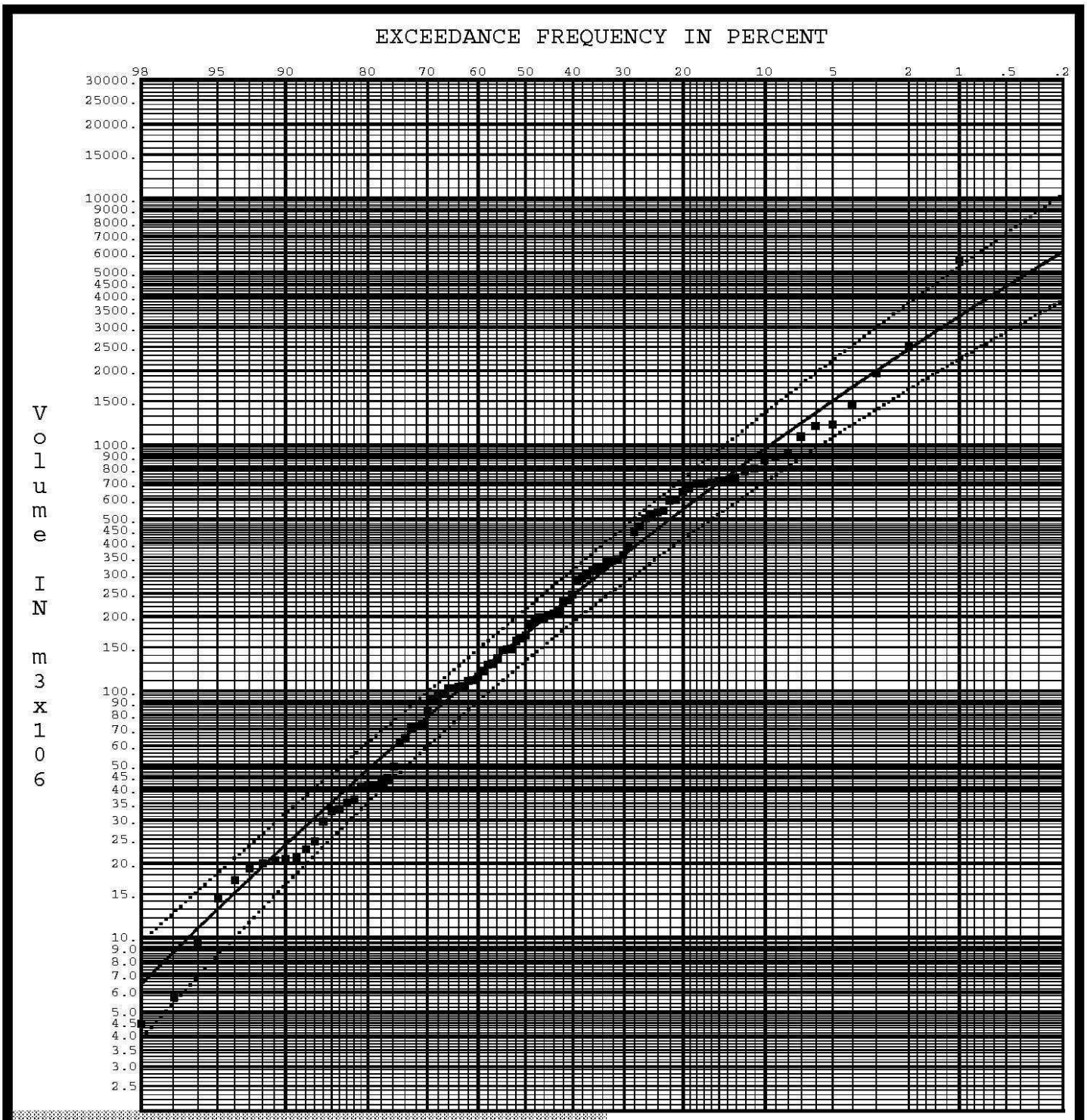
FREQUENCY STATISTICS
LOG TRANSFORM OF FLOW, CMS

MEAN	2.2262	HISTORIC EVENTS	0
STANDARD DEV	.1675	HIGH OUTLIERS	0
SKEW	-.0363	LOW OUTLIERS	1
REGIONAL SKEW	.2000	ZERO OR MISSING	0
ADOPTED SKEW	.2000	SYSTEMATIC EVENTS	68

NUMBER OF EVENTS

FLOOD FREQUENCY CURVE
MANITOBA WATER STEWARDSHIP
WATERHEN RIVER 05LH005
NEAR WATERHEN Zone MS
BLUO_____ DEC 2012
BASIN AREA = 55059.2 SQ. KM.
WATER YEARS IN RECORD
1924-1930,1951-2011


EXCEEDANCE FREQUENCY IN PERCENT
98 95 90 80 70 60 50 40 30 20 10 5 2 1 .5 .2
Volume IN m3x106
20000. 15000. 10000. 9000. 8000. 7000. 6000. 5000. 4500. 4000. 3500. 3000. 2500. 2000. 1500. 1000. 900. 800. 700. 600. 500. 450. 400. 350. 300. 250. 200. 150. 100. 90. 80. 70.
Volume Frequency (without Exp. Prob.)
Weibull Plotting Positions
5% and 95% Confidence Limits
FREQUENCY STATISTICS
LOG TRANSFORM OF Volume, m3x106
NUMBER OF EVENTS
MEAN 2.8449
STANDARD DEV .3648
SKEW -.1608
REGIONAL MAP SKEW --
ADOPTED SKEW -.2000
HISTORIC EVENTS 0
HIGH OUTLIERS 0
LOW OUTLIERS 0
ZERO OR MISSING 0
SYSTEMATIC EVENTS 99
Assiniboine R Annual Volume
BASIN AREA = DA=93700 SQ KM
WATER YEARS IN RECORD
1913-2011



Volume Frequency (without Exp. Prob.)
Weibull Plotting Positions
5% and 95% Confidence Limits

FREQUENCY STATISTICS
LOG TRANSFORM OF Volume, m3x106

MEAN	2.2003	HISTORIC EVENTS	0
STANDARD DEV	.6289	HIGH OUTLIERS	0
SKEW	-.2889	LOW OUTLIERS	0
REGIONAL MAP SKEW	--	ZERO OR MISSING	0
ADOPTED SKEW	-.3000	SYSTEMATIC EVENTS	99

NUMBER OF EVENTS

Souris River Annual Volume
BASIN AREA = 61600 SQ KM
WATER YEARS IN RECORD
1913-2011

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Frequency Curve of April to October Runoff Volumes at Holland, 1961-2011

Source: FRTF, 2013

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APPENDIX D

RATING CURVES AND DISCHARGE-DATE DAMAGE RELATIONSHIPS OF THE UPPER ASSINIBOINE RIVER

APPENDIX D1
RATING CURVES

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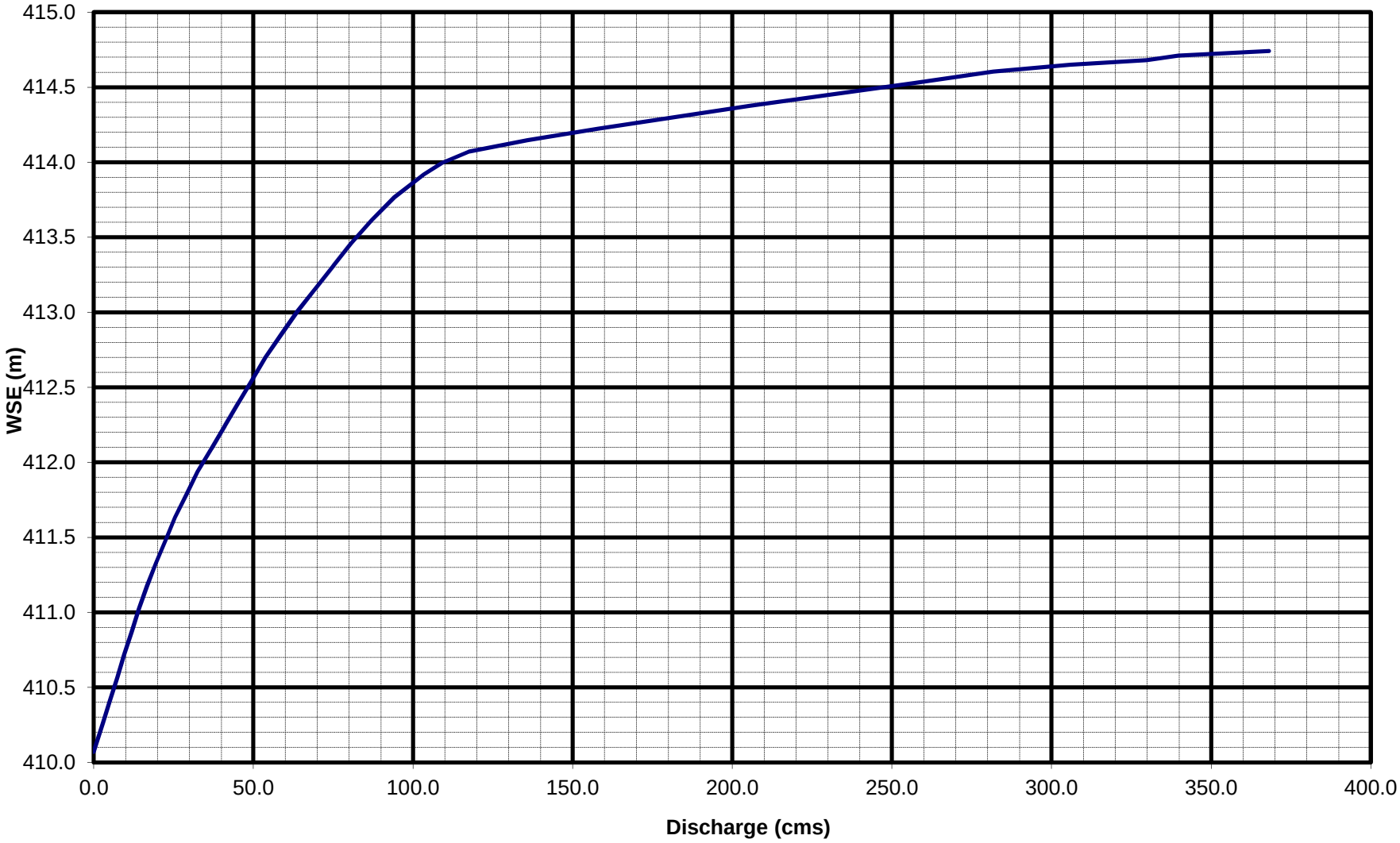
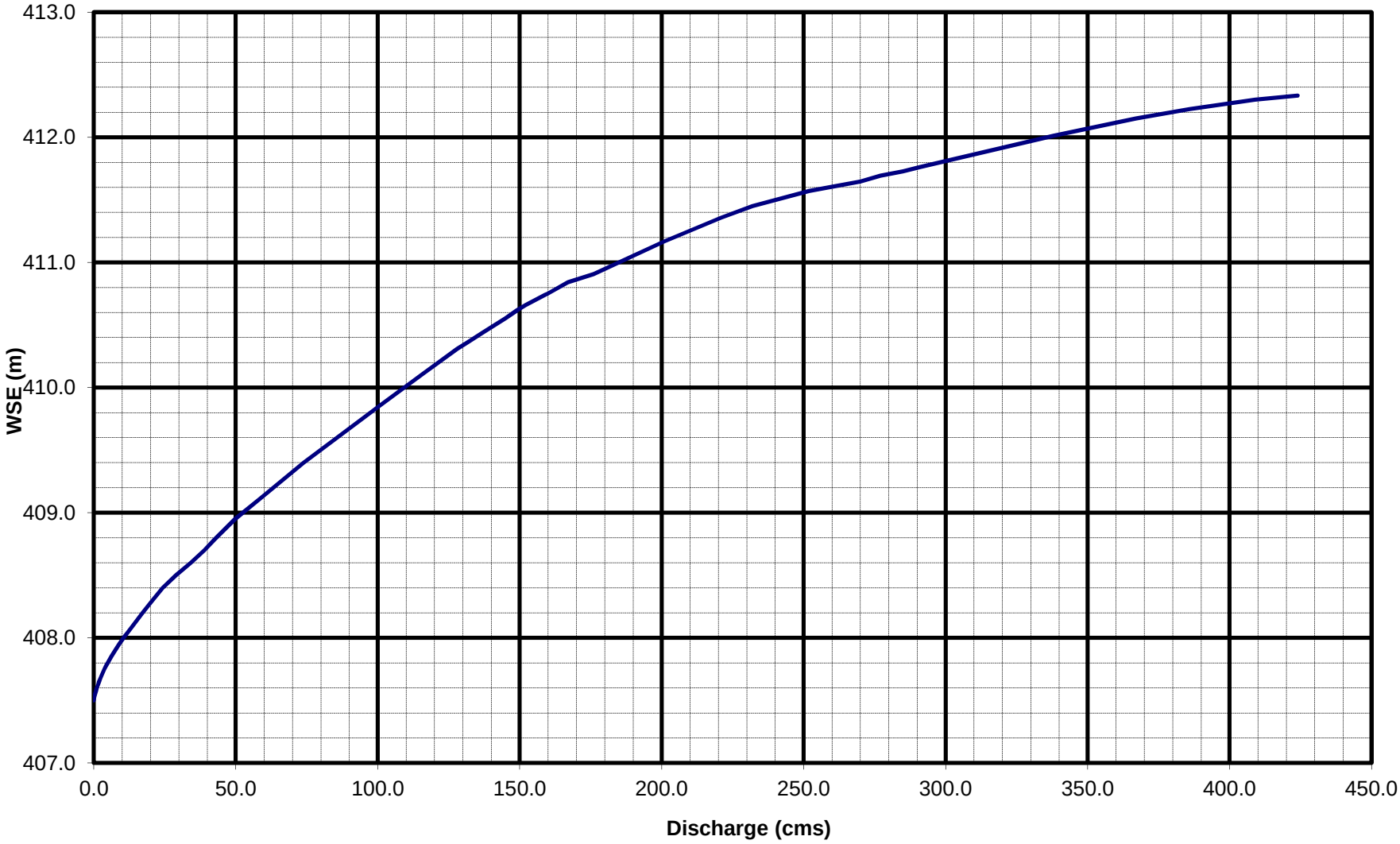
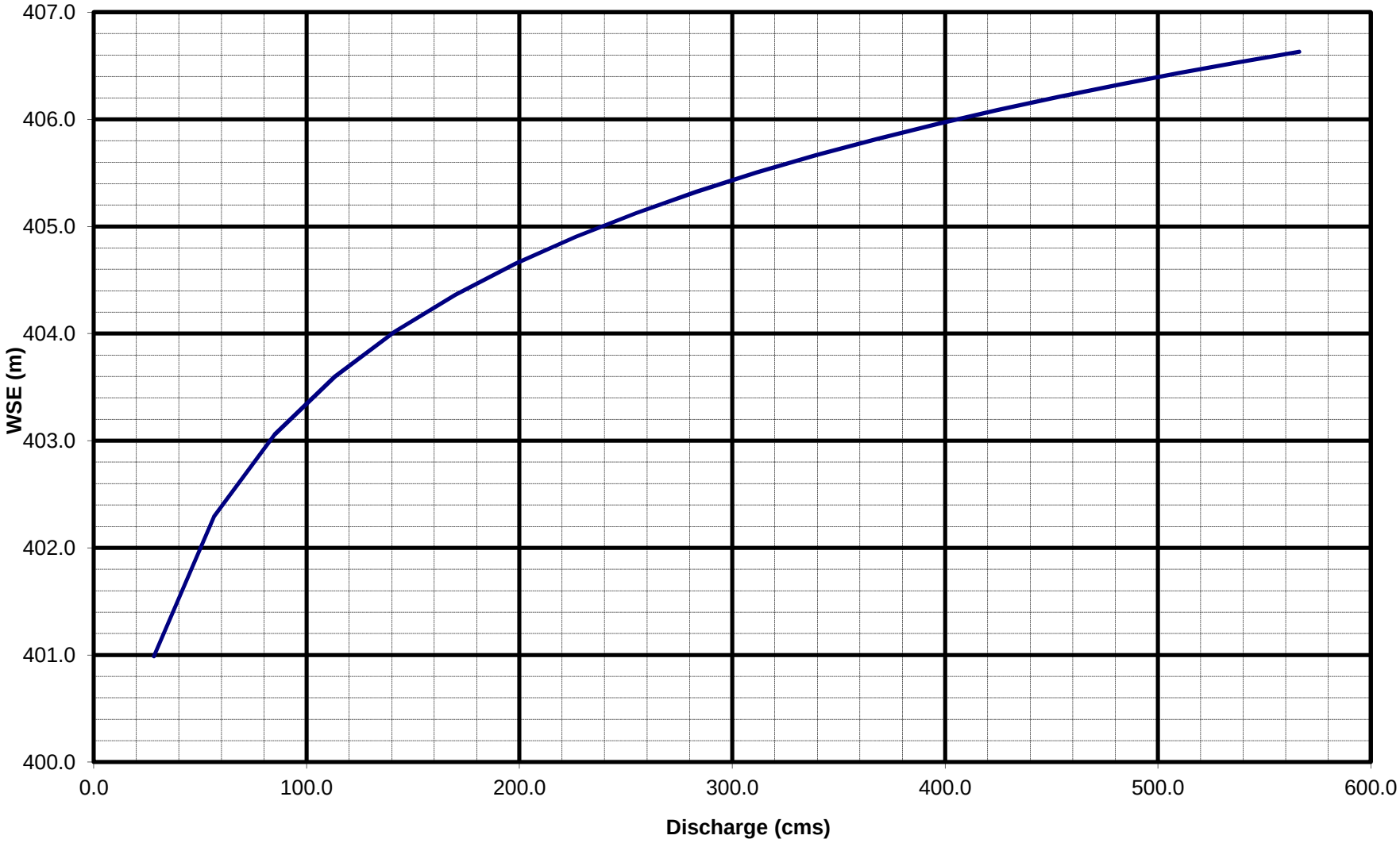
Assiniboine River at Shellmouth Bridge



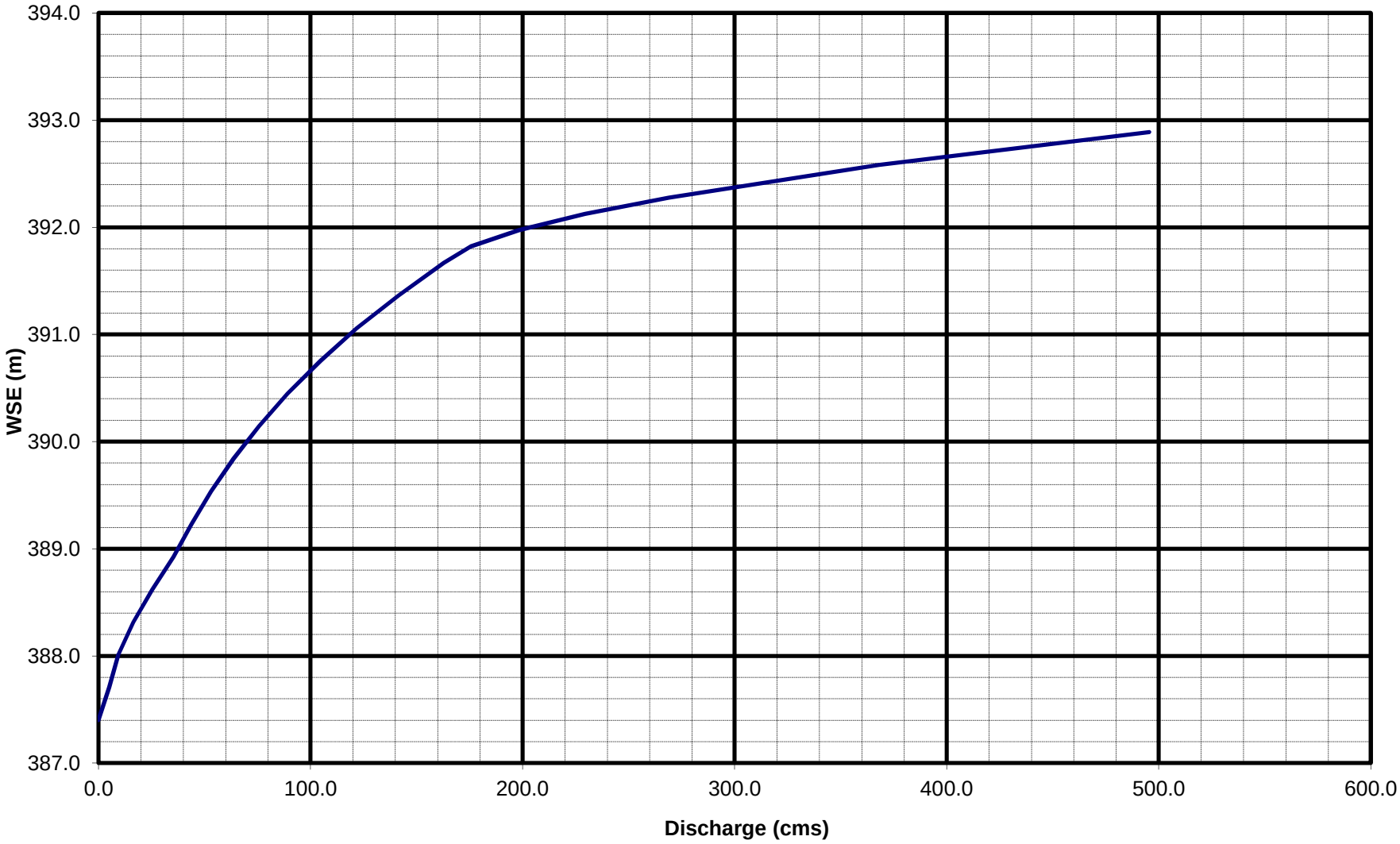
Assiniboine River near Russell



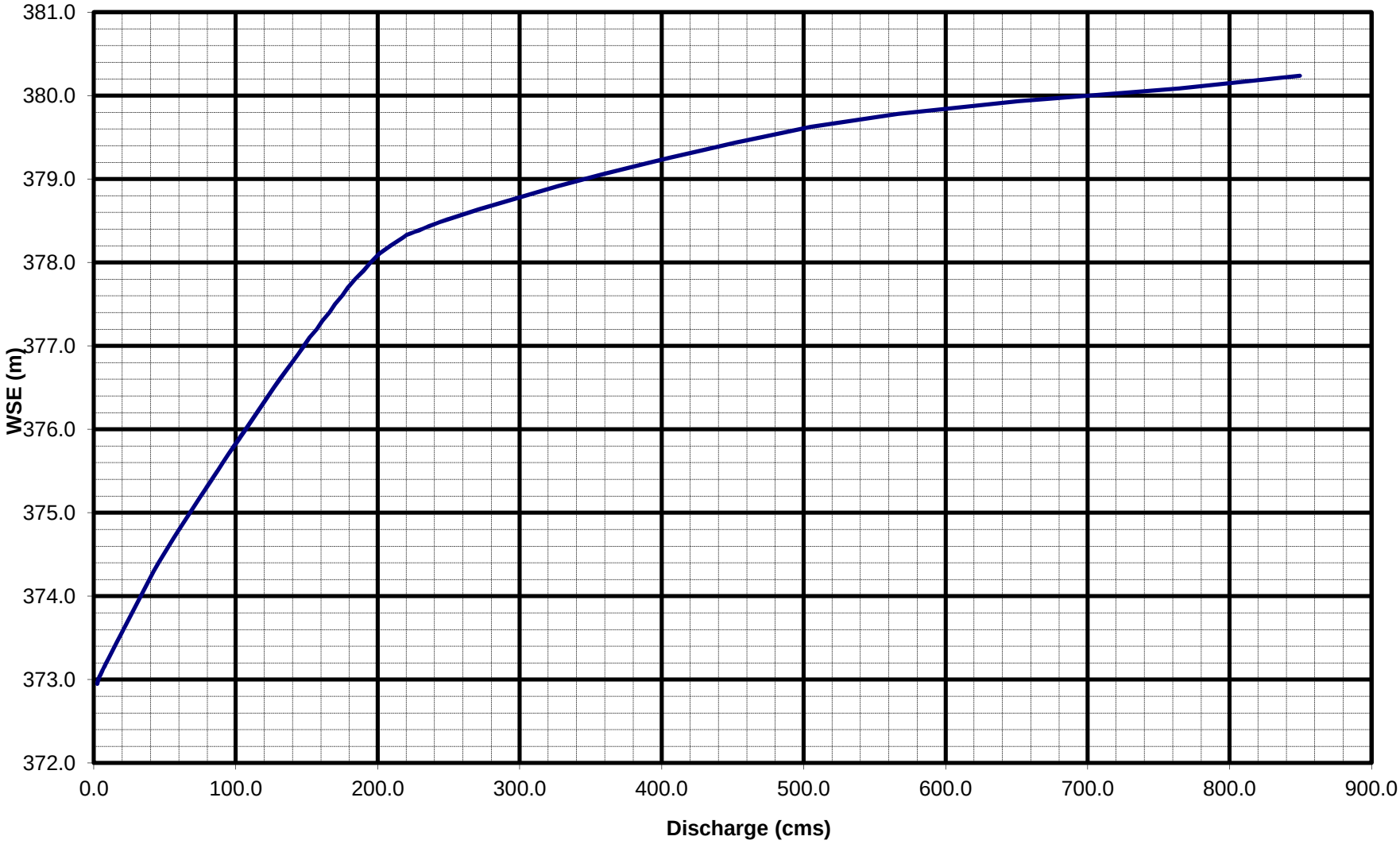
Assiniboine River near Millwood



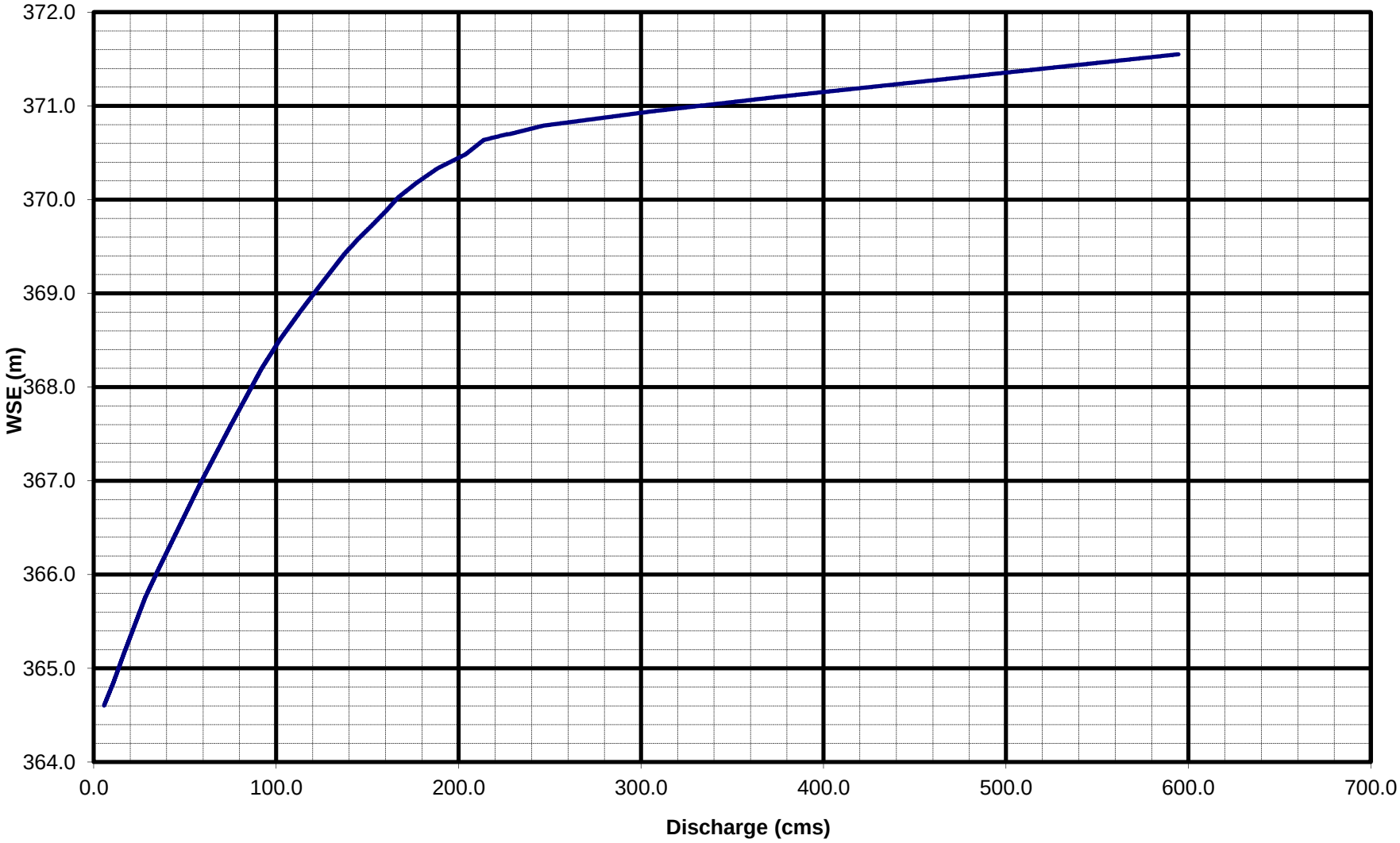
Assiniboine River at St. Lazare



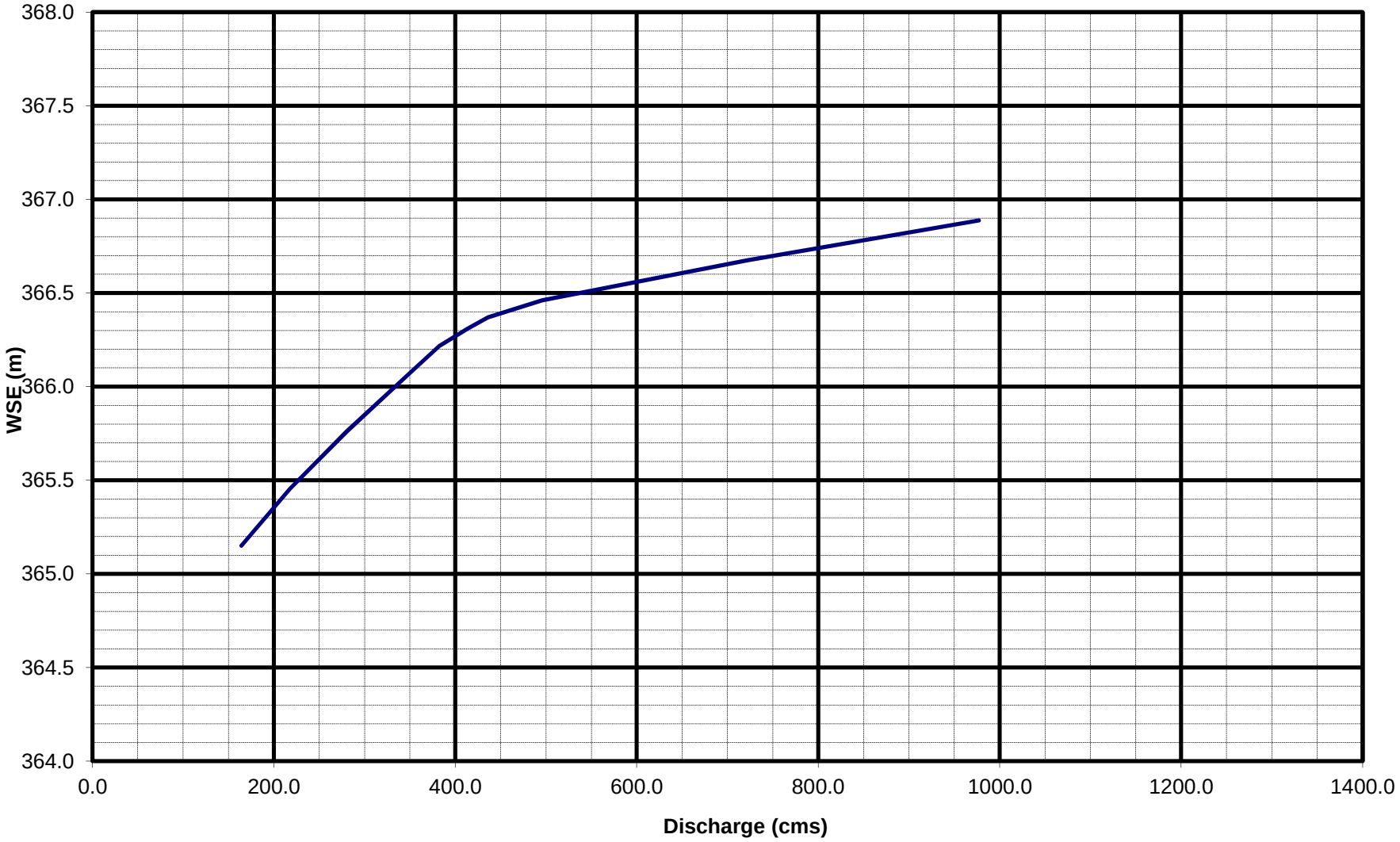
Assiniboine River near Miniota



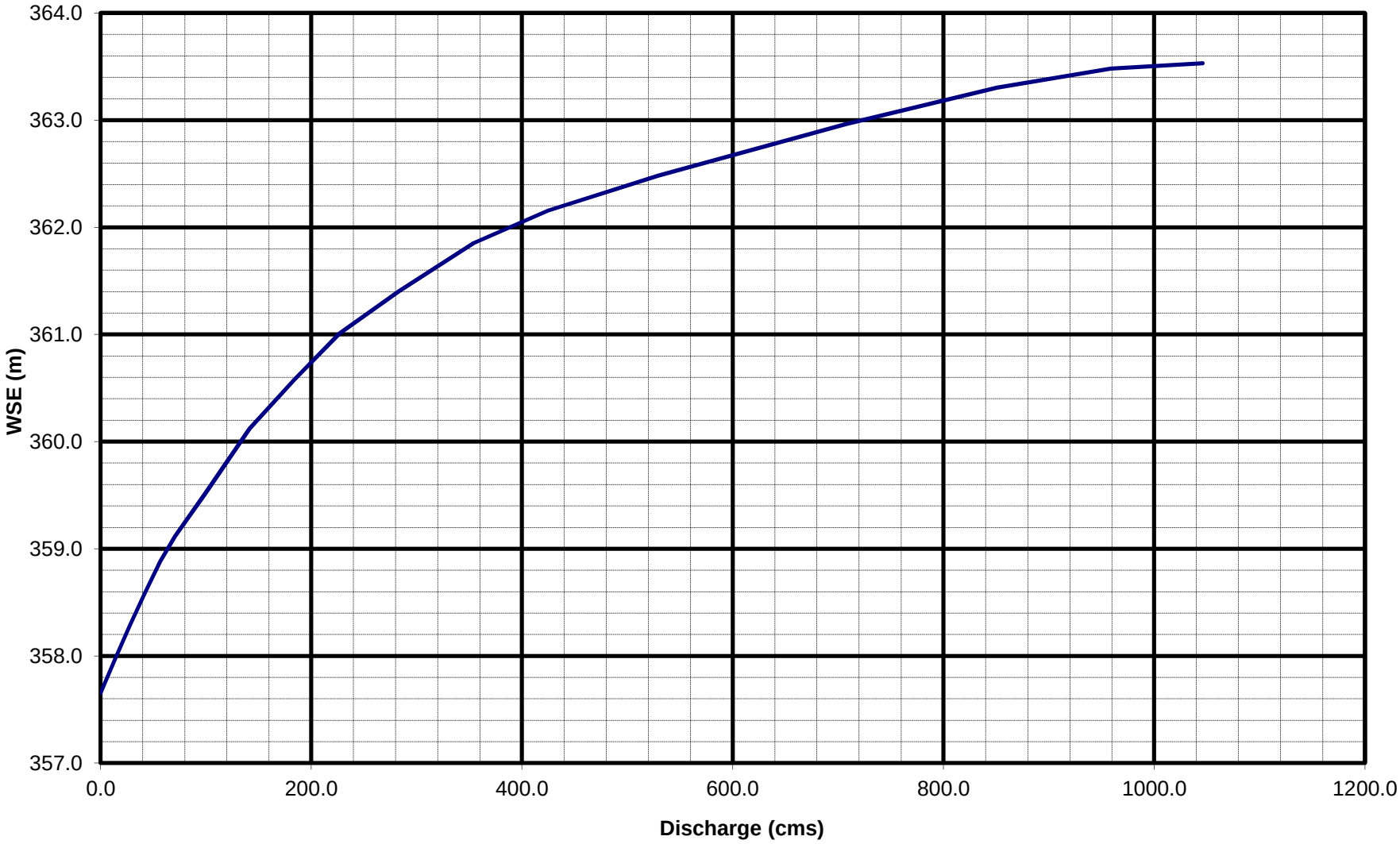
Assiniboine River near Virden



Assiniboine River near Griswold



Assiniboine River near Brandon (05MH013)



APPENDIX D2
DISCHARGE-DATE DAMAGE RELATIONSHIPS

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Discharge-Date-Crop Damage Relationship
Assiniboine Valley--Shellmouth Dam to Silver Creek

Figure 1

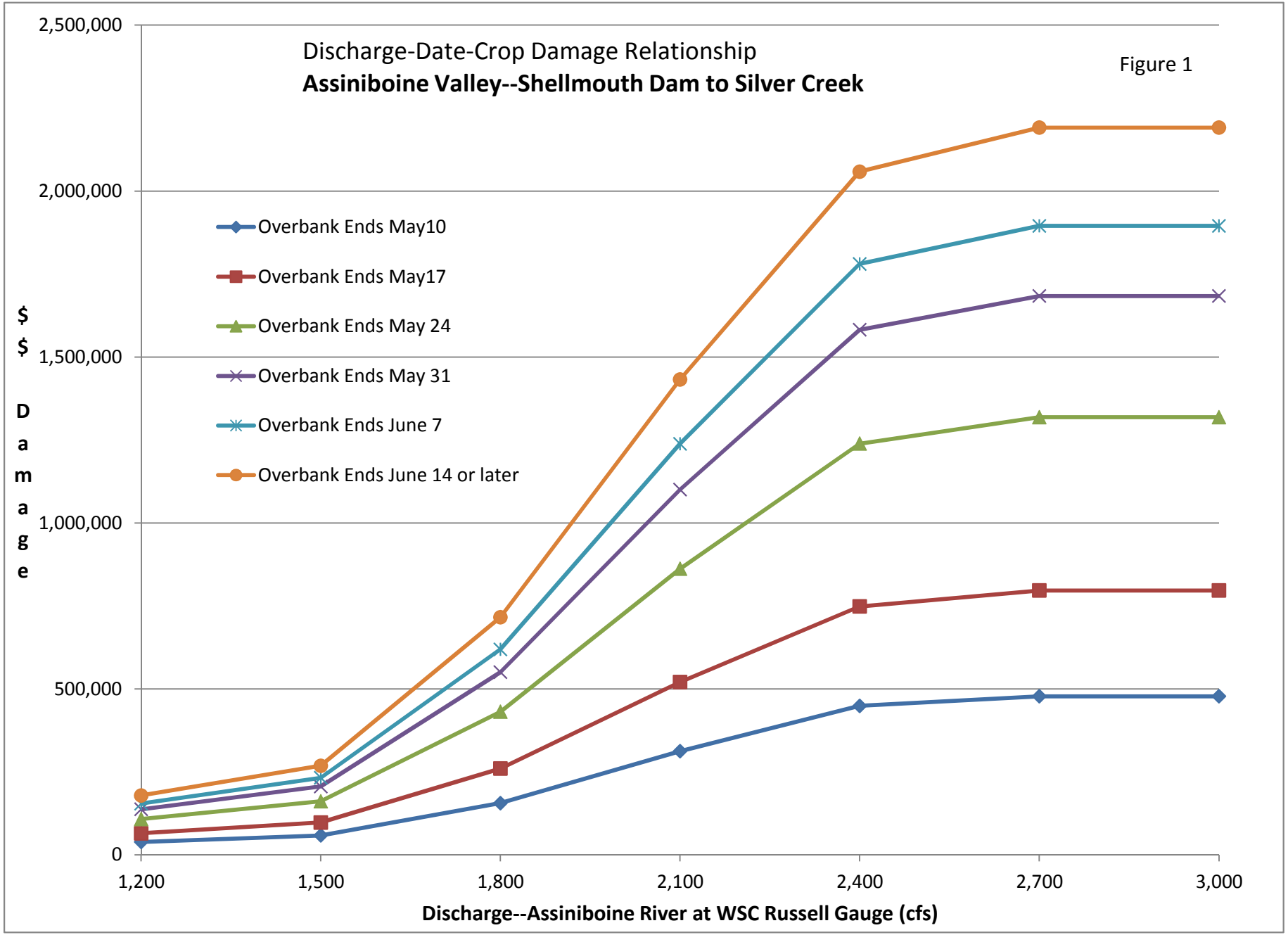
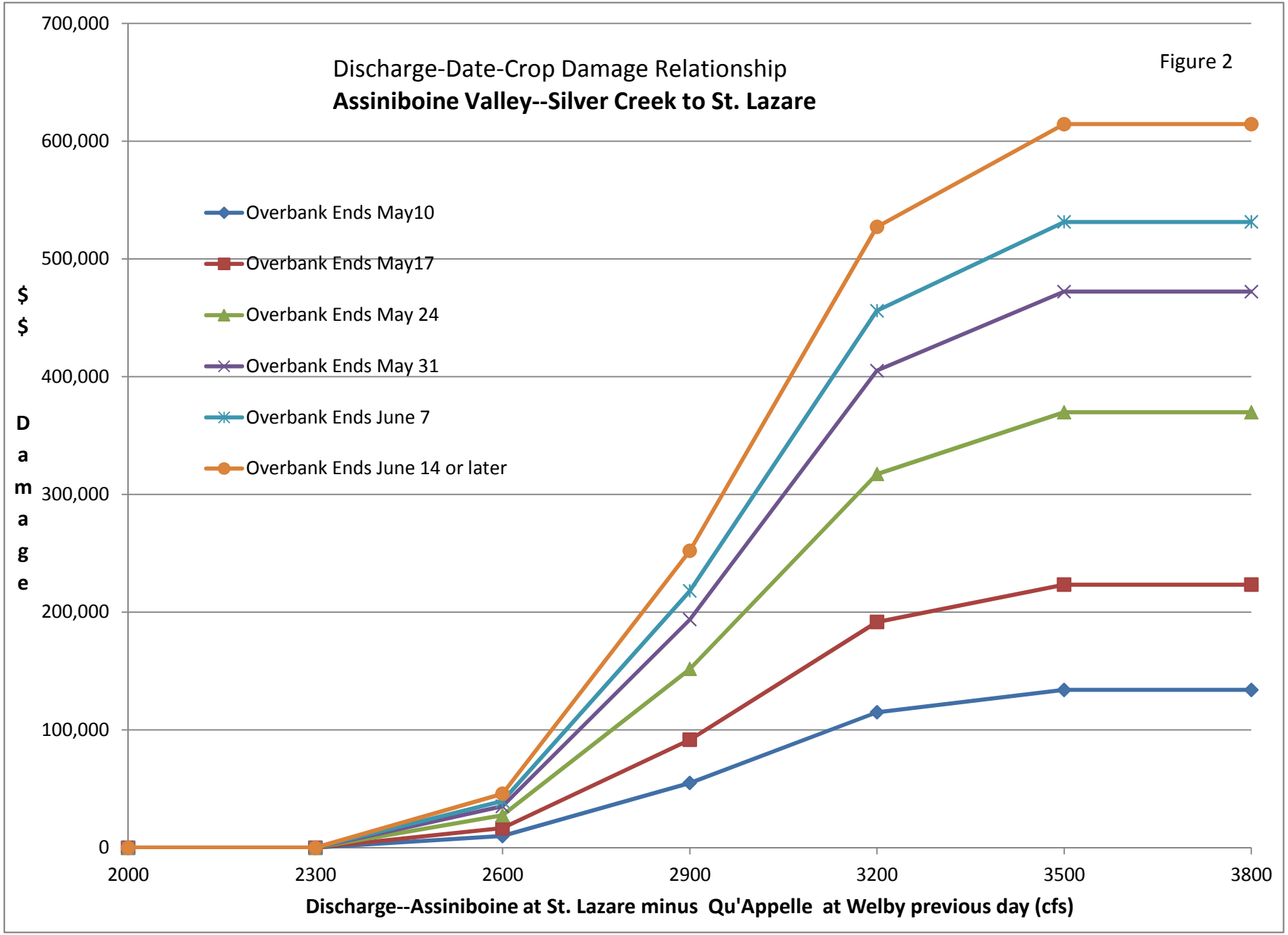
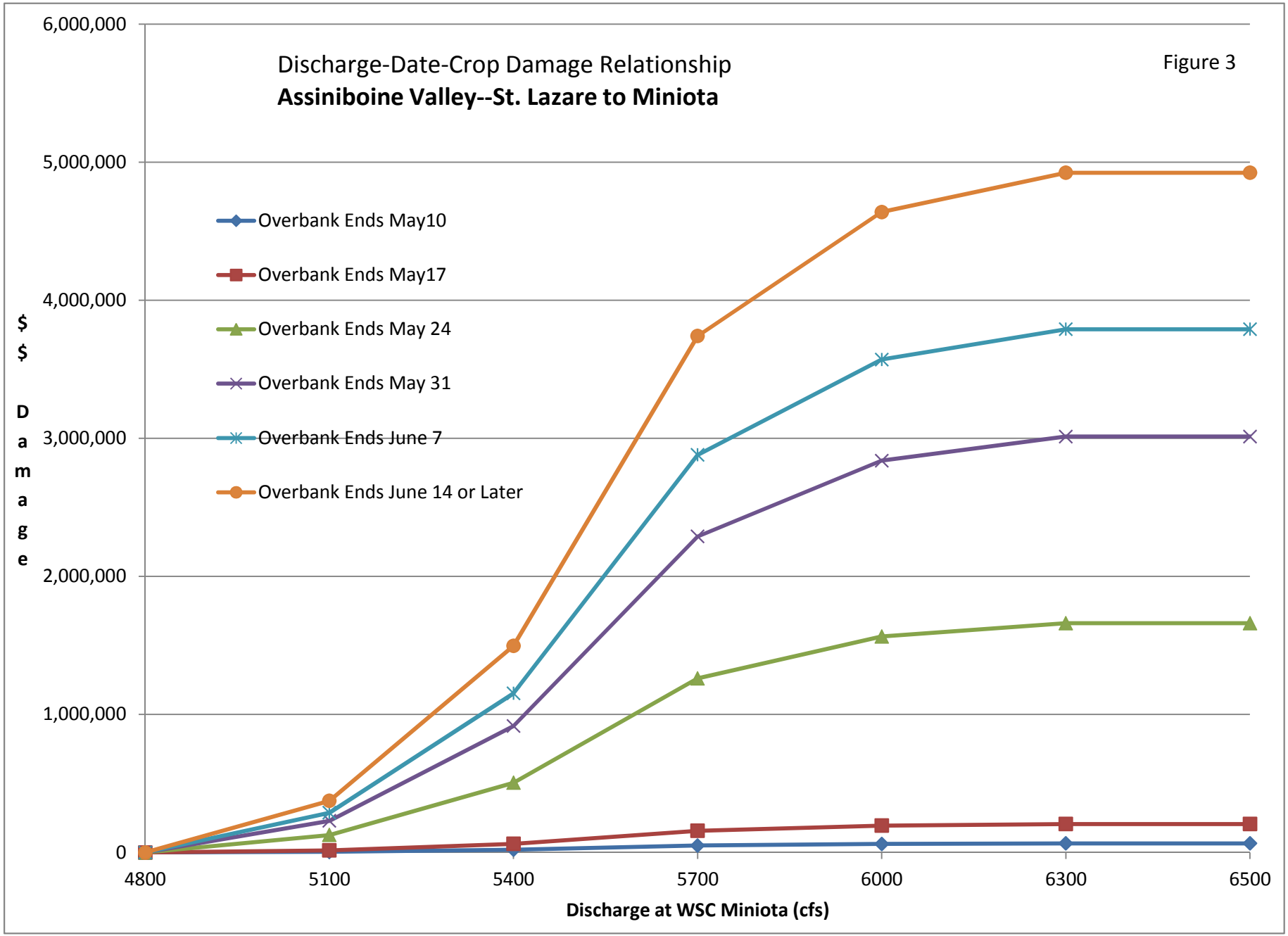


Figure 2

Discharge-Date-Crop Damage Relationship
Assiniboine Valley--Silver Creek to St. Lazare




Discharge-Date-Crop Damage Relationship
Assiniboine Valley--St. Lazare to Miniota
Figure 3
6,000,000
5,000,000
4,000,000
3,000,000
2,000,000
1,000,000
0
$ $ Damage
Overbank Ends May10
Overbank Ends May17
Overbank Ends May 24
Overbank Ends May 31
Overbank Ends June 7
Overbank Ends June 14 or Later
4800
5100
5400
5700
6000
6300
6500
Discharge at WSC Miniota (cfs)

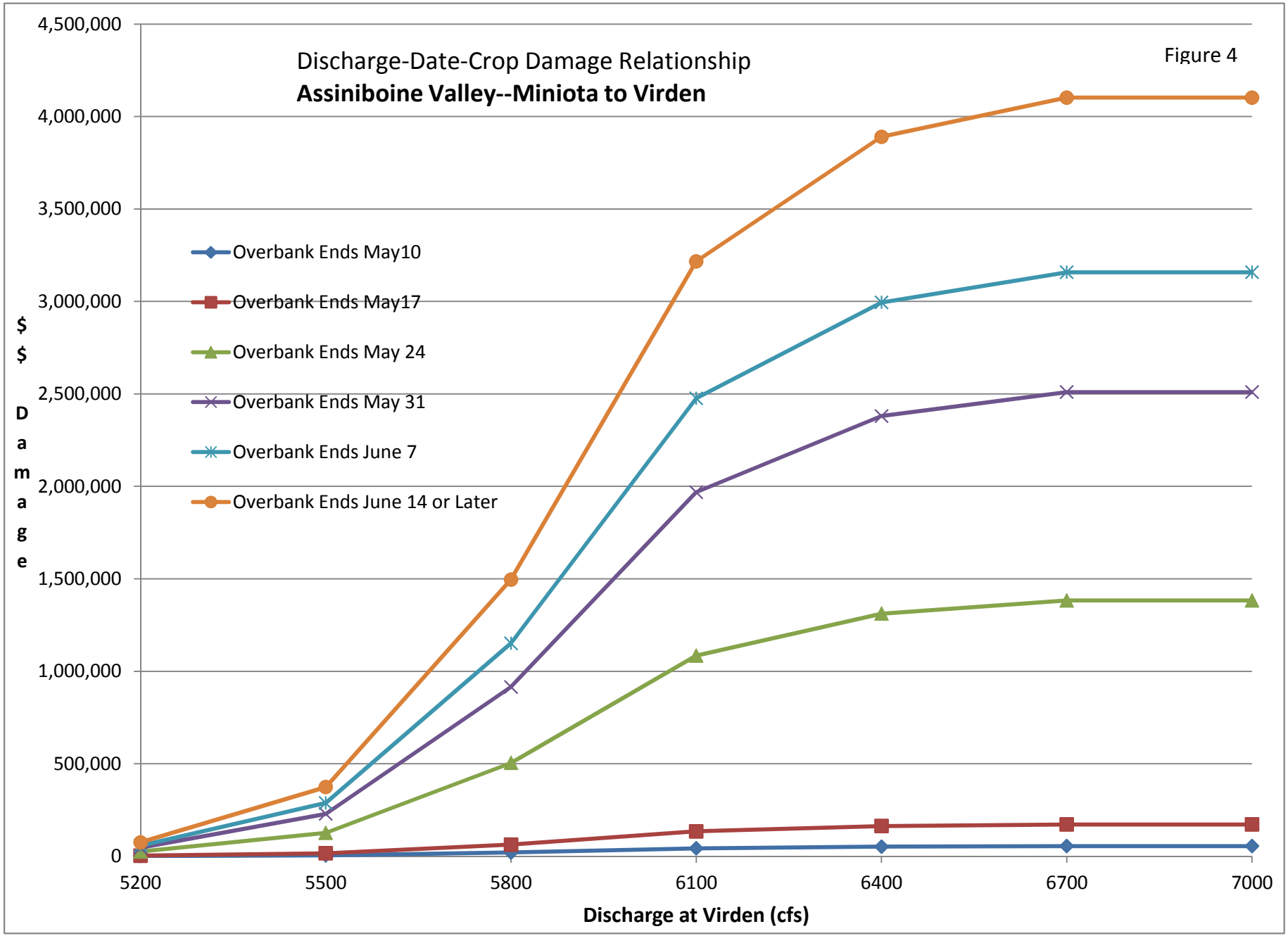

Discharge-Date-Crop Damage Relationship
Assiniboine Valley--Miniota to Virden
Figure 4
4,500,000
4,000,000
3,500,000
3,000,000
2,500,000
2,000,000
1,500,000
1,000,000
500,000
0
$ $ Damage
Overbank Ends May10
Overbank Ends May17
Overbank Ends May 24
Overbank Ends May 31
Overbank Ends June 7
Overbank Ends June 14 or Later
5200
5500
5800
6100
6400
6700
7000
Discharge at Virden (cfs)

Figure 5

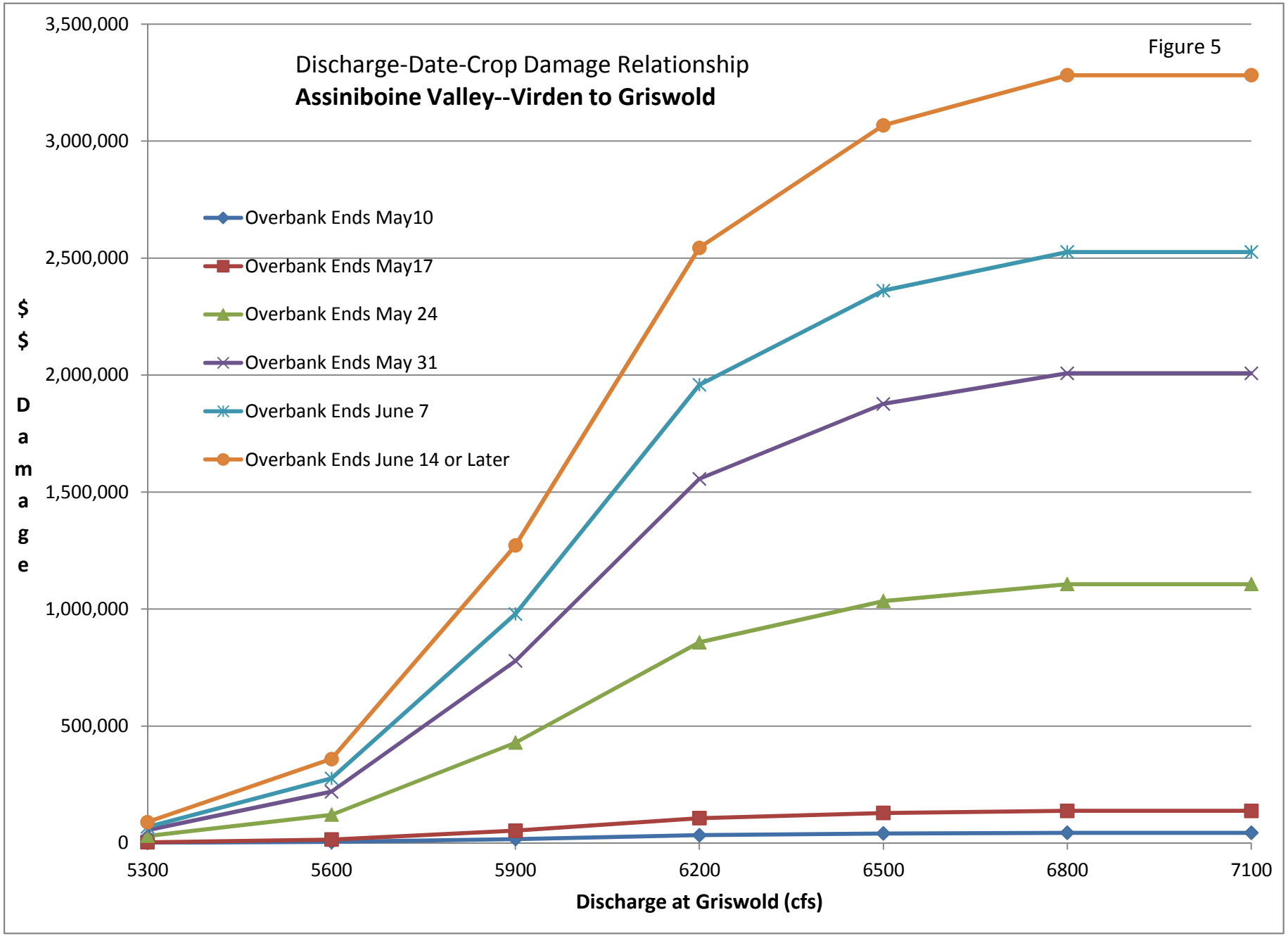
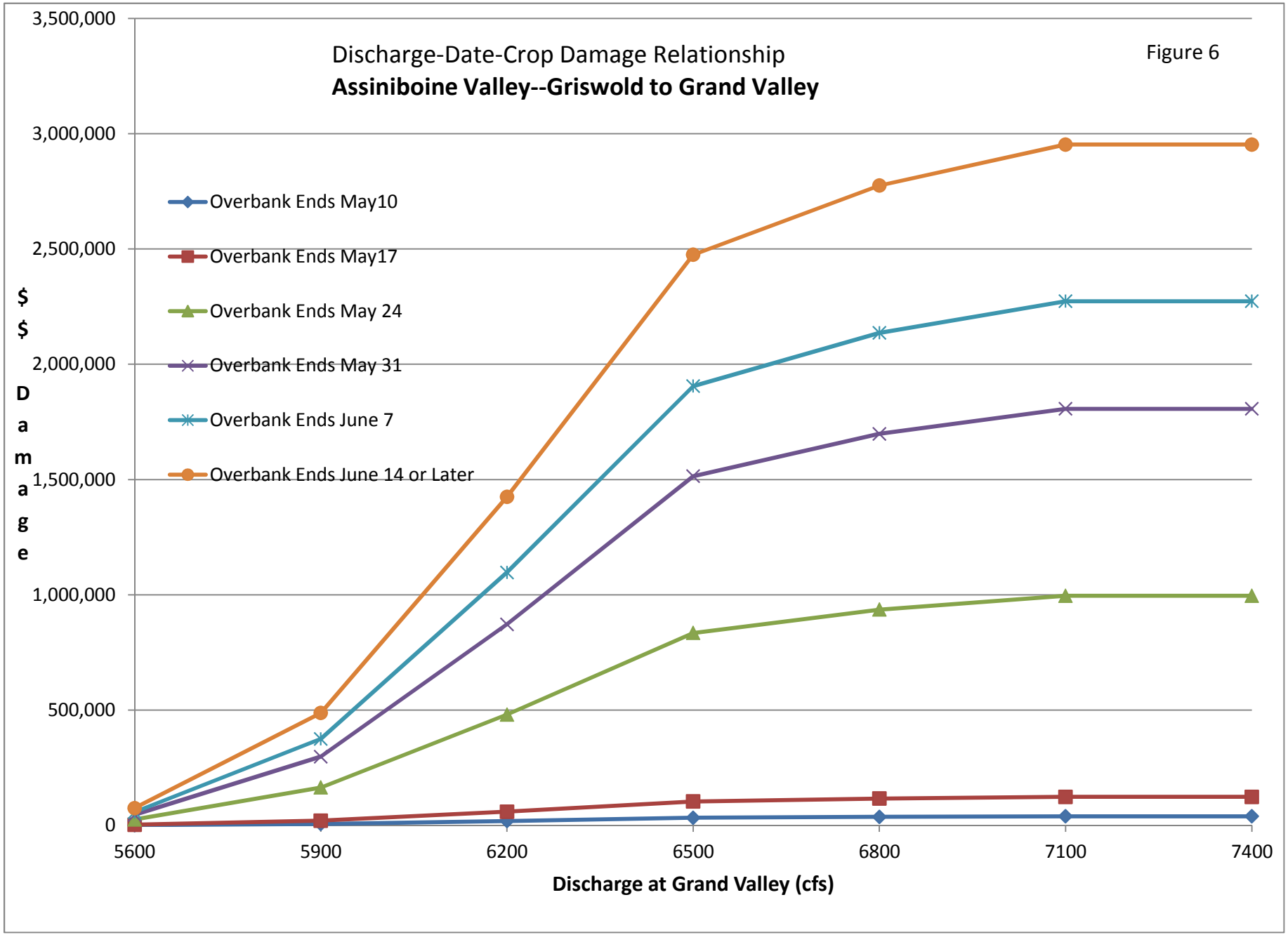

Discharge-Date-Crop Damage Relationship
Assiniboine Valley--Virden to Griswold
$ $ Damage
3,500,000
3,000,000
2,500,000
2,000,000
1,500,000
1,000,000
500,000
0
Overbank Ends May10
Overbank Ends May17
Overbank Ends May 24
Overbank Ends May 31
Overbank Ends June 7
Overbank Ends June 14 or Later
5300
5600
5900
6200
6500
6800
7100
Discharge at Griswold (cfs)

Figure 6


Discharge-Date-Crop Damage Relationship
Assiniboine Valley--Griswold to Grand Valley
3,500,000
3,000,000
2,500,000
2,000,000
1,500,000
1,000,000
500,000
0
$ $ Damage
Overbank Ends May10
Overbank Ends May17
Overbank Ends May 24
Overbank Ends May 31
Overbank Ends June 7
Overbank Ends June 14 or Later
5600
5900
6200
6500
6800
7100
7400
Discharge at Grand Valley (cfs)

APPENDIX E

SUMMARY OF 2011 FLOOD DAMAGES AND POST-FLOOD MITIGATION INITIATIVES

Summary of Data on 2011 Flood Damages & Mitigation

Program	Acronym	Source of Data	Brief Description of Program	Type of Data	Smallest Spatial Breakdown of Data for Analysis
Disaster Financial Assistance	DFA	EMO	Assisted Manitobans and communities who experienced damages from the flood of 2011	Damage	Township
2011 Excess Moisture Insurance	EMI	MASC	A basic feature of Manitoba's AgriInsurance Program, provides insurance for land that is too wet to seed.	Damage	Township
Flood 2011 - Building and Recovery Action Plan (BRAP)					
Lake Manitoba Financial Assistance Program Part A - Lake Manitoba Pasture Flooding Assistance Component	LMBA	MASC	Provided financial assistance to help Manitoba livestock producers manage their feed requirements due to the loss of pasture in the designated Lake Manitoba Flood Zone. Part A allowed producers to rent new pasture and purchase feed for animals displaced from flooded land.	Damage	Rural Municipality
Lake Manitoba Financial Assistance Program Part B - Lake Manitoba Agricultural Infrastructure, Transportation and Crop / Forage Loss Component	LMBB	MASC	Assisted agricultural crop and livestock producers in managing and recovering from the impacts of the 2011 flooding in the Lake Manitoba Flood Zone. Part B provided assistance for flood mitigation measures, lost crop production, damage to agricultural infrastructure, and extra costs for transporting livestock and feed. Producers were also compensated for debris cleanup, as well as damage to fences, corrals and crop inventory.	Damage	Rural Municipality
Lake Manitoba Financial Assistance Program Part C - Lake Manitoba Business, Principal and Non- Principal Residence Component	LMBC	MASC	Provided compensation for the cost of uninsurable property damage and flood protection measures taken as a direct result of the elevated water levels in the Lake Manitoba Flood Zone. Part C provided compensation to: principal residence owners for mitigation measures, structures, infrastructure, landscaping, contents, and temporary accommodation costs in cases of evacuation; principal residents of First Nations for compensation for loss of personal contents; non-principal residence owners for mitigation measures, building and infrastructure losses; and business owners for flood mitigation measures, building and infrastructure losses, inventory losses and income loss.	Damage	Rural Municipality
Lake Manitoba Financial Assistance Program Part D - Lake Manitoba Flood Protection for Principal Residences, Non-Principal residences and Business Structures	LMBD	MASC	Provided financial assistance for flood protection measures undertaken individually or cooperatively for the purpose of protecting principal residences, non-principal residences and business structures in the designated Lake Manitoba Flood Zone. Primarily intended to compensate for the construction of rock barriers and dikes to protect structures from flood damage, funding was also available to hire professional engineers for the design of permanent flood protection measures. Part D also covered the first $20,000 in projects eligible for the Individual Flood Protection Initiative and for the Financial Assistance for Cottage Owners Program, administered by Manitoba Infrastructure and Transportation.	Flood Mitigation	Rural Municipality
Hoop and Holler Compensation Program	H&H	MASC	Provided compensation to residents and businesses (including farms) within the designated area for property damage, income loss, and flood protection measures taken as a direct result of the controlled release of water from the Assiniboine River near Hoop and Holler Bend, or from the overflow of water diverted from the Assiniboine River into the Portage Diversion.	Damage	Rural Municipality
2011 Dauphin River Flood Assistance Program	DRFAP	MASC	Provided compensation for income losses to commercial fishers in the Dauphin River area who were unable to access the fishery and fish processing facilities due to record high water levels resulting from maximum outflows from Lake Manitoba.	Damage	Rural Municipality
2011 Lake Dauphin Emergency Flood Protection Program	LDEFPP	MASC	Provided financial assistance for emergency structural flood protection measures undertaken for the purpose of protecting principal residences and non-principal residences in the Lake Dauphin Flood Zone.	Damage	Rural Municipality
Shoal Lakes Agricultural Flooding Assistance Program	SLAFAP	MASC	Provided financial support for agricultural production losses caused by chronic flooding in the Shoal Lakes Complex in the Interlake area of Manitoba.	Damage	Rural Municipality
Lake St. Martin Fishers Program	LSMFP	MASC	Provided compensation for net income losses due to the lost fish production revenues of Lake St. Martin commercial fishers who could not participate in the 2011/12 winter commercial fishery due to ice or flooding, or because they were evacuated from the Community.	Damage	Rural Municipality
2012 Dauphin River Commercial Fishers Income Loss Assistance Program	DRCFILAP	MASC	Provided financial support to commercial fishers who experienced income losses during the 2012 summer commercial fishing season, due to lack of access to the fishery or to fish processing facilities as a direct result of elevated water levels on the Dauphin River.	Damage	Rural Municipality
2011 – Manitoba AgriRecovery Program					
2011 Manitoba Excess Moisture Assistance Program	MEMAP	MASC	Assisted crop producers affected by extreme excess moisture and flooding in 2011. Producers received assistance of $30 per acre for land that was too wet to seed by June 20, 2011, as well as for crop that was seeded and subsequently destroyed by excess moisture.	Damage	Rural Municipality
2011 Manitoba Transportation Assistance Program	MTAP	MAFRI	Assisted livestock producers who incurred extraordinary transportation costs as a result of forage shortages caused by extreme moisture conditions in 2011. Program payments were made based on the need to transport feed to livestock or livestock to feed over instances that were greater than what is normally experienced.	Damage	Regional Offices
2011 Manitoba Forage Shortfall Assistance Program	MFSAP	MAFRI	Assisted livestock producers who incurred extraordinary costs to maintain the feed requirement for their livestock herds as a result of the forage shortage caused by flooding or excess moisture conditions in 2011. Program assistance was based on covering shortfalls that would have occurred over the 2011/12 pasture and winter feeding periods.	Damage	Regional Offices
2011 Manitoba Forage Restoration Assistance Program	MFRAP	MASC	Assisted forage producers to restore established tame forage and forage seed damaged by excess moisture in 2011. Producers were eligible for compensation of $50 per acre to help re-establish forage and forage seed crops. Producers were required to destroy and reseed the forage crop prior to compensation being paid.	Damage	Provincial
2011 Manitoba Greenfeed Assistance Program	MGAP	MAFRI	Assisted producers to increase livestock feed production by seeding and harvesting greenfeed. Producers who seeded greenfeed on cultivated acres that were unseeded due to excess moisture received $15 per acre for acres over and above their normal amount of greenfeed acres.	Other	n/a
2011 Manitoba Infrastructure and Individual Assessment Program	MIIAP	MASC	Assisted crop and livestock producers in recovering from flood losses to agricultural property and inventory that were not eligible for Disaster Financial Assistance or for the Flood 2011 – Building and Recovery Action Plan. Eligible producers were compensated for measures taken to mitigate flood damage of agricultural property and inventory, and for agricultural property damage caused by flooding.	Flood Mitigation & Damages	Provincial
Individual Flood Proofing Initiative & Financial Assistance for Cottage Owners	IFPI & FACO	MIT	Provides financial assistance to owners of flood prone homes, farms, businesses and cottages throughout Manitoba to flood protect their buildings and structures by raising, moving or diking.	Flood Mitigation	Township
2011 Community Flood Protection Program	CFPP	MIT	The 2011 Community Flood Protection Program will see the construction of permanent community flood protection infrastructure, in particular in those municipalities which have been impacted by 2011 flooding, especially flooding which occurred in the Assiniboine River basin and around Lake Manitoba.	Flood Mitigation	Varies

Notes:
1 The programs which have the type of data identified as "Damage" provided financial assistance for losses due to the 2011 flood such as flood fighting costs, damage to property, loss of income…
2 The programs which have the type of data identified as "Flood Mitigation" provided financial assistance for flood protection measures undertaken for the purpose of protecting against future floods.
3 The programs which have the type of data identified as "Other" provided financial assistance for purposes other than those stated above.

APPENDIX F

LEGACY TOOLS

LIST OF LEGACY TOOLS

Item	Type	Region	Format
1	Flood Damage Models	Dauphin Lake	Excel
		Lake Manitoba	Excel
		Lower Assiniboine River	Excel
		Upper Assiniboine River	Excel
		Souris River	Excel
2	Monte Carlo Models	Lake Manitoba	Excel
		City of Winnipeg	MAT Lab
3	Flood Routing Models	Souris River	Excel
		Upper Assiniboine River	Excel
		Lake Manitoba, Lake St. Martin	Excel
		Lake Winnipegosis	Excel
4	Backwater Models	Portage Diversion	HEC-RAS
		Mossy River	HEC-RAS
		Assiniboine River near Brandon	HEC-RAS
		Lower Assiniboine River (2 Parts)	HEC-RAS
		City of Winniepg	HEC-RAS
		Fairford River	HEC-RAS
5	Wind Set-up Model	Lake Manitoba	Fortran
6	Bathymetric Data	Lake Manitoba	GIS
7	Background Reports	Various	PDF

Legacy tools and documentation for use provided to MIT separately.

APPENDIX G

LOWER ASSINIBOINE RIVER DIKE MAPS

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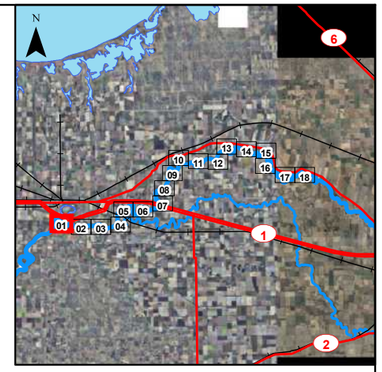


LEGEND:

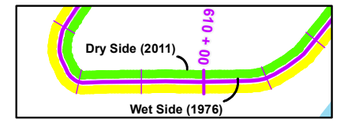
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:

- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400

Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	14/05/05	ISSUED FOR INFORMATION	MSW

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 1 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

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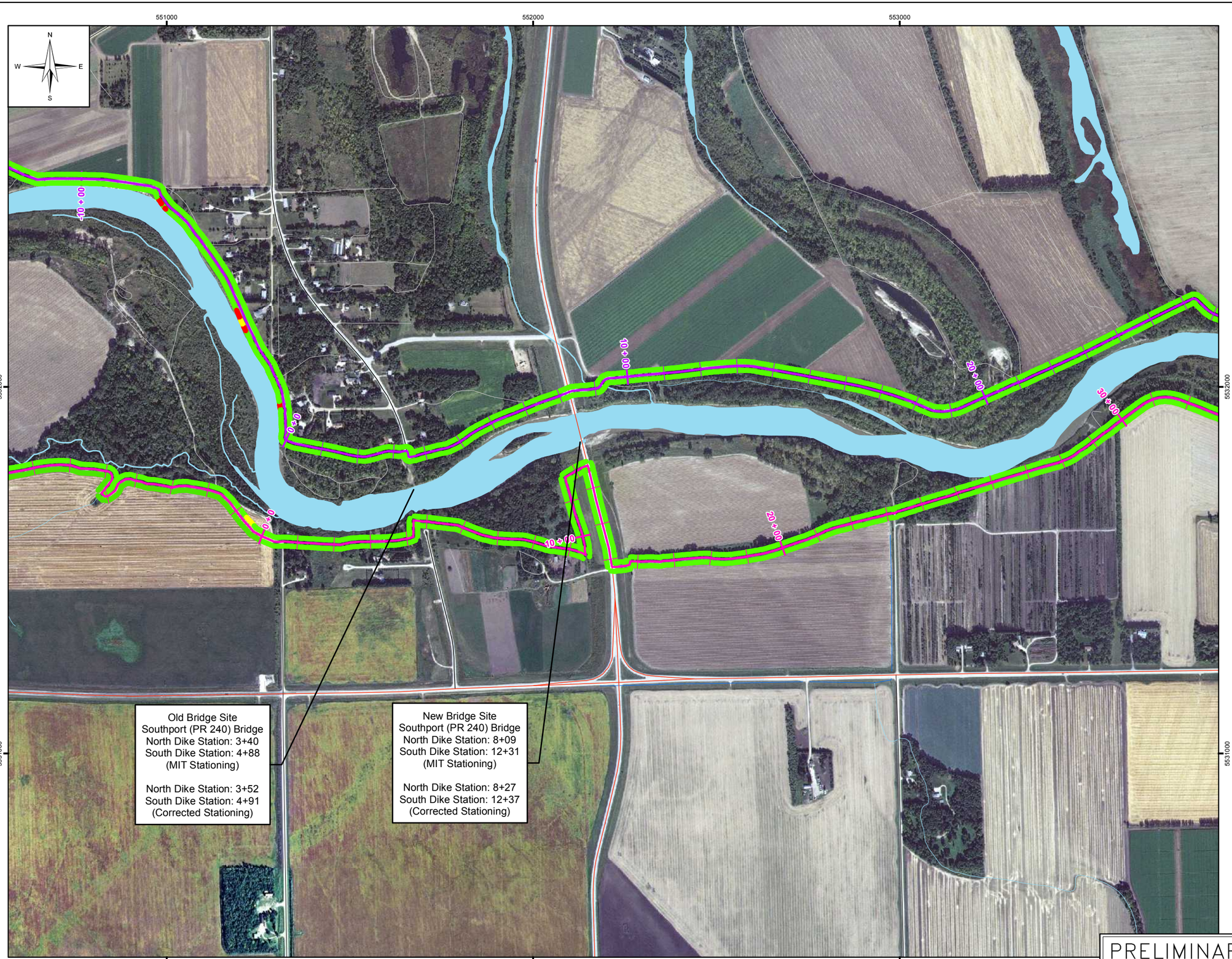
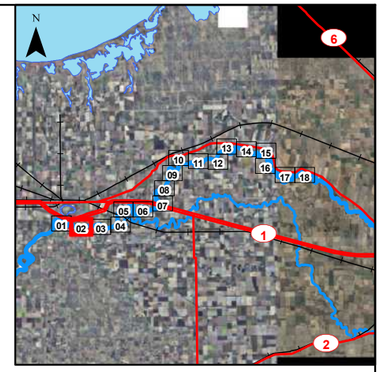




LEGEND:

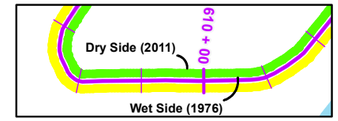
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:

- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400

Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	14/05/05	ISSUED FOR INFORMATION	MSW

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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

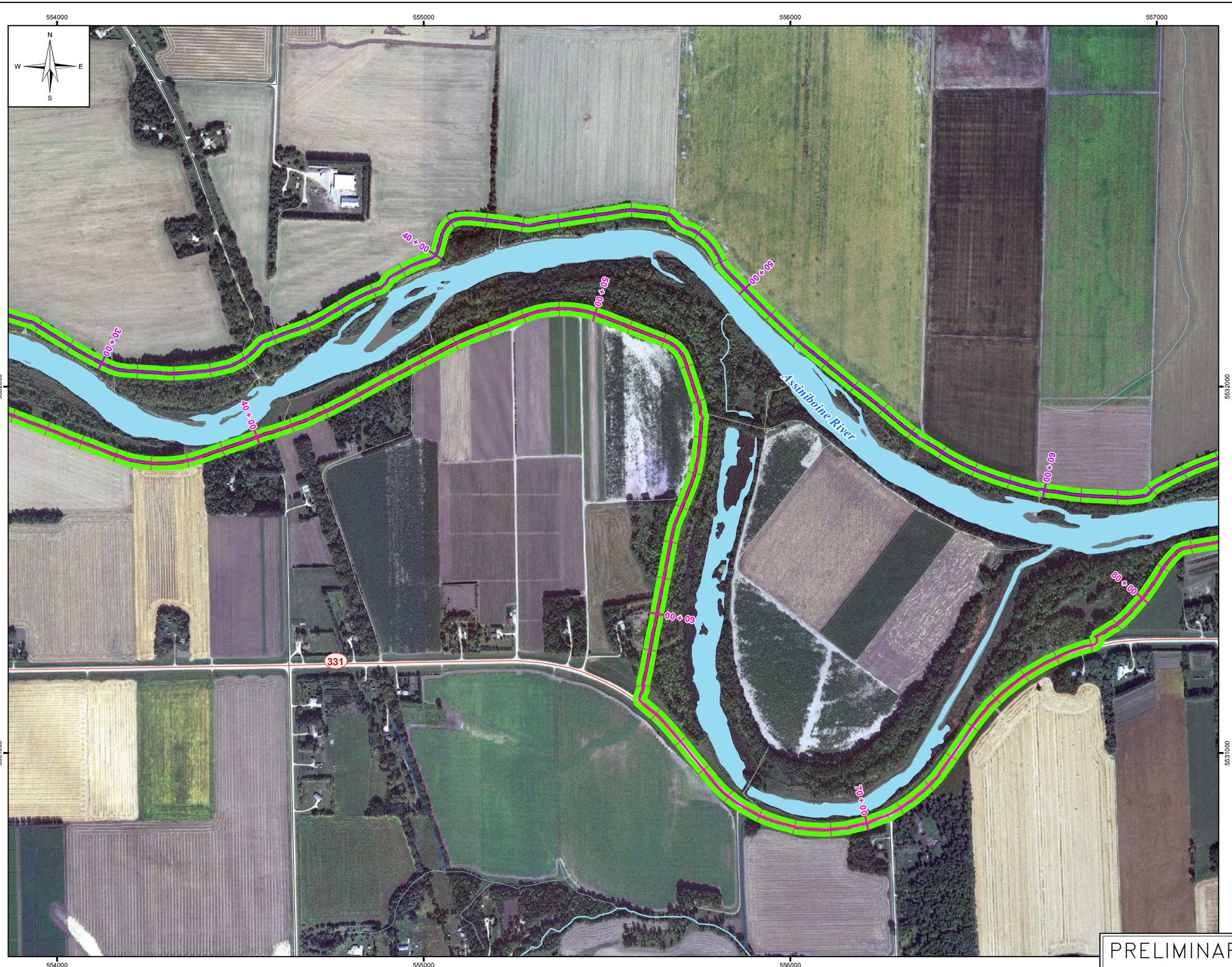
FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 2 OF 18

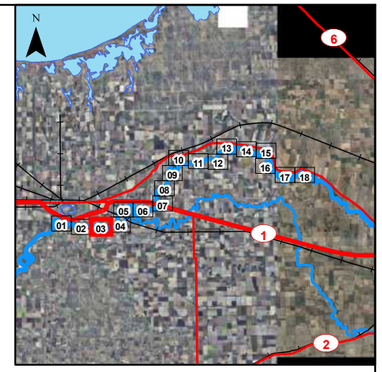
PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

MAY 2014 | FIGURE 01 | REV: A

FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

Portions of data presented are owned by the Province of Manitoba and are produced under the licence agreement with the Province of Manitoba 2013 Queen's Printer.





LEGEND:

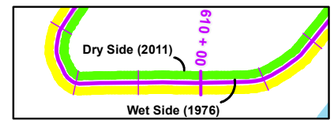
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:

- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400

Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	14/05/05	ISSUED FOR INFORMATION	MSW

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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 3 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
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11"x17" PLOT SCALE 1:1

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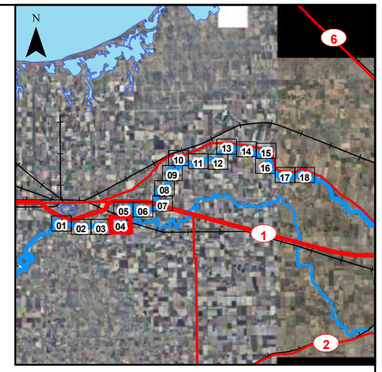


LEGEND:

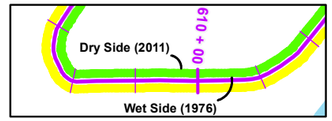
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:

- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400

Metres

SCALE: 1:10,000 METRIC 11"x17"

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 4 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

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11"x17" PLOT SCALE 1:1

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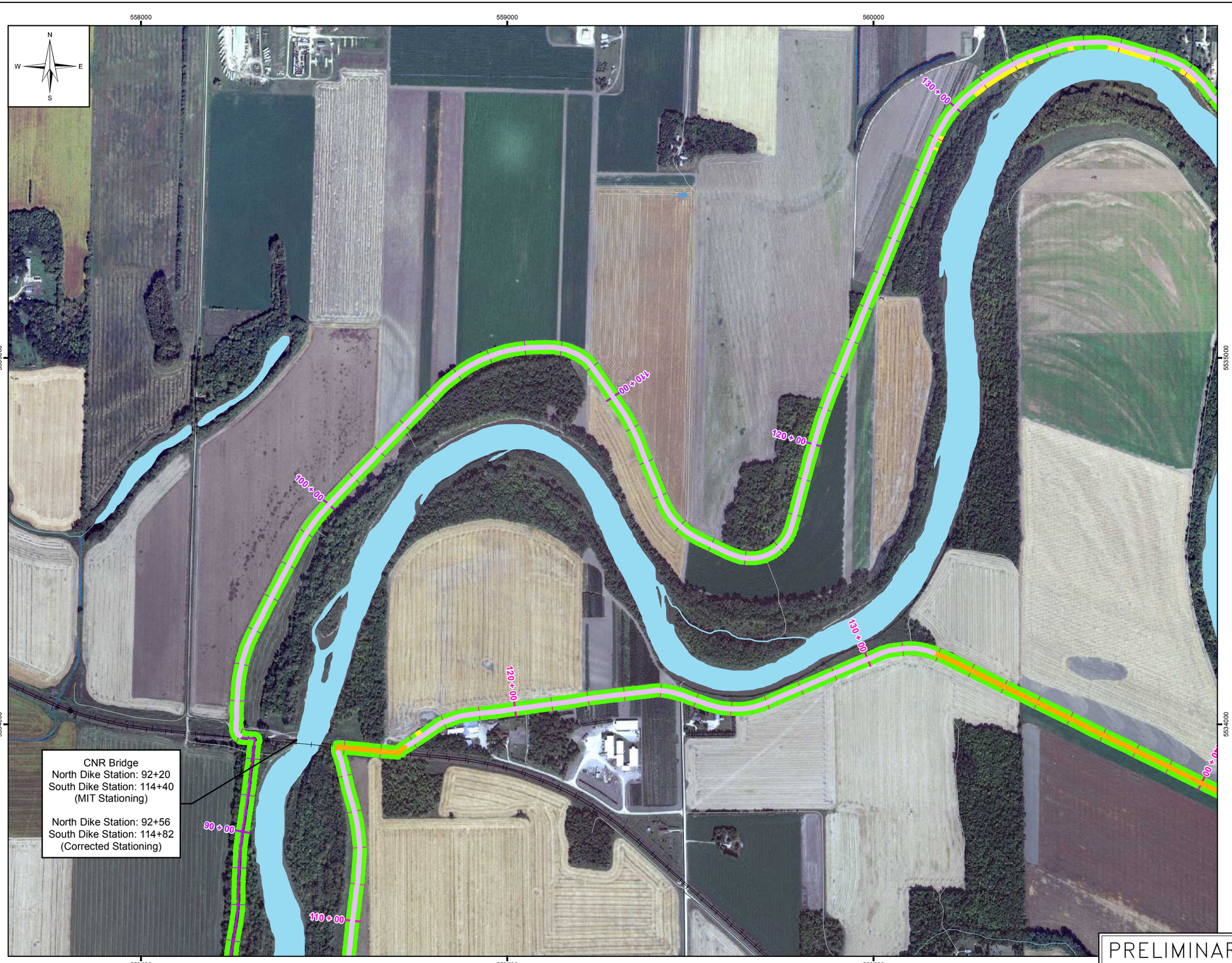
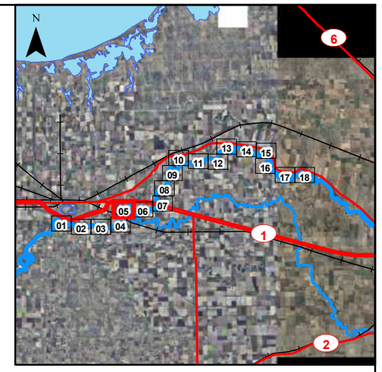




LEGEND:

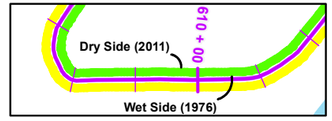
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:
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DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 5 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

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11"x17" PLOT SCALE 1:1

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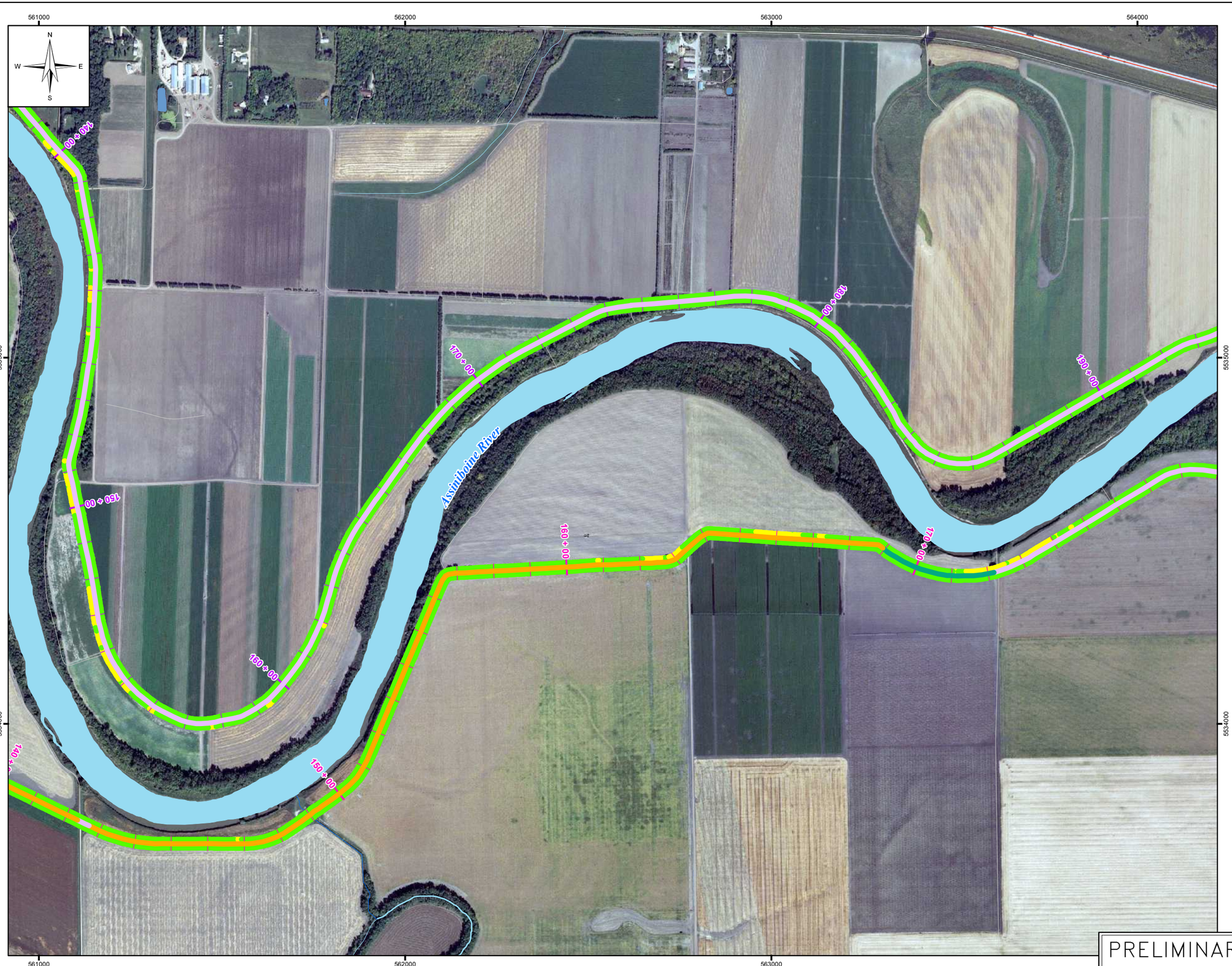
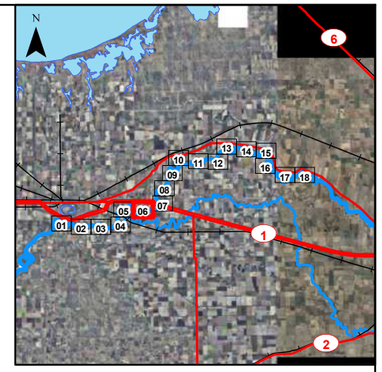




LEGEND:

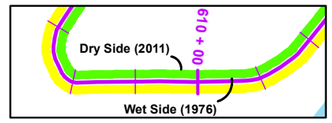
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:

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DRAFT

100 0 100 200 300 400

Metres

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 6 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
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11"x17" PLOT SCALE 1:1

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565000 566000 567000

5536000

5535000

N W E S

210 + 00

210 + 00

200 + 00

200 + 00

190 + 00

190 + 00

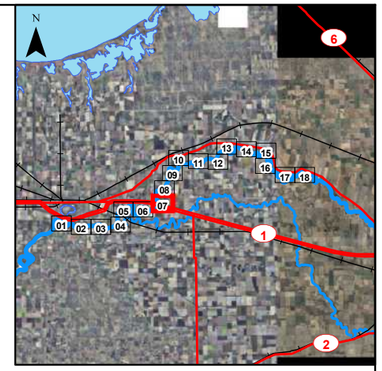
1

PTH 1W Bridge
North Dike Station: 202+20
South Dike Station: 195+50
(MIT Stationing)

North Dike Station: 202+77
South Dike Station: 196+00
(Corrected Stationing)

PTH 1E Bridge
North Dike Station: 200+00
South Dike Station: 190+50
(MIT Stationing)

North Dike Station: 200+48
South Dike Station: 191+05
(Corrected Stationing)



LEGEND:

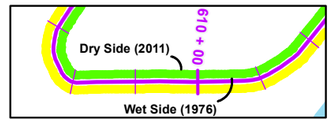
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:
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DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 7 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

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11"x17" PLOT SCALE 1:1

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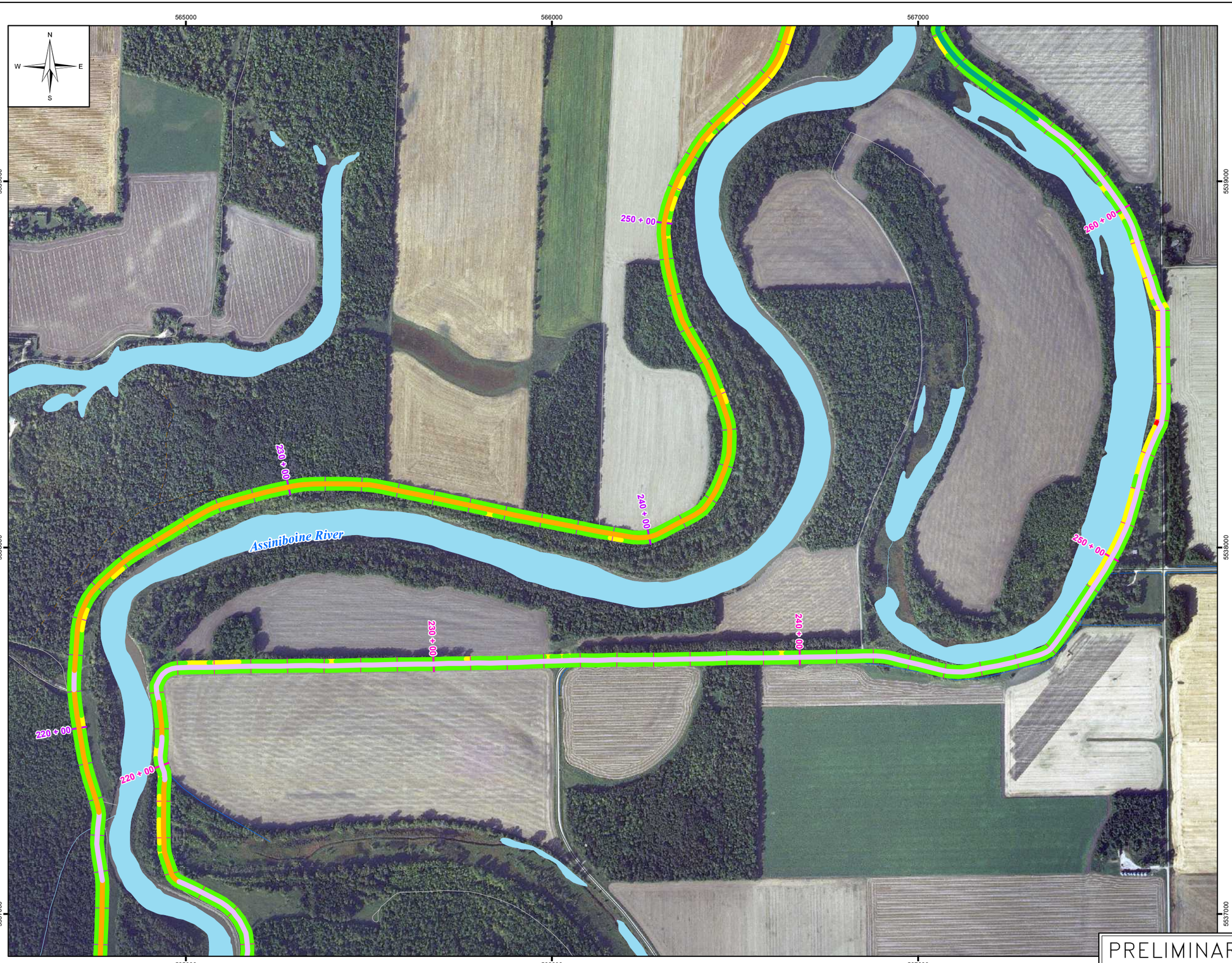
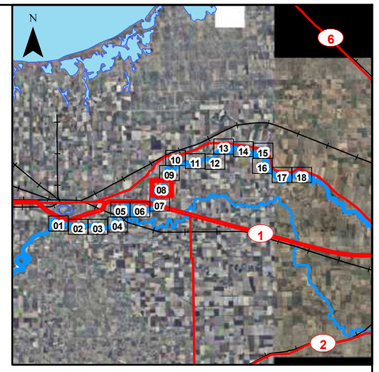




LEGEND:

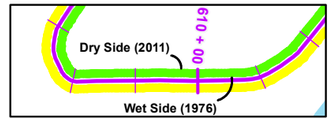
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



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DRAFT

100 0 100 200 300 400

Metres

SCALE: 1:10,000 METRIC 11"x17"

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 8 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

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11"x17" PLOT SCALE 1:1

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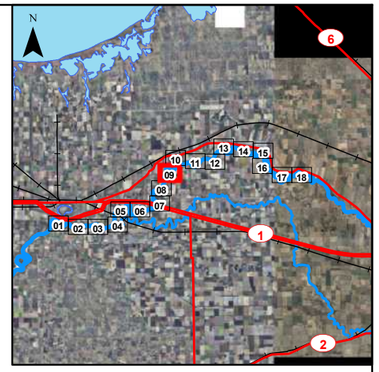


LEGEND:

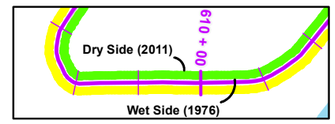
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:
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DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

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Elevations are in metres above sea level (MSL)

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 9 OF 18

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

MAY 2014 | FIGURE 01 | REV: A

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11"x17" PLOT SCALE 1:1

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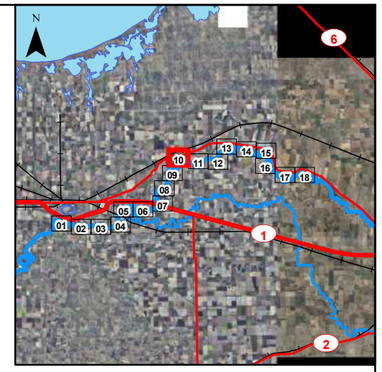


LEGEND:

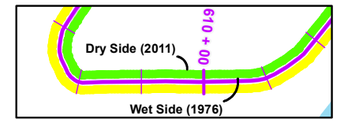
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:

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DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 10 OF 18

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

MAY 2014 | FIGURE 01 | REV: A

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11"x17" PLOT SCALE 1:1

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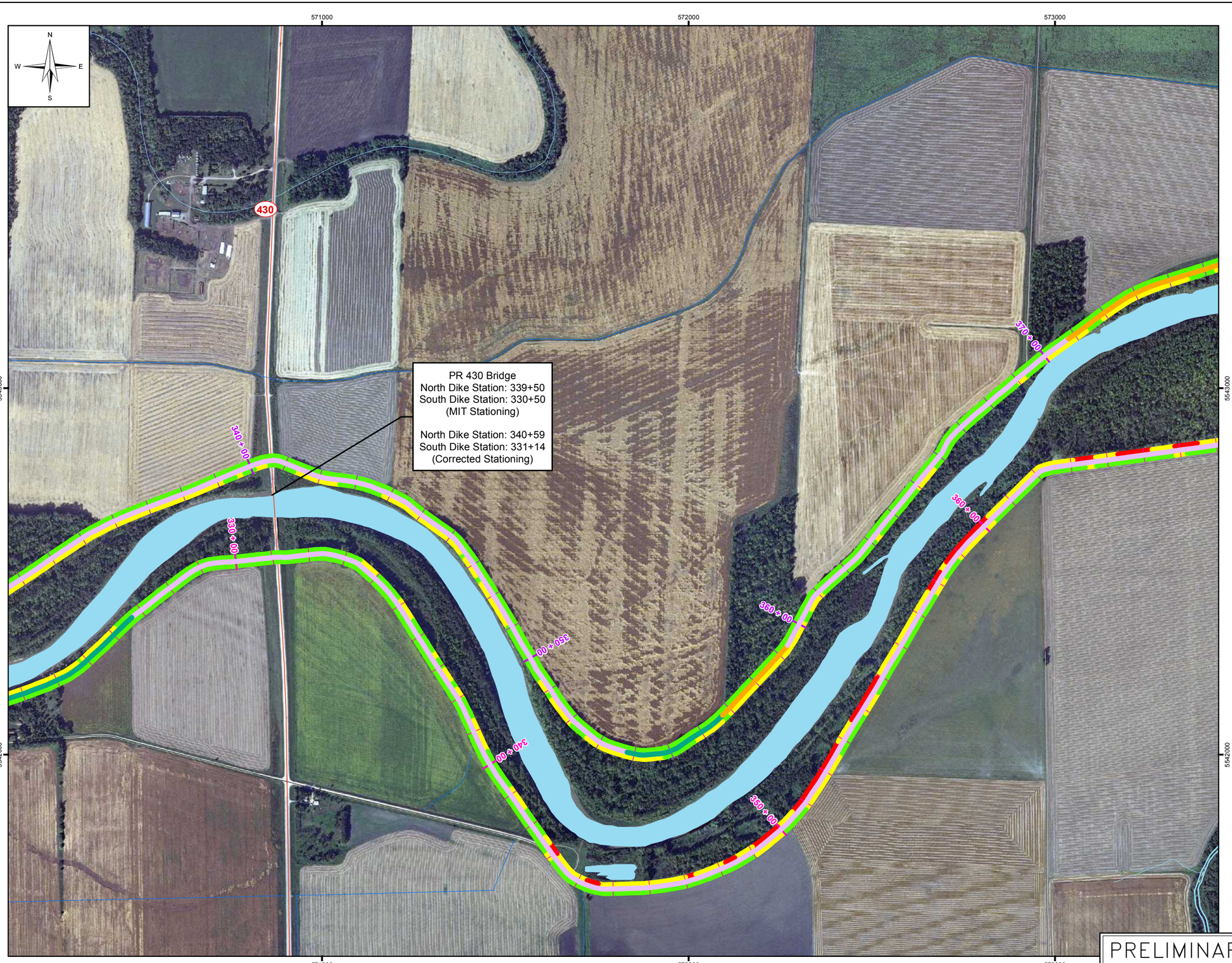

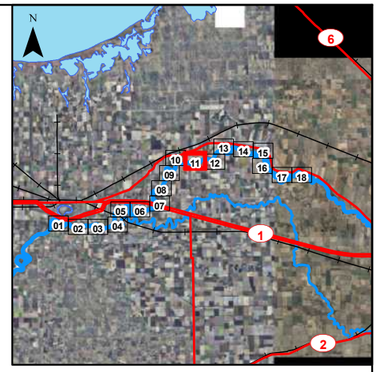

LEGEND:

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Land Ownership

- Easement
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Freeboard Conditions

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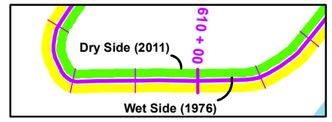

NOTE:
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DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS

SHEET 11 OF 18

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

MAY 2014 | FIGURE 01 | REV: A

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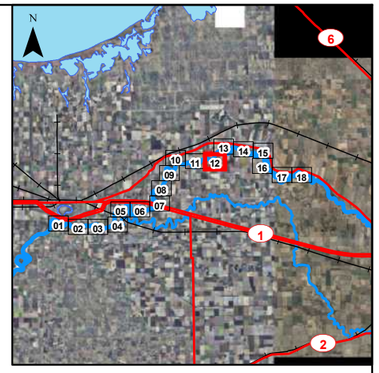

LEGEND:

- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

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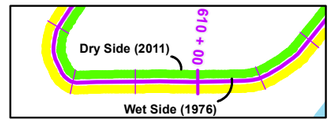

NOTE:

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DRAFT

100 0 100 200 300 400

Metres

SCALE: 1:10,000 METRIC 11"x17"

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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 12 OF 18

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

MAY 2014 | FIGURE 01 | REV: A

FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

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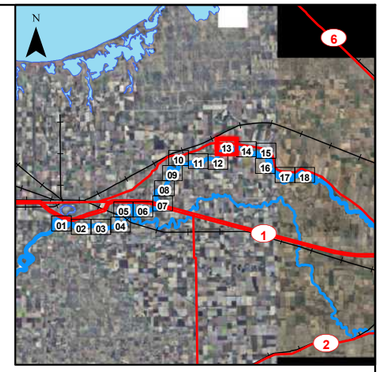


LEGEND:

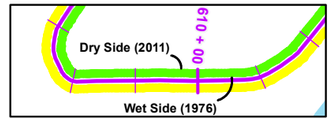
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:

- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400

Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	14/05/05	ISSUED FOR INFORMATION	MSW

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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 13 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

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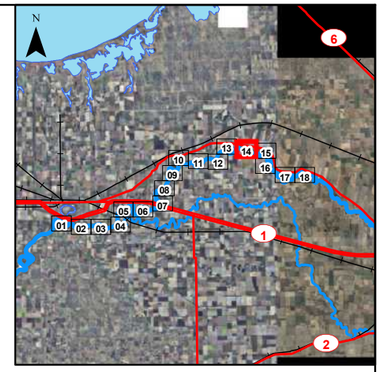


LEGEND:

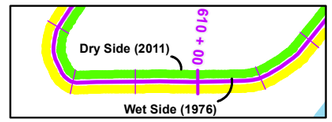
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:

- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 14 OF 18

MAY 2014 | FIGURE 01 | REV: A

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FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

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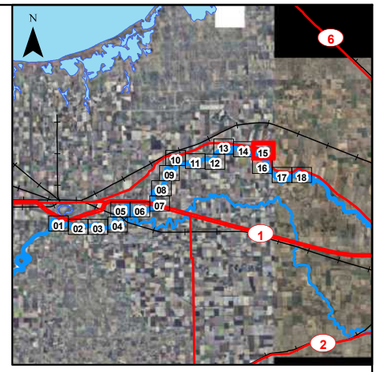


LEGEND:

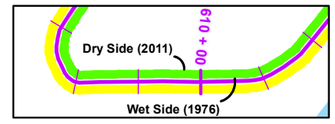
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:
- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	14/05/05	ISSUED FOR INFORMATION	MSW

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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 15 OF 18

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

MAY 2014 | FIGURE 01 | REV: A

FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

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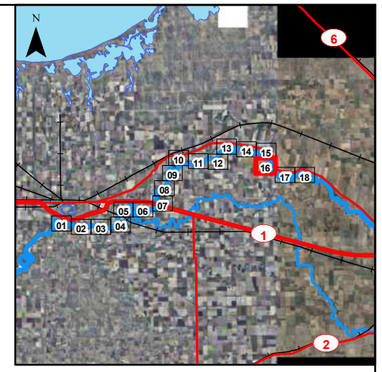


LEGEND:

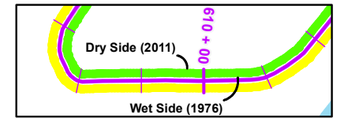
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:
- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	14/05/05	ISSUED FOR INFORMATION	MSW

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 16 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

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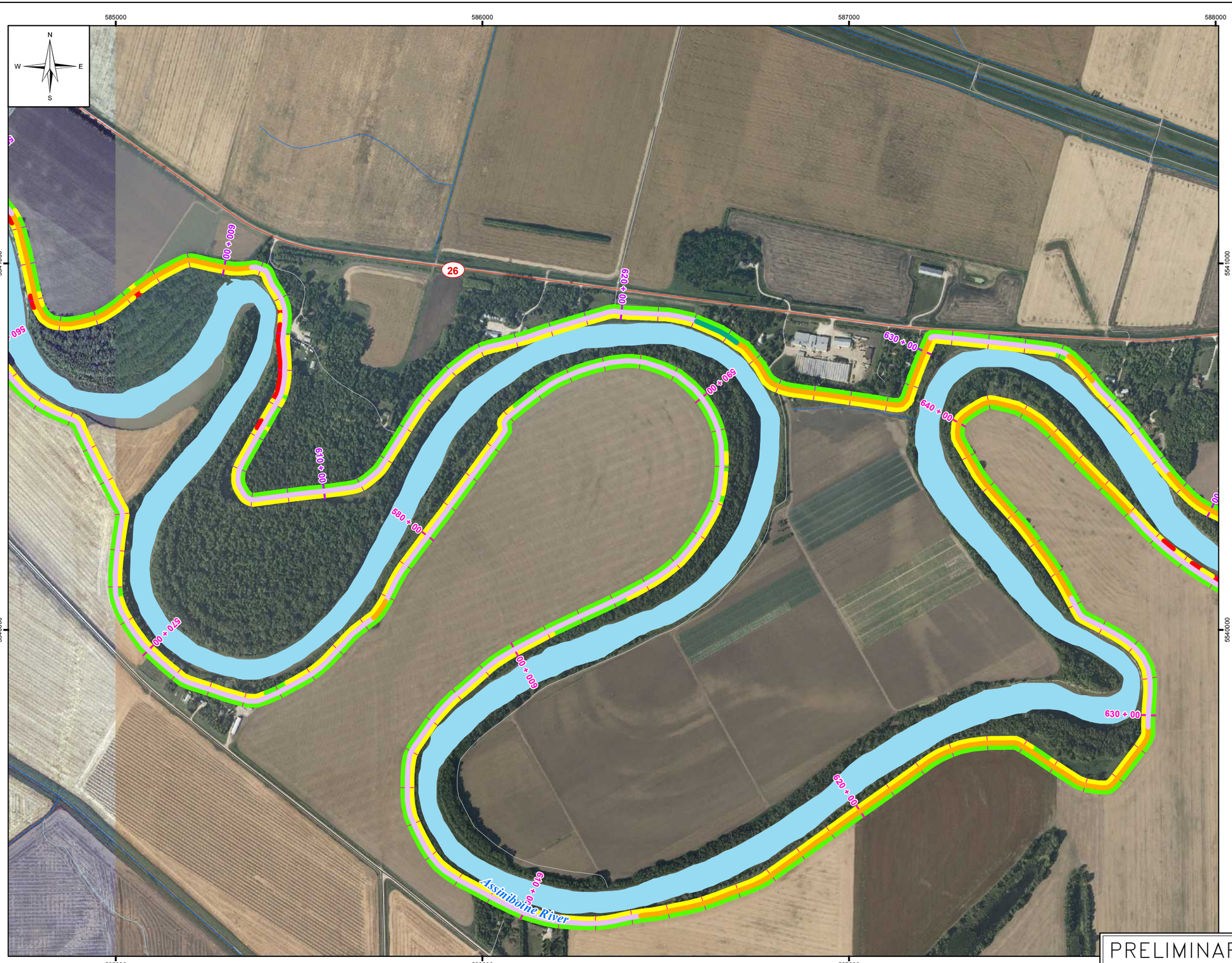
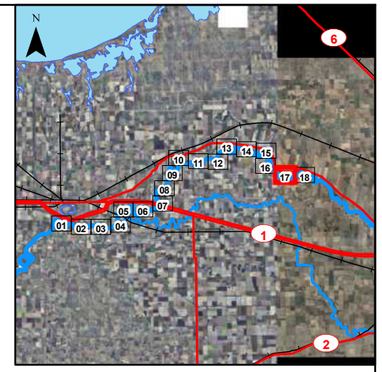




LEGEND:

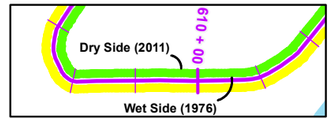
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:
- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	14/05/05	ISSUED FOR INFORMATION	MSW

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 17 OF 18

MAY 2014 | FIGURE 01 | REV: A

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

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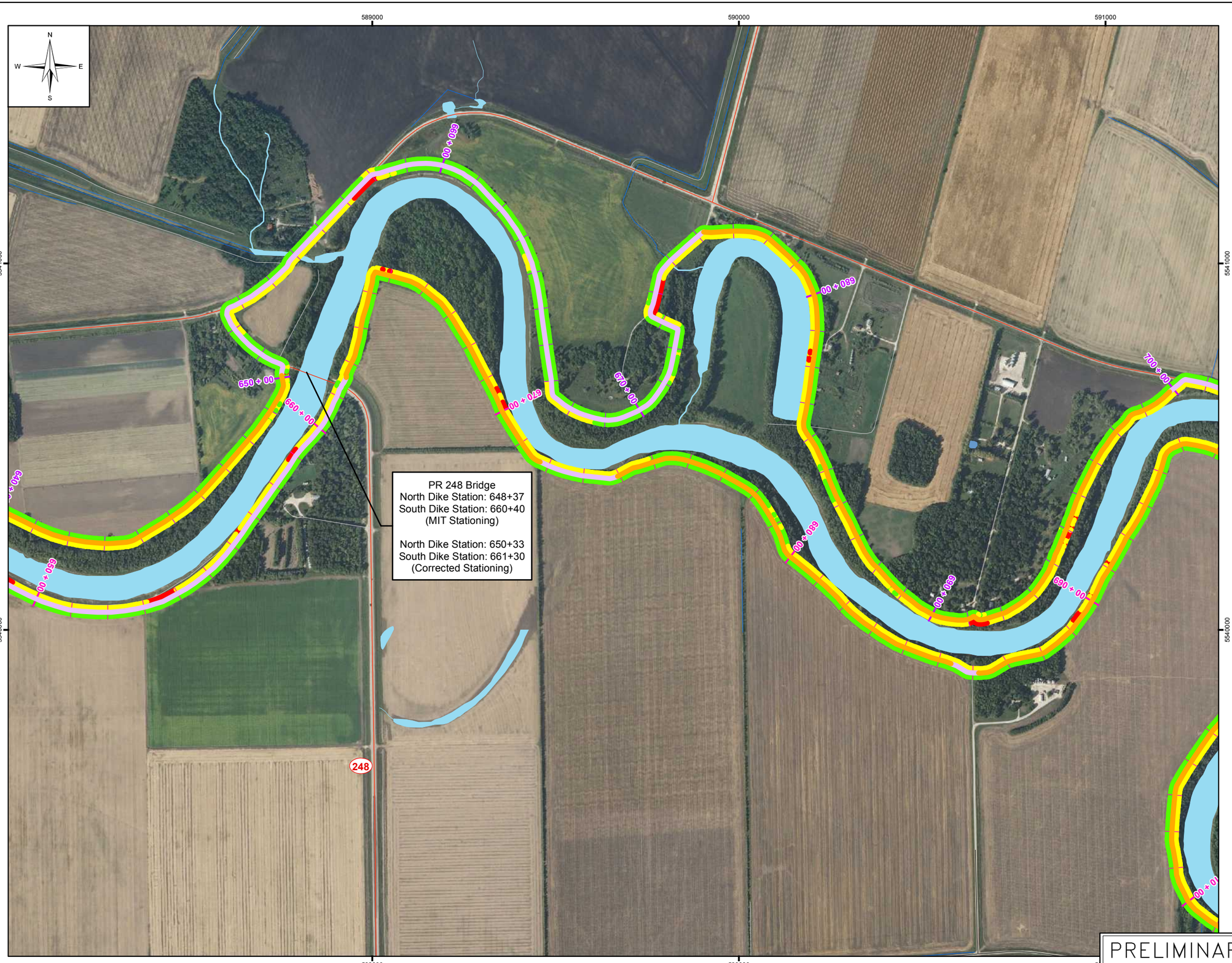
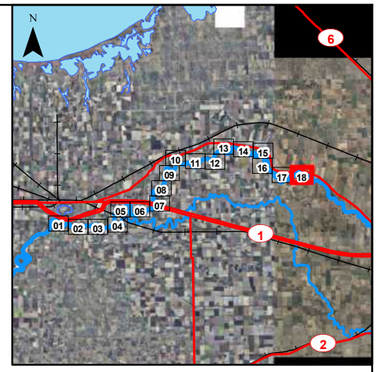




LEGEND:

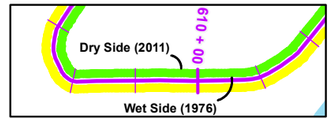
- South Dike CL
- North Dike CL

Land Ownership

- Easement
- Govt Owned
- Private

Freeboard Conditions

- No Freeboard Present
- 0 – 2ft Freeboard
- > 2ft Freeboard



NOTE:
- Dry Side = Freeboard conditions for 2011 Flood Event.
- Wet Side = Freeboard conditions for 1976 Flood Event.

DRAFT

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	14/05/05	ISSUED FOR INFORMATION	MSW

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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

2013 EMERGENT REPAIRS
ASSINIBOINE DIKES

FREEBOARD CONDITIONS FOR 1976 AND 2011 FLOOD EVENTS
SHEET 18 OF 18

MAY 2014 | FIGURE 01 | REV: A

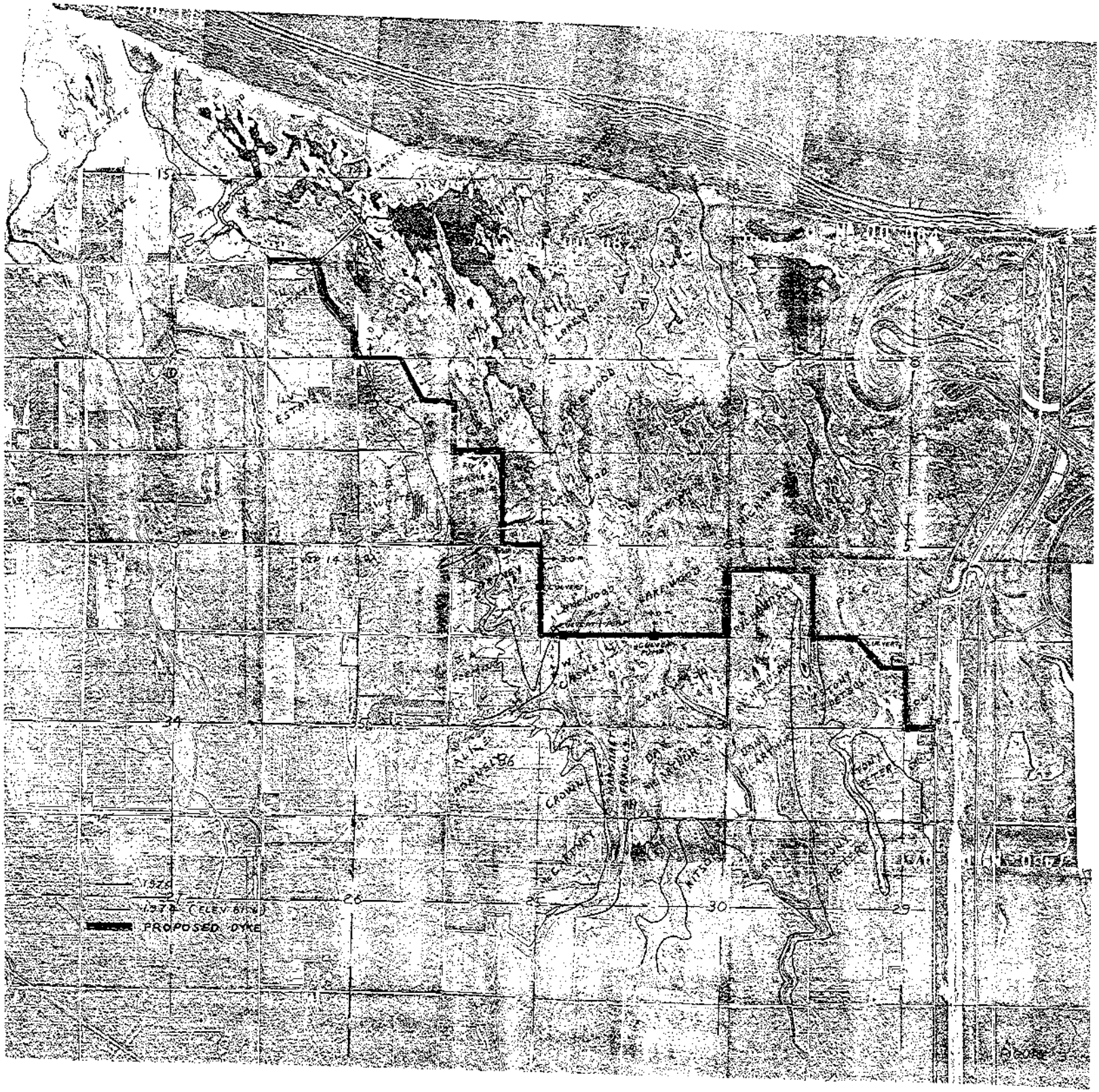
PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

FileName: P:\Projects\2013\13-0300-004\Dwg\GIS\MXD\Task04\RevA\13-0300-014-F01_RevA.mxd
11"x17" PLOT SCALE 1:1

APPENDIX H

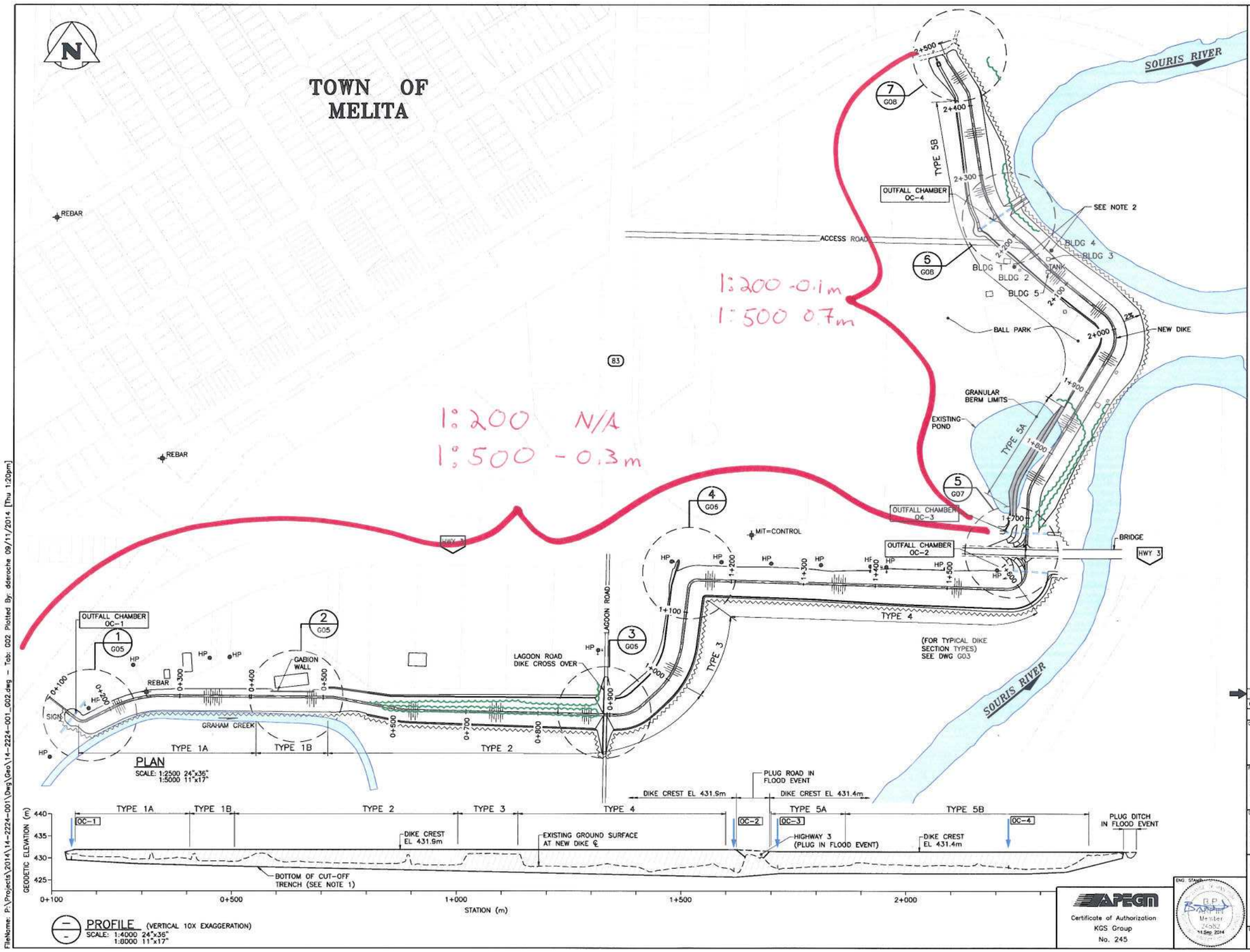
SCHEMATICS OF DIKE ALIGNMENTS

Schematic of Proposed Agricultural Dike Alignment for Elimination of Failsafe



Province of Manitoba, Department of Mines, Resources and Environment Management
Water Resources Division, 1976, Portage Diversion Proposed Modifications to Reach 1A

TOWN OF MELITA



LEGEND:

REBAR	IRON BAR
OC-2	OUTFALL CHAMBER 2
	SILT FENCE
TYPE 4	DESIGNATES DIKE SECTION GEOMETRY (SEE DWG G03)
	OUTLINE OF TREES
F.P.L.	FLOOD PROTECTION LEVEL ELEVATION 431.4m NORTH OF HWY 3 431.9m SOUTH OF HWY 3

NOTES:

1. INVERT OF CUT-OFF TRENCH IS TYPICALLY 1.0 METRE BELOW STRIPPED FOUNDATION SURFACE AND MAYBE ADJUSTED AS APPROVED BY THE CONTRACT ADMINISTRATOR.
2. BUILDINGS 1 AND 2 SHALL REMAIN WITHOUT DAMAGES. BUILDING 3, 4 AND 5 SHALL BE REMOVED OFF SITE BY OTHERS. RELOCATION/RECONFIGURATION OF BALL PARK TO BE DONE BY OTHERS.

X.Xm - Required height increase

50 0 50 100 150 200m
SCALE: 1:2500 METRIC 24"x36"
1:5000 METRIC 11"x17"

NO.	YY/MM/DD	REVISIONS / ISSUE	BY
0	14/05/10	ISSUED FOR TENDER AND CONSTRUCTION	BPA

CLIENT: **THE TOWN OF MELITA**

PROJECT: COMMUNITY FLOOD PROTECTION SYSTEM MELITA, MB

DWG. DESCRIPTION: DIKE WORKS GENERAL ARRANGEMENT PLAN AND PROFILE

DESIGN BY:	RGN	DATE (YY/MM/DD):	14/07/11
DESIGN CHECK:	BPA	DATE:	14/05/10
DRAWN BY:	PwD/ESm	DATE:	14/07/11
DWG CHECK:	JS	DATE:	14/05/10

KGS GROUP CONSULTING ENGINEERS

DWG. NO: 14-2224-001 G02 REV: 0

APEGM Certificate of Authorization KGS Group No. 245

ENG. STAMP: B.P. Banal Member 14 Sep 2014

FileName: P:\Projects\2014\14-2224-001\Dwg\Geo\14-2224-001_002.dwg -- Tab: 002 Plotted By: dderoche 09/11/2014 [Thu 1:20pm]

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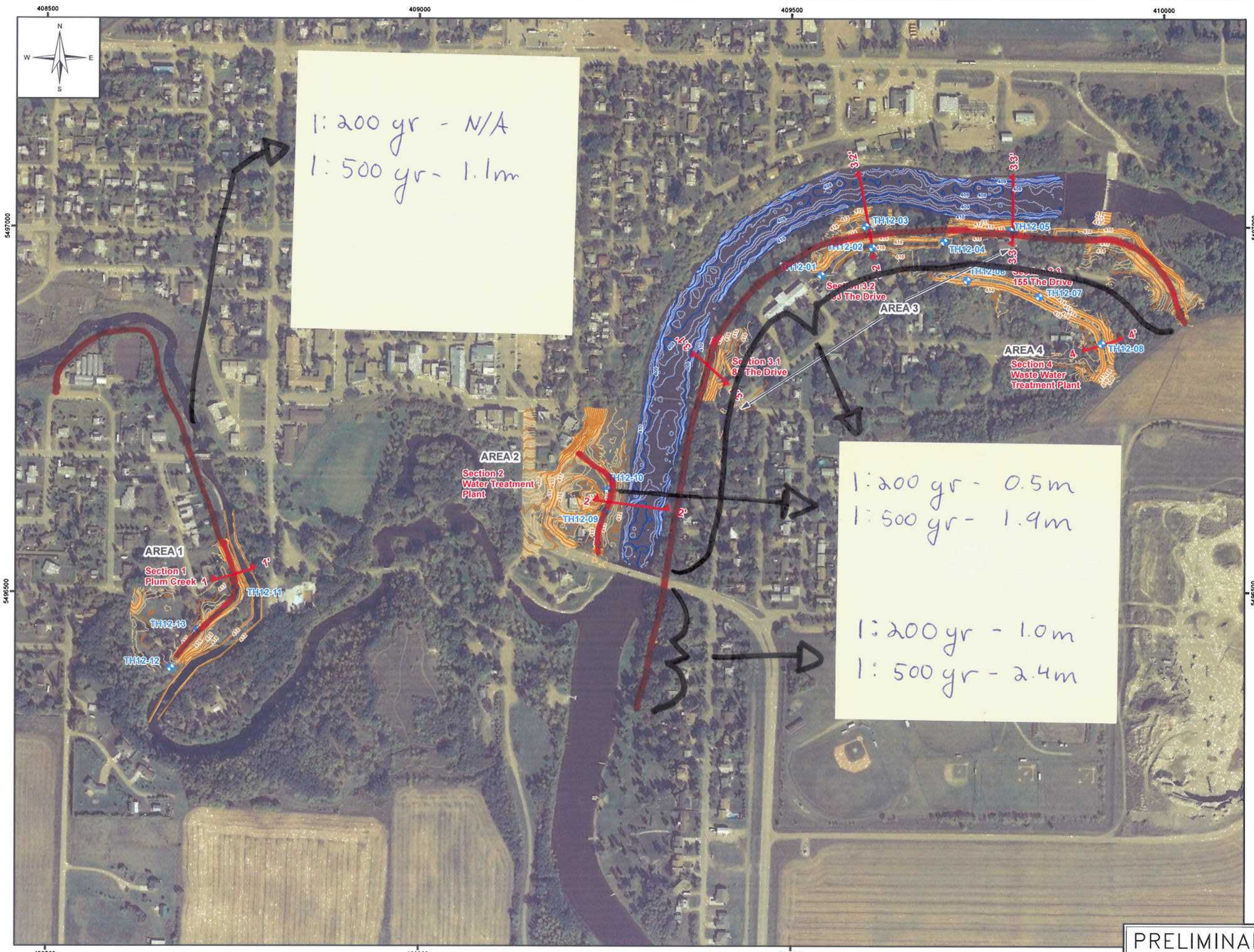

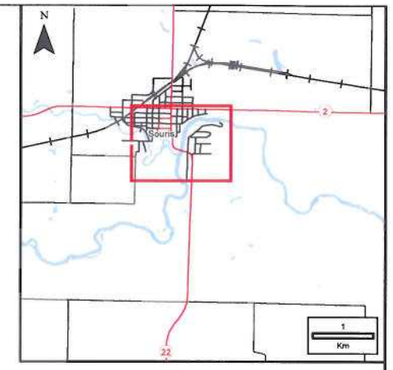

LEGEND:

- KGS GROUP 2012 Testhole
- 1m Index Bathymetry
- 0.25m Bathymetry
- 1m Index Contour
- 0.25m Contour
- Cross Section
- Dike Alignment

x.xm - Required height increase

Notes:
1. Section 1: Plum Creek
2. Section 2: Water Treatment Plant
3. Section 3.1: 85 The Drive
4. Section 3.2: 133 The Drive
5. Section 3.3: 155 The Drive
6. Section 4: Waste Water Treatment Plant
7. All units are metric and in metres unless otherwise specified. Transverse Mercator Projection, NAD 1983, Zone 14. Elevations are in metres above sea level (MSL)
8. Contours based on topographic surveys completed by G.D. Newton & Associates Inc. (2012) and bathymetric survey by KGS Group (2012).

50 0 50 100 150 200
Metres

SCALE: 1:5,000 METRIC 11"x17"

NO.	YY/MM/DD	DESCRIPTION	BY
A	13/10/23	ISSUED WITH FINAL REPORT	AMH

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

G.D. NEWTON & ASSOCIATES INC.

GEOTECHNICAL ASSESSMENT OF THE EXISTING SOURIS DIKE

SITE PLAN SHOWING TESTHOLE LOCATIONS AND ANALYZED SECTIONS

OCTOBER 2013 | DRAWING 01 | REV. 0

PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

FileName: P:\Projects\2012\12-1436-001\Dwg\GIS\MXD\Town_of_Souris_Temp_Dikes_Report\Rev0\12-1436-001_F01_Rev0.mxd
11"x17" PLOT SCALE 1:1

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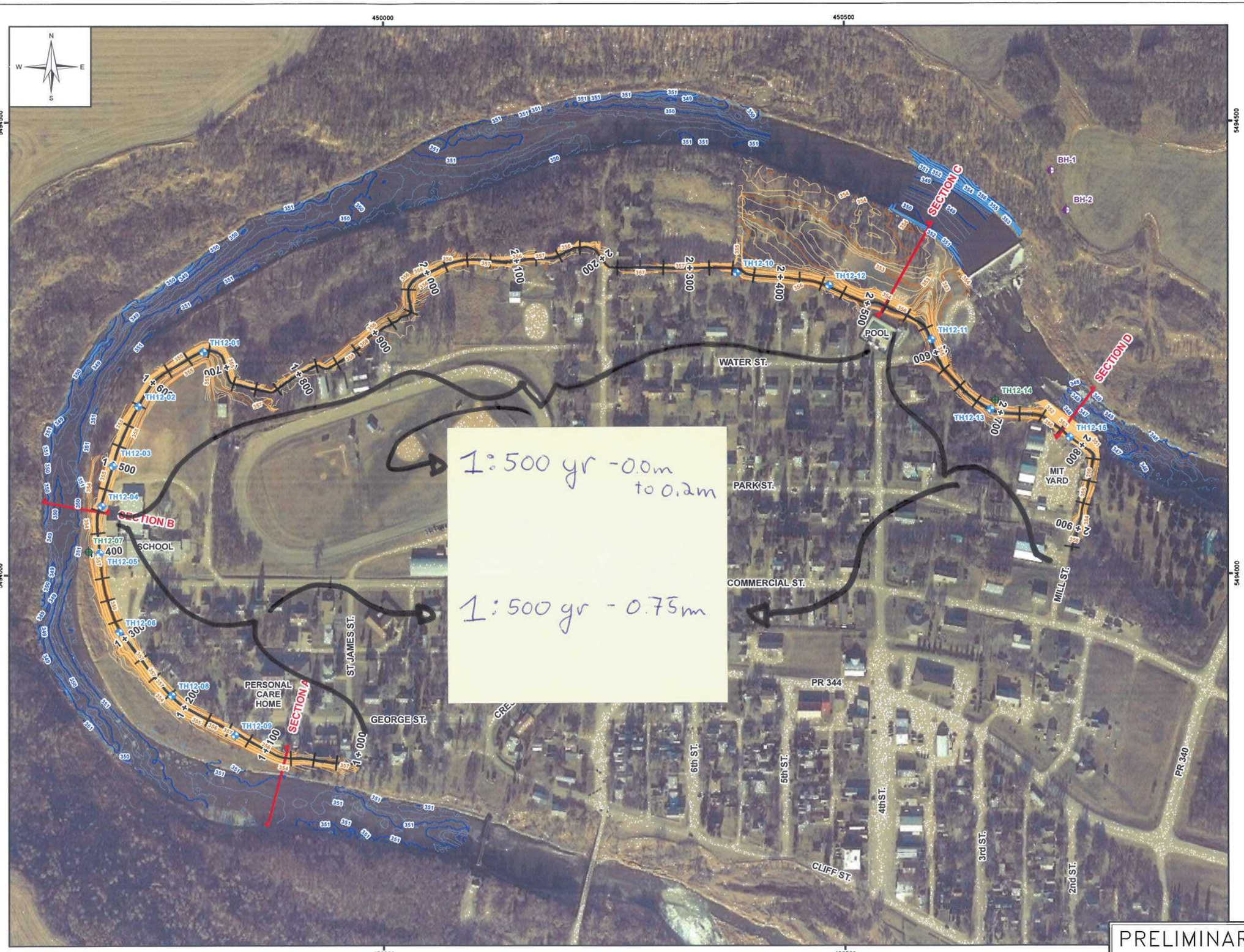
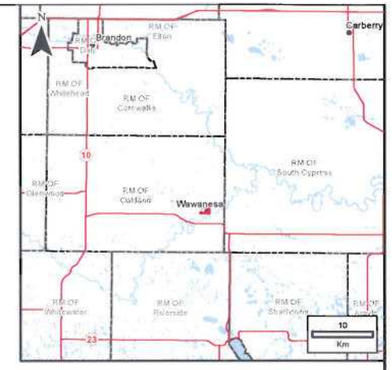


1:500 yr -0.0m to 0.2m

1:500 yr - 0.75m



LEGEND:

- KGS Group 2012 Testhole
- KGS Group 2012 Standpipe
- MIT 2011 Testhole
- Dike Alignment
- Cross Section
- 1m Index Contour
- 0.25m Contour
- 1m Index Bathymetry
- 0.25m Bathymetry

X.Xm - Required height increase

Notes:
1. Contours generated from G.D. Newton & Associates Inc. (2011), and KGS Group (2012) topographic, and bathymetric surveys.
2. All units are metric and in metres unless otherwise specified. Transverse Mercator Projection, NAD 1983, Zone 14. Elevations are in metres above sea level (MSL).

DRAFT

50 0 50 100 150
Metres

SCALE: 1:4,000 METRIC 11"x17"

NO.	YY/MM/DD	DESCRIPTION	BY
A	12/12/14	ISSUED WITH DRAFT REPORT	AMH

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KGS GROUP CONSULTING ENGINEERS

G.D. NEWTON & ASSOCIATES INC.

GEOTECHNICAL ASSESSMENT OF THE EXISTING WAWANESA DIKE

SITE PLAN
EXISTING CONDITIONS
AND TESTHOLE LOCATIONS

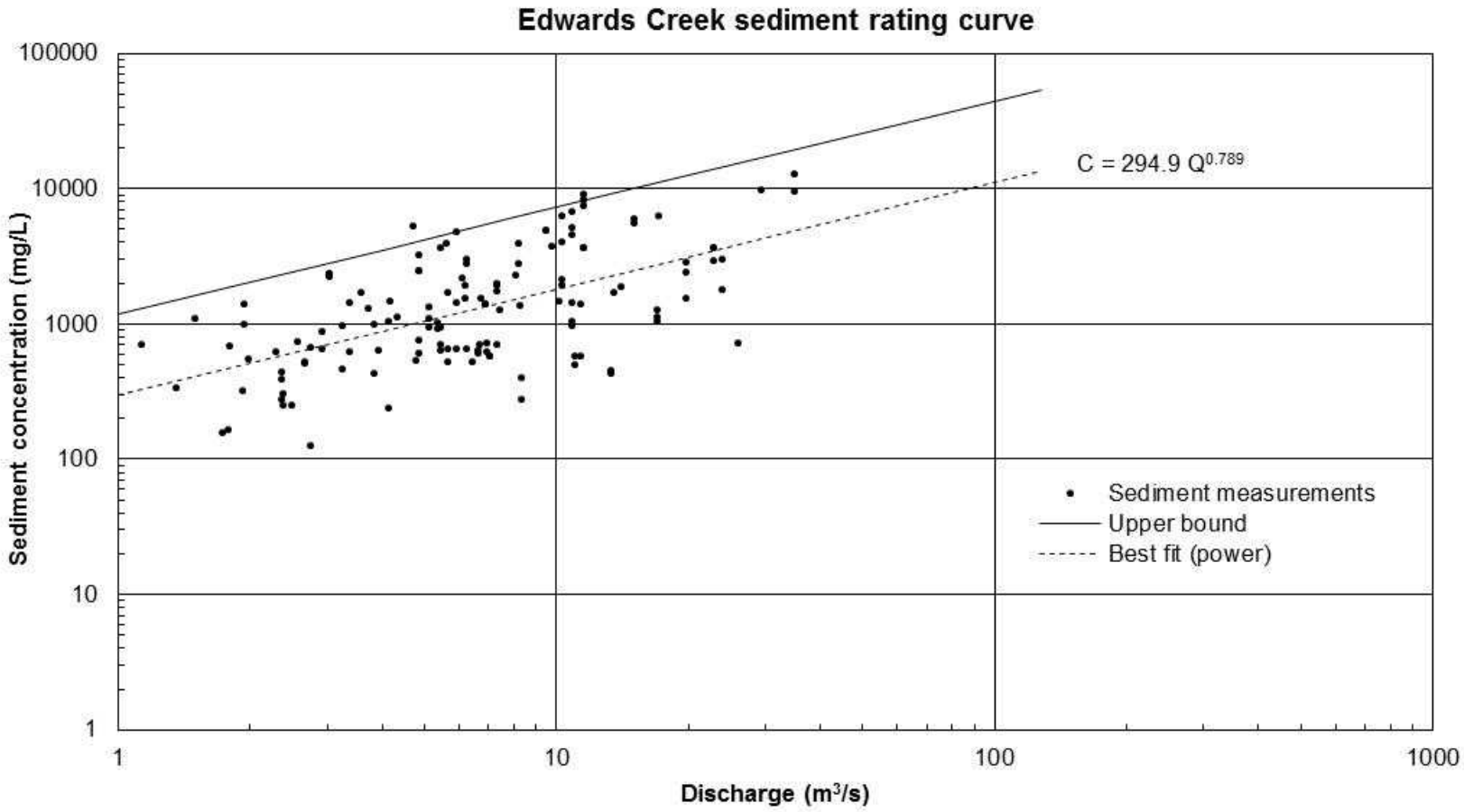
DECEMBER 2012 | DRAWING 01 | REV A

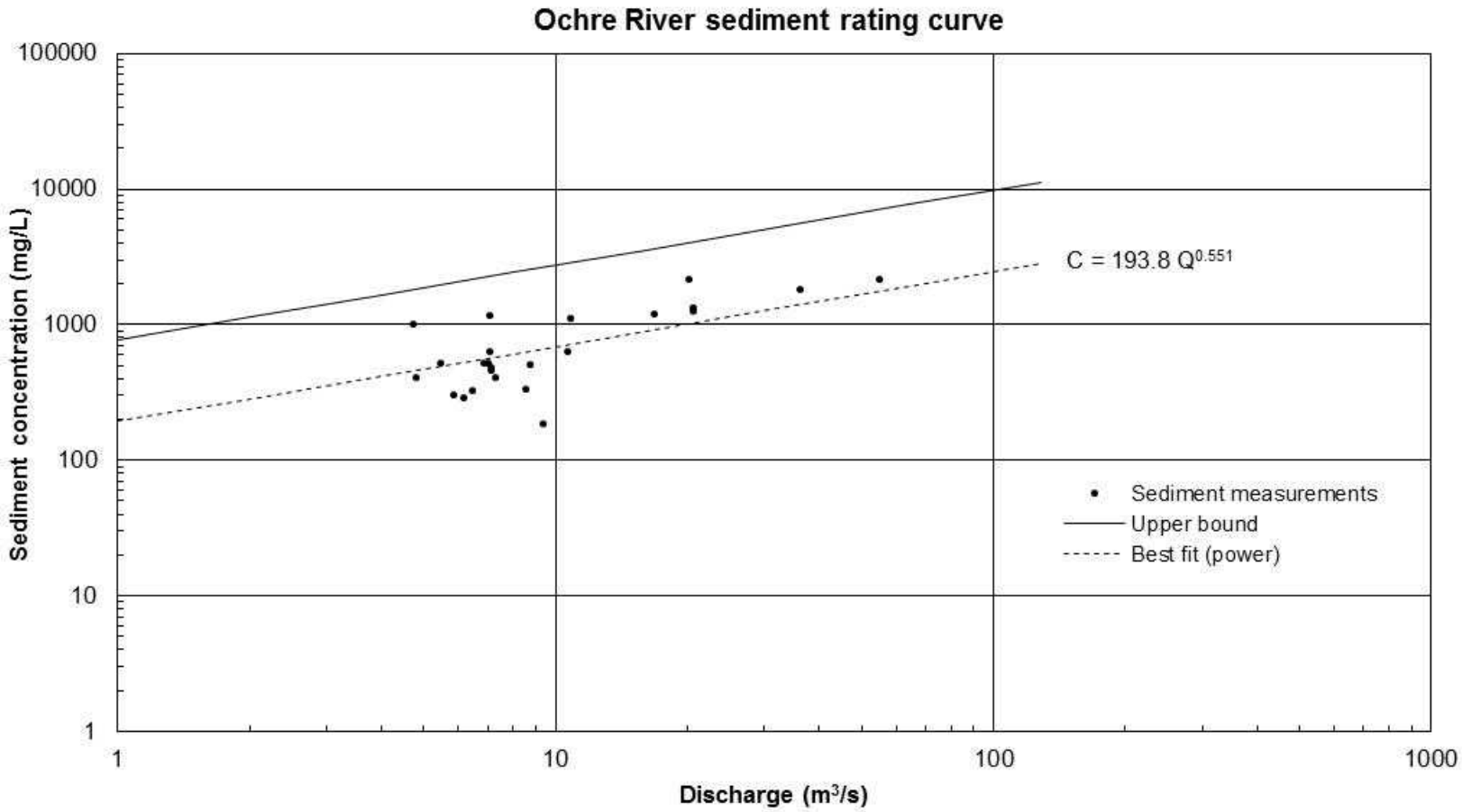
PRELIMINARY
NOT TO BE USED FOR CONSTRUCTION

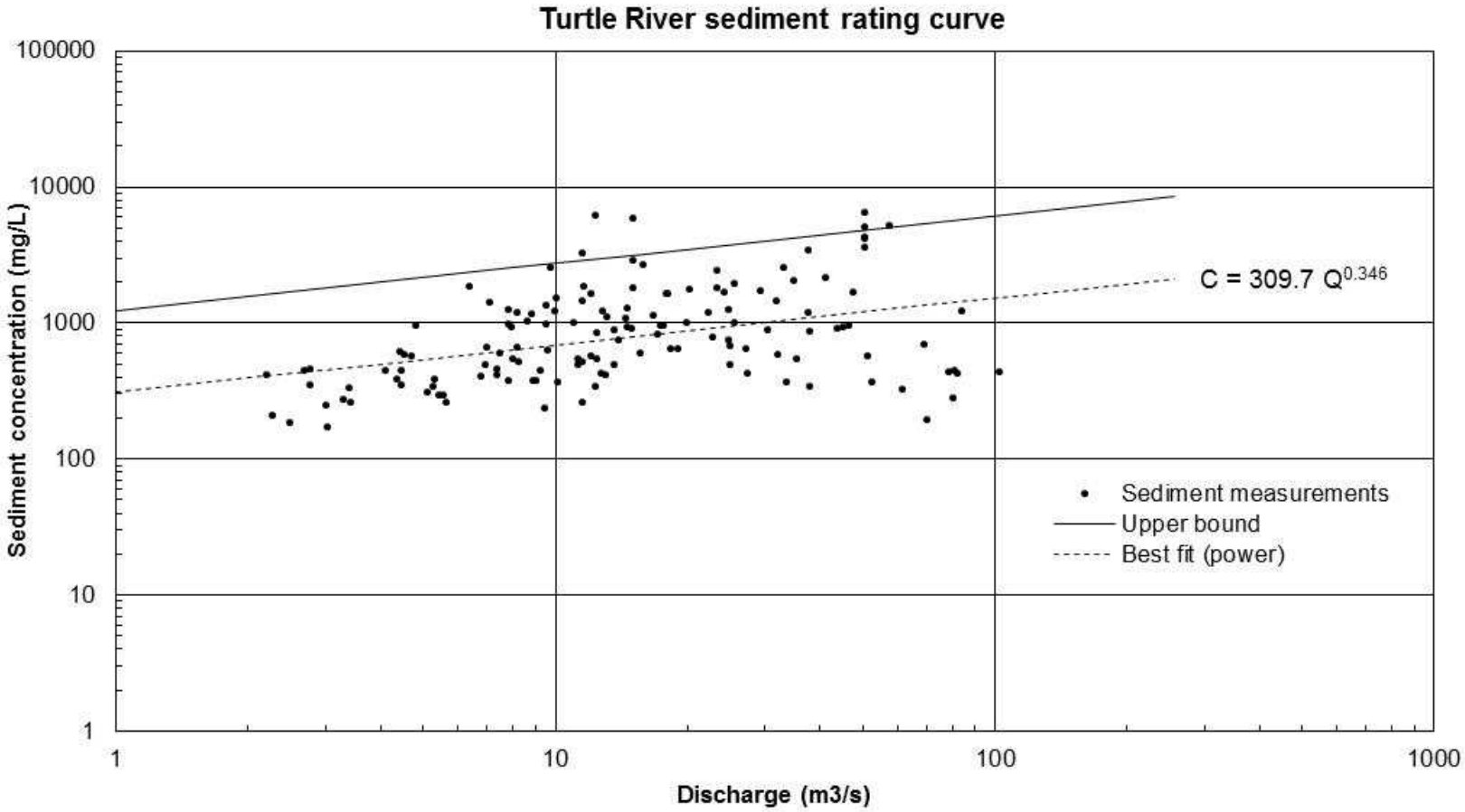
FileName: P:\Projects\2012\12-1436-002\Dwg\GIS\MXD\RevA\12-1436-002_F01.mxd
11"x17" PLOT SCALE 1:1

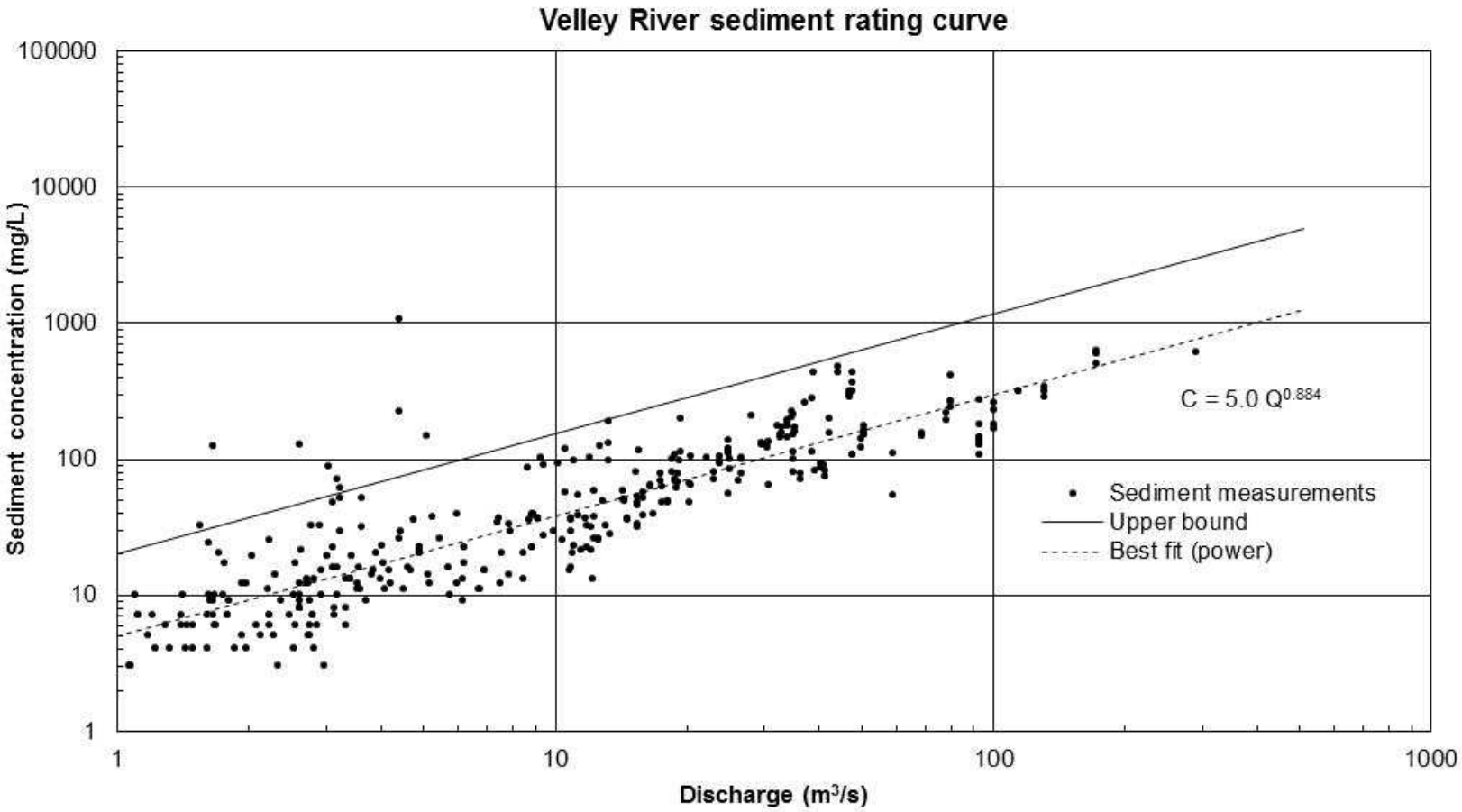
APPENDIX I

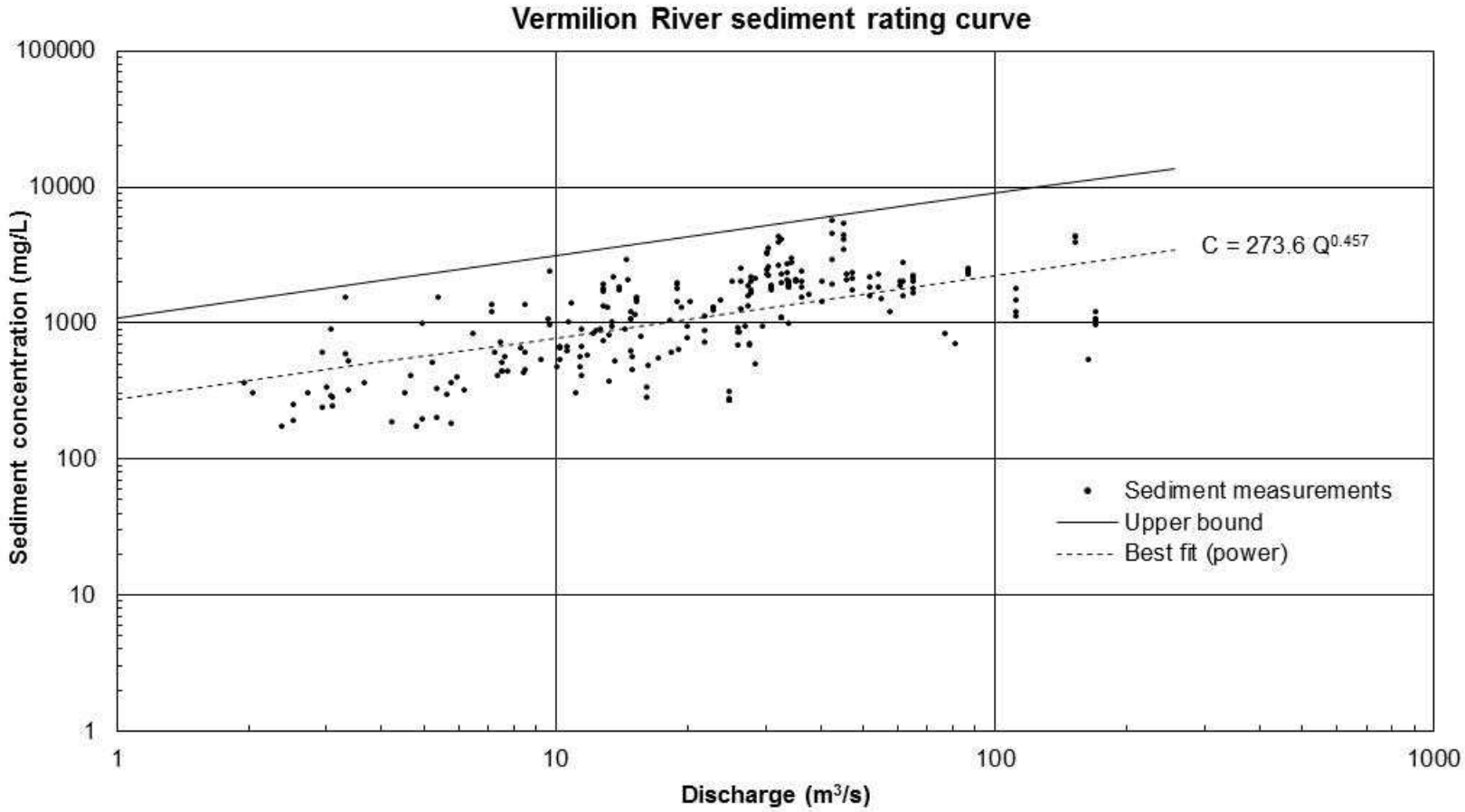
SUSPENDED SEDIMENT RATING CURVES FOR DAUPHIN LAKE TRIBUTARIES

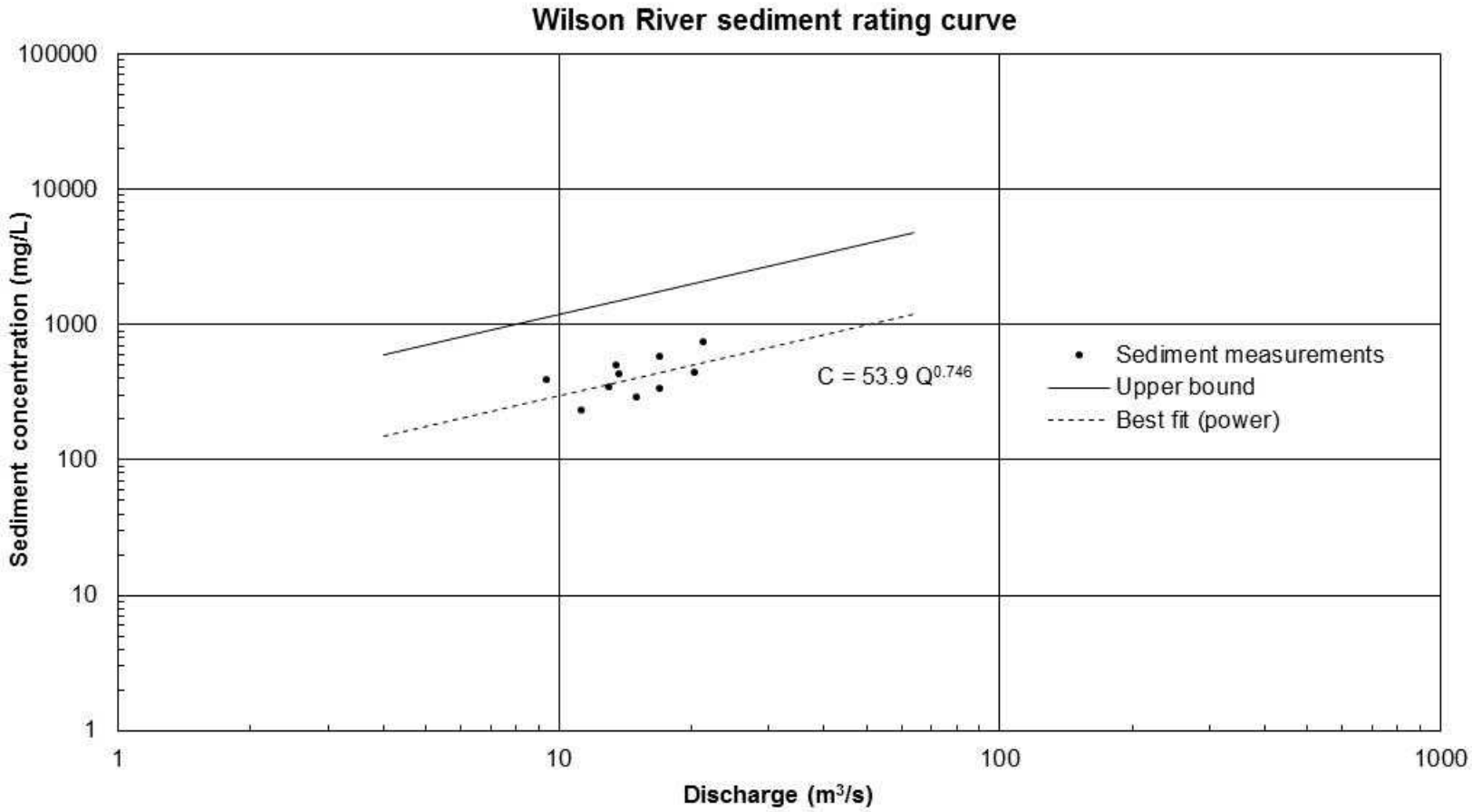

Edwards Creek sediment rating curve
100000
10000
1000
100
10
1
Sediment concentration (mg/L)
1
10
100
1000
Discharge (m3/s)
C = 294.9 Q0.789
Sediment measurements
Upper bound
Best fit (power)


Ochre River sediment rating curve
Sediment concentration (mg/L)
100000
10000
1000
100
10
1
1
10
100
1000
Discharge (m3/s)
C = 193.8 Q0.551
Sediment measurements
Upper bound
Best fit (power)


Turtle River sediment rating curve
Sediment concentration (mg/L)
100000
10000
1000
100
10
1
1
10
100
1000
Discharge (m3/s)
C = 309.7 Q^0.346
Sediment measurements
Upper bound
Best fit (power)


Velley River sediment rating curve
Sediment concentration (mg/L)
100000
10000
1000
100
10
1
1
10
100
1000
Discharge (m3/s)
C = 5.0 Q0.884
Sediment measurements
Upper bound
Best fit (power)


Vermilion River sediment rating curve
Sediment concentration (mg/L)
100000
10000
1000
100
10
1
1
10
100
1000
Discharge (m3/s)
C = 273.6 Q0.457
Sediment measurements
Upper bound
Best fit (power)


Wilson River sediment rating curve
100000
10000
1000
100
10
1
Sediment concentration (mg/L)
1
10
100
1000
Discharge (m3/s)
C = 53.9 Q0.746
Sediment measurements
Upper bound
Best fit (power)

APPENDIX J

FLOODED AREA MAPS

LOWER ASSINIBOINE RIVER IN WINNIPEG

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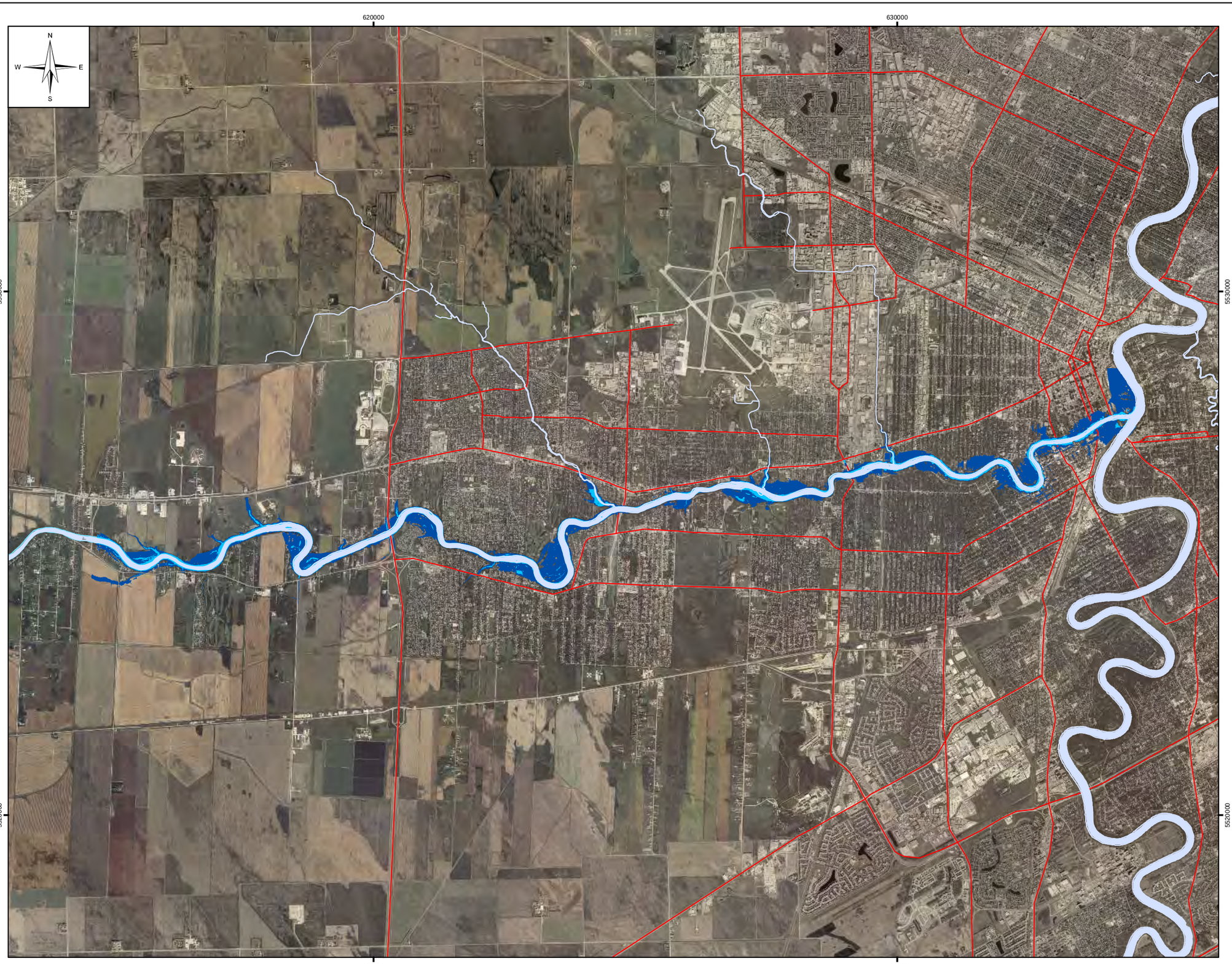

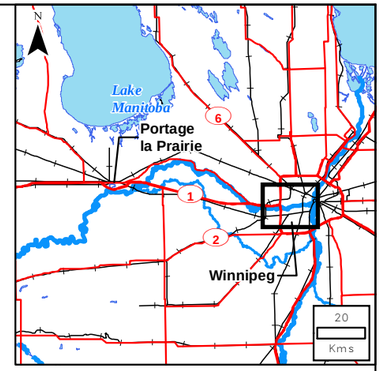

LEGEND:

- Inundated Area
- Areas At-risk of Basement Flooding
- Roads
- Rivers
- Rivers / Lakes

DRAFT

1 0 1 2 3

Kilometres

SCALE: 1:70,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	15/07/10	ISSUED WITH DRAFT REPORT	PAL

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

FLOODED AREA MAP OF LOWER ASSINIBOINE RIVER IN WINNIPEG 20,000 CFS

JULY 2015	DWG J-1	REV: A

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11"x17" PLOT SCALE 1:1

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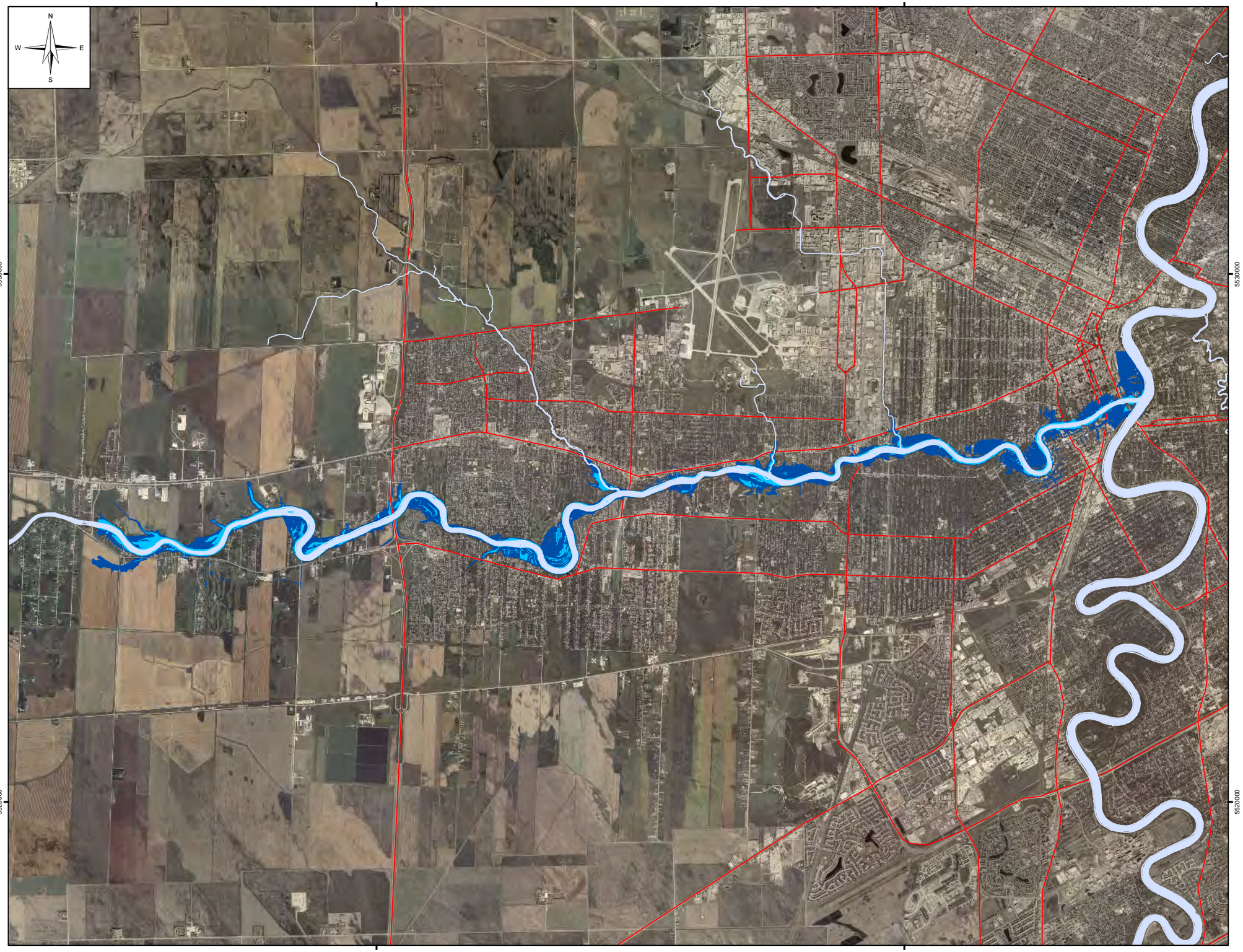

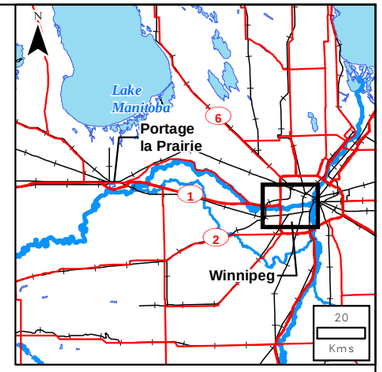

LEGEND:

- Inundated Area
- Areas At-risk of Basement Flooding
- Roads
- Rivers
- Rivers / Lakes

DRAFT

1 0 1 2 3

Kilometres

SCALE: 1:70,000 METRIC 11"x17"

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Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
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FLOODED AREA MAP OF LOWER ASSINIBOINE RIVER IN WINNIPEG 25,000 CFS

JULY 2015 | DWG J-2 | REV: A

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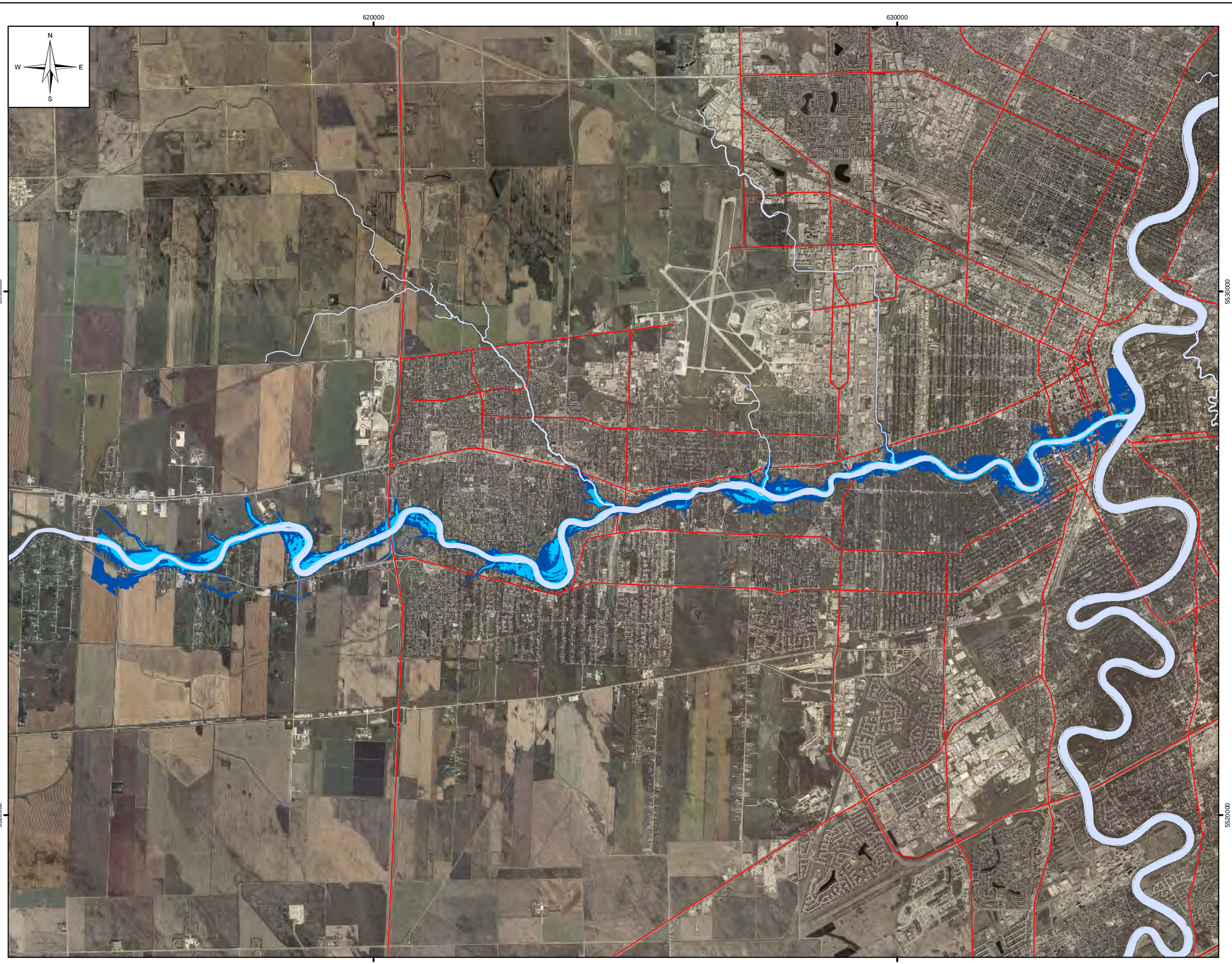

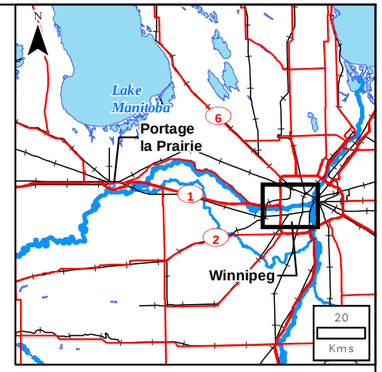

LEGEND:

- Inundated Area
- Areas At-risk of Basement Flooding
- Roads
- Rivers
- Rivers / Lakes

DRAFT

1 0 1 2 3

Kilometres

SCALE: 1:70,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

FLOODED AREA MAP OF LOWER ASSINIBOINE RIVER IN WINNIPEG 30,000 CFS

JULY 2015	DWG J-3	REV: A

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\RevA\Final Report\12-0300-011_DJ-3_RevA.mxd
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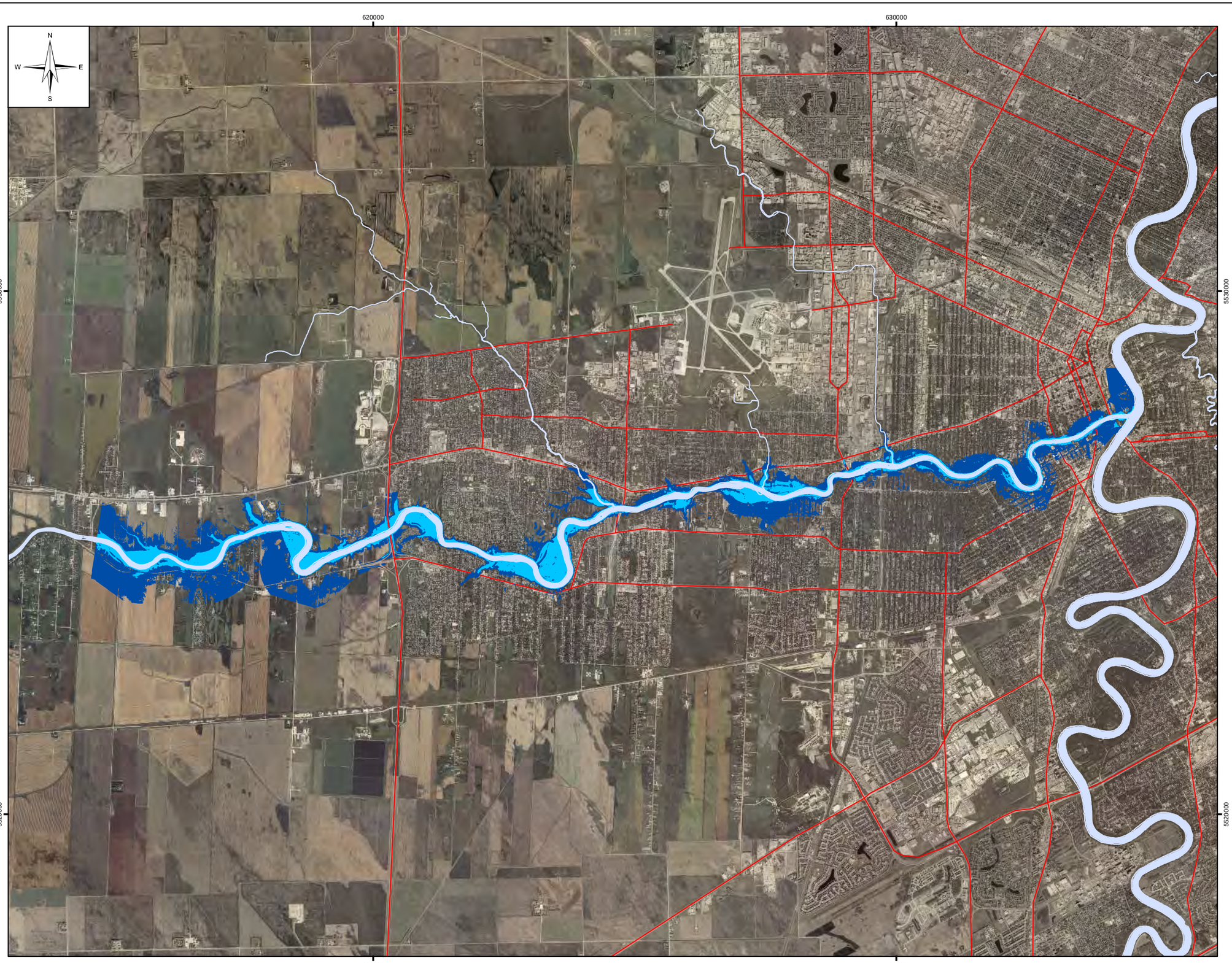

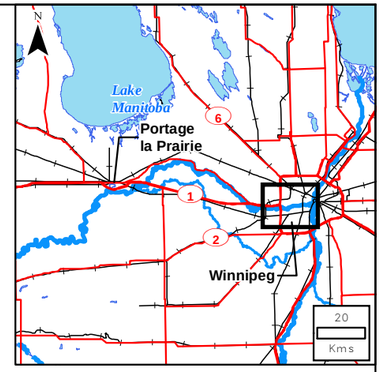

LEGEND:

- Inundated Area
- Areas At-risk of Basement Flooding
- Roads
- Rivers
- Rivers / Lakes

DRAFT

1 0 1 2 3

Kilometres

SCALE: 1:70,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
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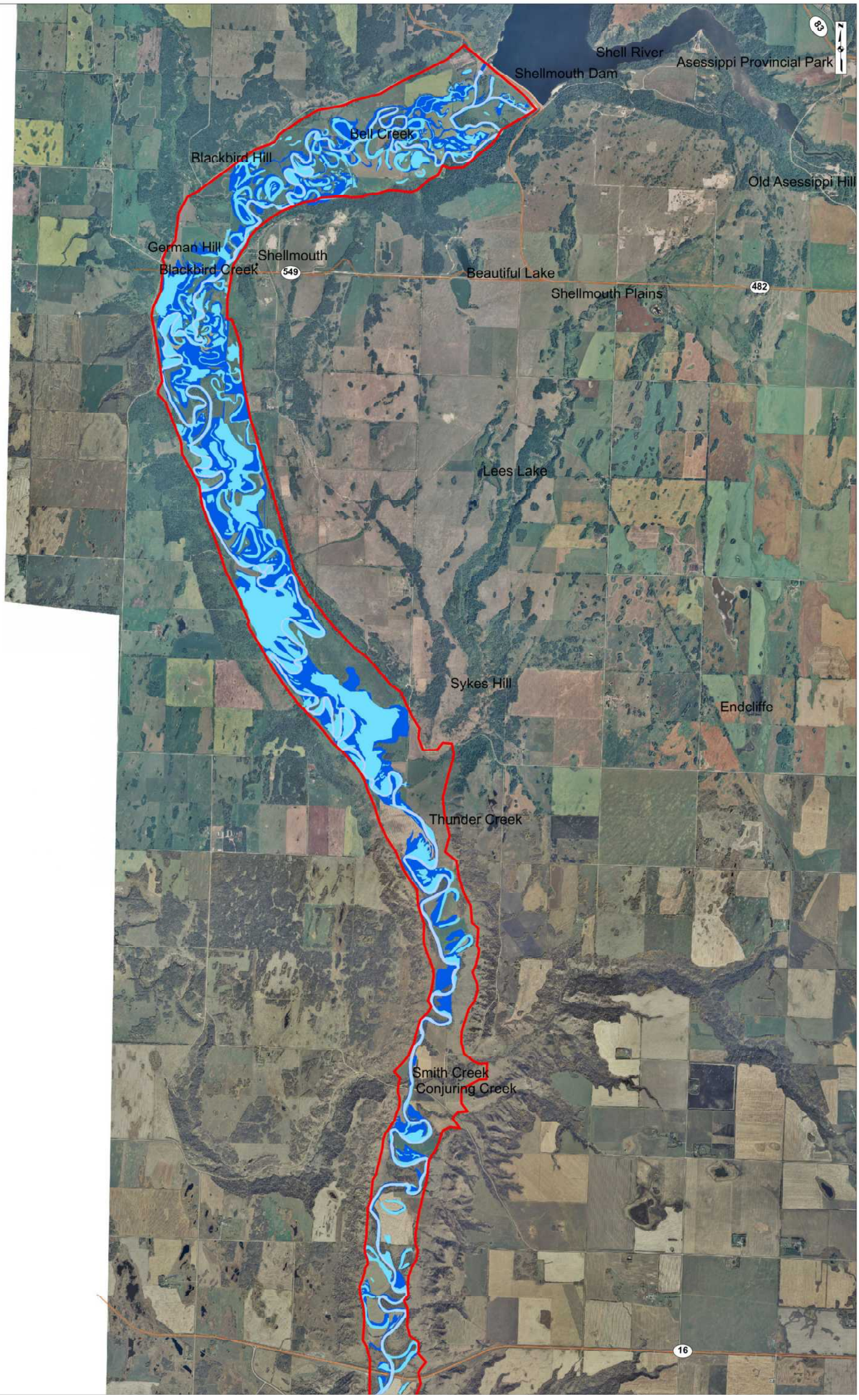
ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

FLOODED AREA MAP OF LOWER ASSINIBOINE RIVER IN WINNIPEG 40,000 CFS

JULY 2015 | DWG J-4 | REV: A

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\RevA\Final Report\12-0300-011_DJ-4_RevA.mxd
11"x17" PLOT SCALE 1:1

UPPER ASSINIBOINE RIVER FROM SHELLMOUTH DAM TO ST. LAZARE



Assiniboine Floodplain
Assiniboine River
Flooded Area July 19, 2012
Flooded Area July 6, 2012

Sources:
Imagery: Geo Manitoba, 2007
Elevation: 2007 Assiniboine LiDAR
Township: Manitoba Land Initiative
Water: Designated Drains, MDB Water
Map Created: May 23, 2014

PRELIMINARY
FOR DISCUSSION PURPOSES ONLY

0 260 520 1,040 1,560 2,080 Meters

Projection: NAD 83 UTM Zone 14

Assiniboine River
PTH 16 to PR478
2012 July 6 Imagery
2012 July 19 Imagery
Approximate Floodplain

Manitoba
Infrastructure and Transportation
Hydrologic Forecasting and Water Management
Box 14 - 200 Saulteaux Cres.
Winnipeg, Manitoba, R3J 3W3
Tel: (204) 945-2161 Fax: (204) 945-7419

Conjuring Dam

16

Harrowby

Russell

Dancing Hill

Millwood

579

Crescent Ridge Hill

Johnson

Hamiltons Lake

478

Assiniboine Floodplain
Assiniboine River
Flooded Area July 19, 2012
Flooded Area July 6, 2012

Sources:
Imagery: Geo Manitoba, 2007
Elevation: 2007 Assiniboine LiDAR
Township: Manitoba Land Initiative
Water: Designated Lakes, 1:50k Water
Map Created: May 23, 2014

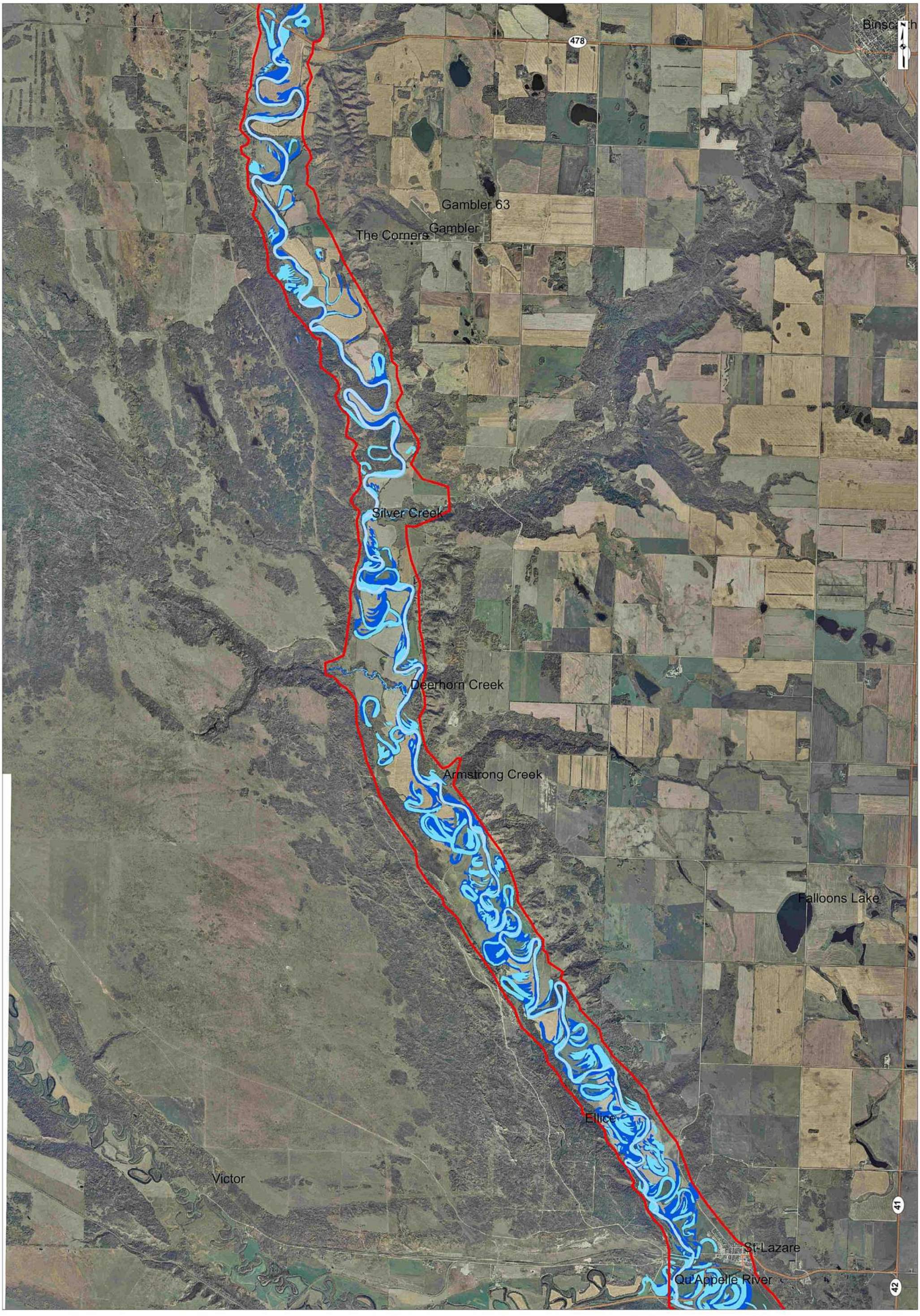
PRELIMINARY
FOR DISCUSSION PURPOSES ONLY

0 230 470 940 1,410 1,880 Meters

Projection: NAD 83 UTM Zone 14

Assiniboine River
PTH 16 to PR478
2012 July 6 Imagery
2012 July 19 Imagery
Approximate Floodplain

Manitoba
Infrastructure and Transportation
Hydrologic Forecasting and Water Management
Box 14 - 200 Saulteaux Cres.
Winnipeg, Manitoba, R3J 3W3
Tel: (204) 945-2161 Fax: (204) 945-7419



Assiniboine Floodplain
Assiniboine River
Flooded Area July 19, 2012
Flooded Area July 0, 2012

Source:
Imagery: Geo Manitoba, 2007
Elevation: 2007 Assiniboine LIDAR
Township: Manitoba Land Initiative
Water: Designated Drains, 500k Water
Map Created: May 23, 2014

PRELIMINARY
FOR DISCUSSION PURPOSES ONLY

0 235 470 940 1,410 1,880 Meters

Projection: NAD 83 UTM Zone 14

Assiniboine River
PR478 to St. Lazare
2012 July 6 Imagery
2012 July 19 Imagery
Approximate Floodplain

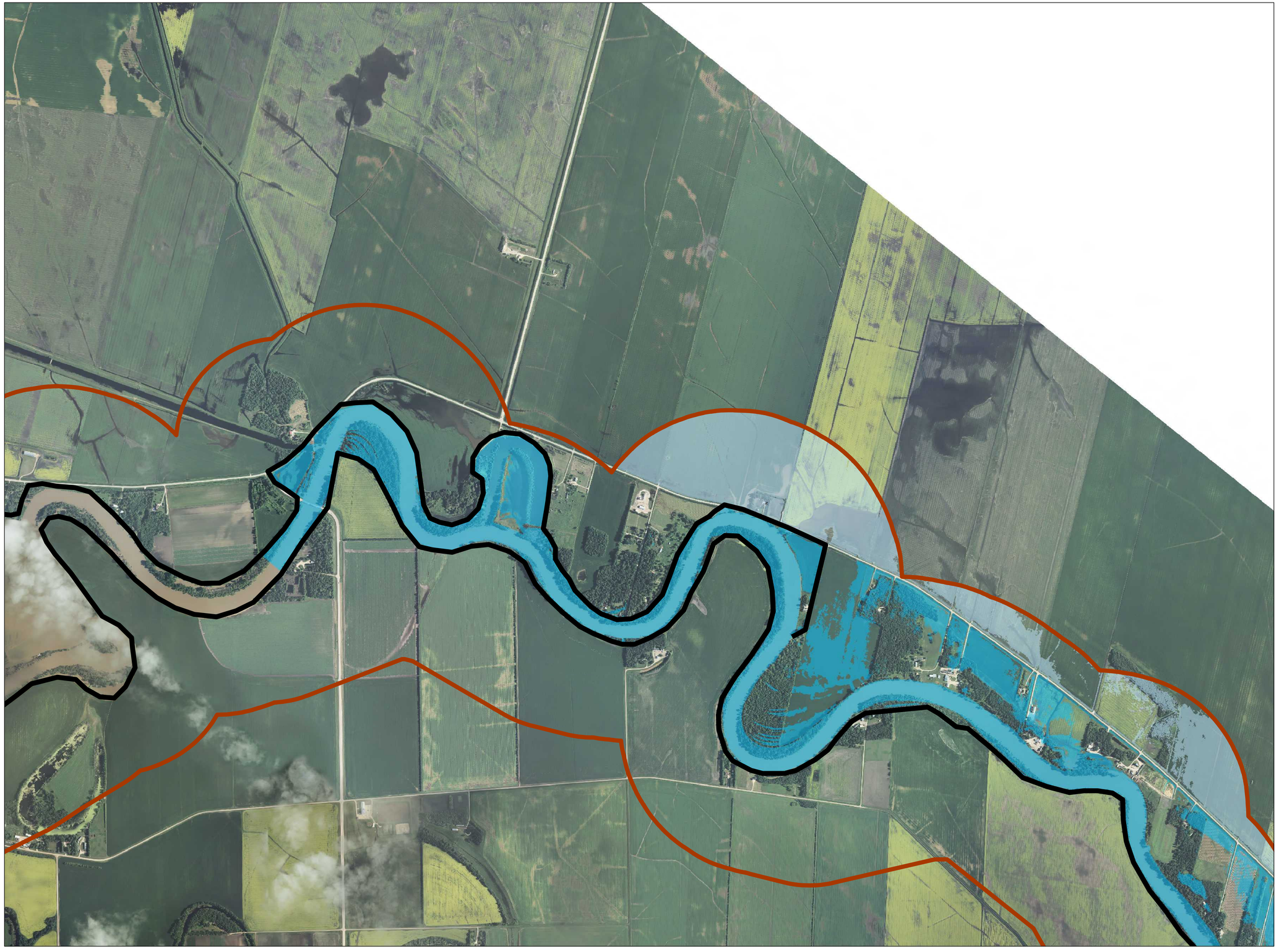
Manitoba
Infrastructure and Transportation
Hydrologic Forecasting and Water Management
Box 14 - 200 Saulteaux Cres.
Winnipeg, Manitoba, R3J 3W3
Tel: (204) 945-2181 Fax: (204) 945-7419

LOWER ASSINIBOINE RIVER FROM BAIE ST. PAUL TO HEADINGLEY

Baie St Paul to Headingley
Flood Extent:
KGS 18,000 cfs

Map 1 of 5

Flood level based on
the 2015 KGS Water Surface Profile



LiDAR Extent
Assiniboine River Dike

2015 KGS

Area affected by 18000 cfs
Uncertainty in area affected by 18000 cfs

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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Baie St Paul to Headingley
Flood Extent:
KGS 18,000 cfs

Map 2 of 5

Flood level based on
the 2015 KGS Water Surface Profile

LiDAR Extent
Assiniboine River Dike
2015 KGS
Area affected by 18000 cfs
Uncertainty in area affected by 18000 cfs

0 0.5 1 2 3 Kilometers
0 0.5 1 2 3 Miles

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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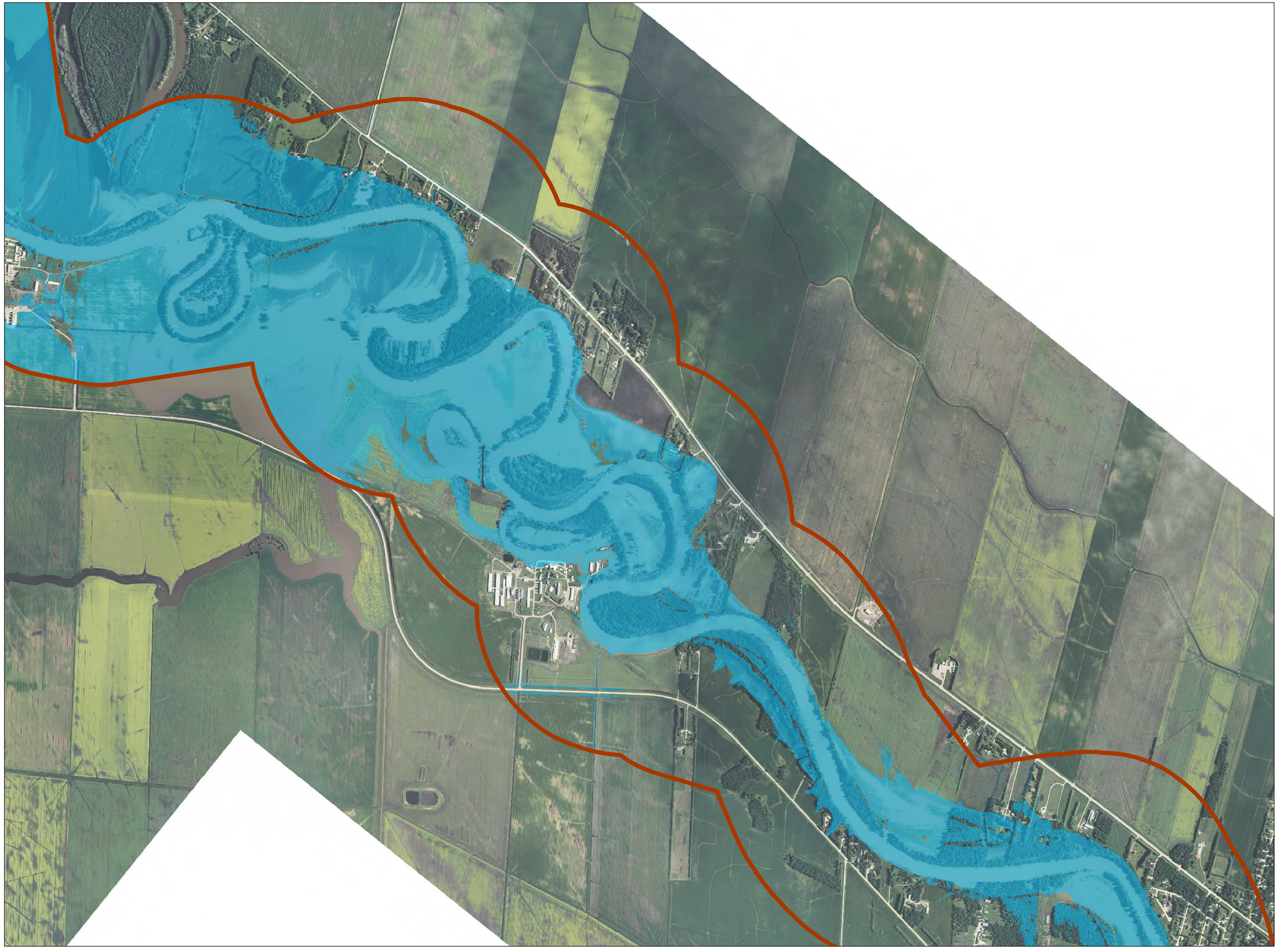
Baie St Paul to Headingley
Flood Extent:
KGS 18,000 cfs

Map 3 of 5

Flood level based on
the 2015 KGS Water Surface Profile



LiDAR Extent
Assiniboine River Dike
2015 KGS
Area affected by 18000 cfs
Uncertainty in area affected by 18000 cfs

0 0.5 1 2 3 Kilometers
0 0.5 1 2 3 Miles

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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Baie St Paul to Headingley
Flood Extent:
KGS 18,000 cfs

Map 4 of 5

Flood level based on
the 2015 KGS Water Surface Profile

LiDAR Extent
Assiniboine River Dike

2015 KGS

Area affected by 18000 cfs
Uncertainty in area affected by 18000 cfs

0 0.5 1 2 3 Kilometers
0 0.5 1 2 3 Miles

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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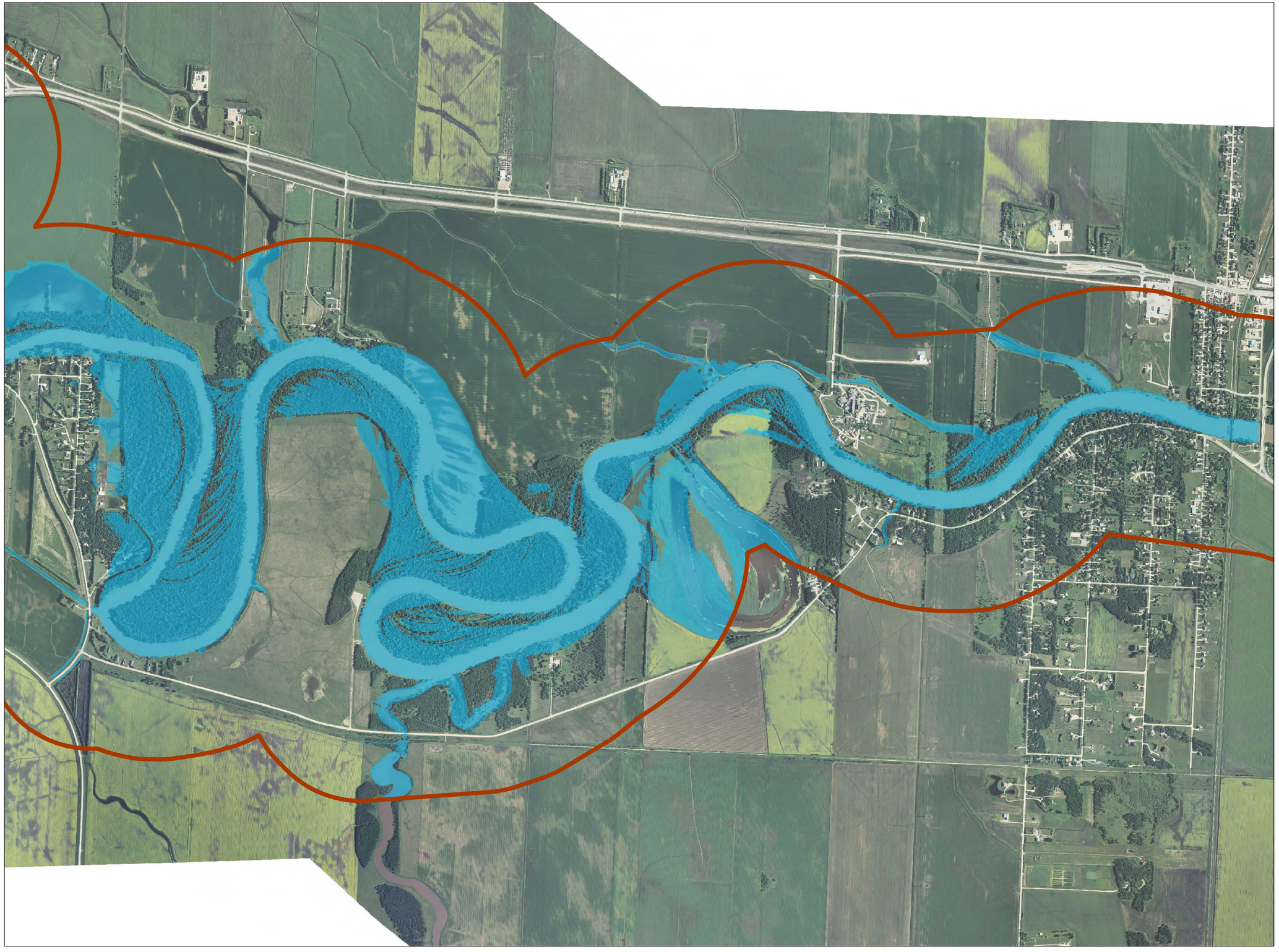
Baie St Paul to Headingley
Flood Extent:
KGS 18,000 cfs

Map 5 of 5

Flood level based on
the 2015 KGS Water Surface Profile



LiDAR Extent
Assiniboine River Dike
2015 KGS
Area affected by 18000 cfs
Uncertainty in area affected by 18000 cfs

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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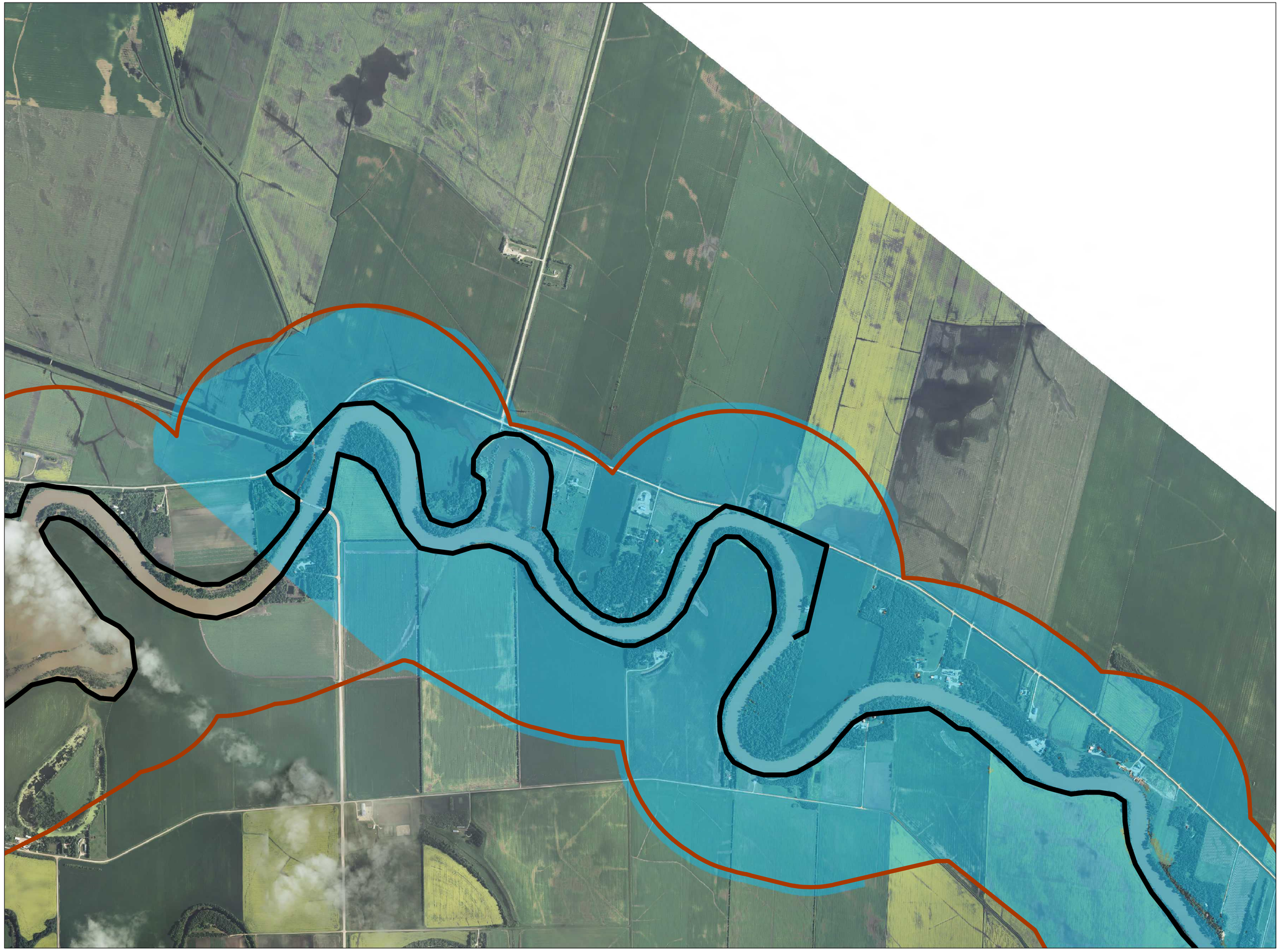
Baie St Paul to Headingley
Flood Extent:
KGS 23,100 cfs

Map 1 of 5

Flood level based on
the 2015 KGS Water Surface Profile



For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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Baie St Paul to Headingley
Flood Extent:
KGS 23,100 cfs

Map 2 of 5

Flood level based on
the 2015 KGS Water Surface Profile

- LiDAR Extent
- Assiniboine River Dike

2015 KGS

- Area affected by 23100 cfs
- Uncertainty in area affected by 23100 cfs

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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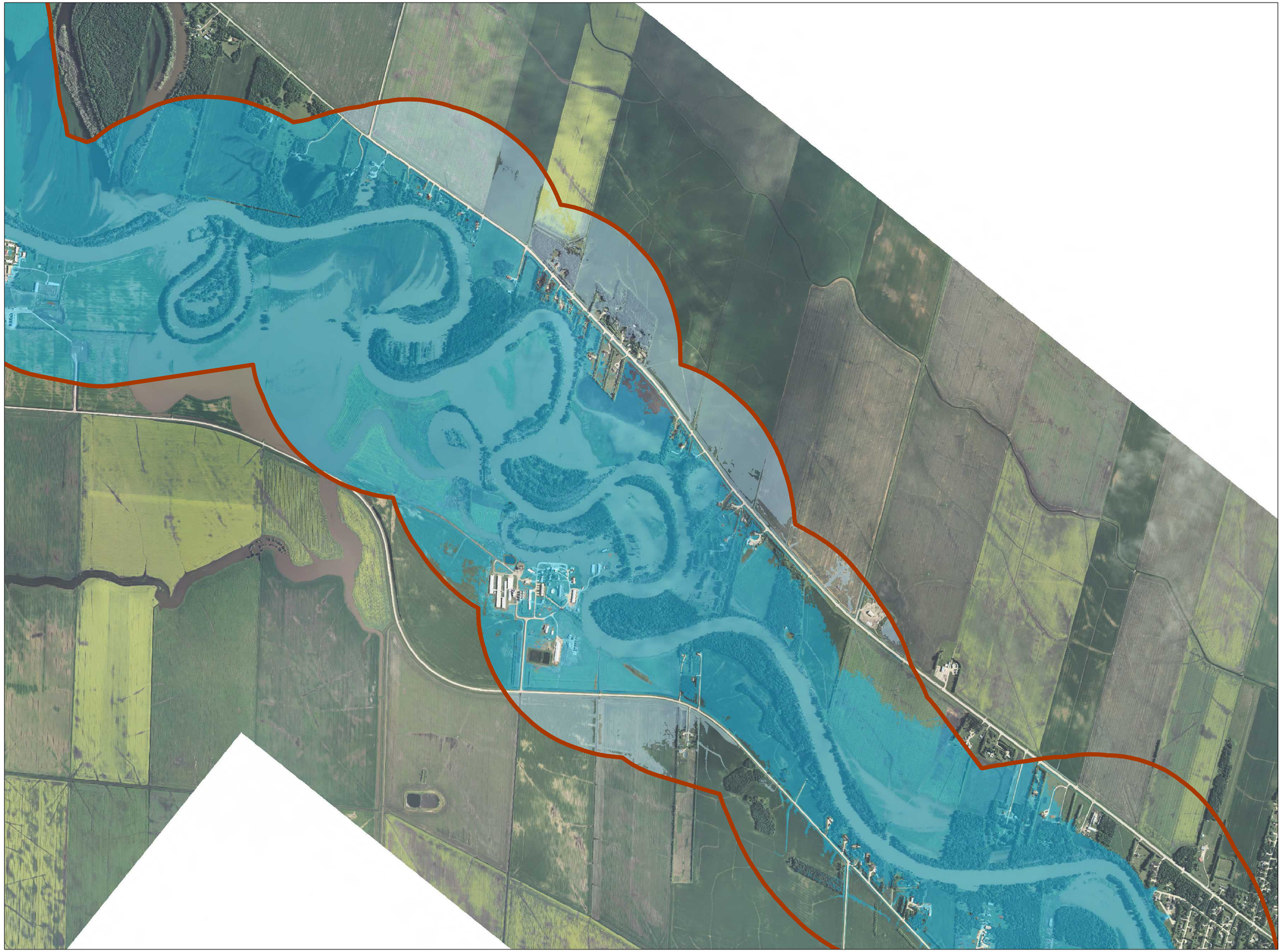
Baie St Paul to Headingley
Flood Extent:
KGS 23,100 cfs

Map 3 of 5

Flood level based on
the 2015 KGS Water Surface Profile



LiDAR Extent
Assiniboine River Dike
2015 KGS
Area affected by 23100 cfs
Uncertainty in area affected by 23100 cfs

0 0.5 1 2 3 Kilometers
0 0.5 1 2 3 Miles

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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Baie St Paul to Headingley
Flood Extent:
KGS 23,100 cfs

Map 4 of 5

Flood level based on
the 2015 KGS Water Surface Profile

- LiDAR Extent
- Assiniboine River Dike

2015 KGS

- Area affected by 23100 cfs
- Uncertainty in area affected by 23100 cfs

0 0.5 1 2 3 Kilometers

0 0.5 1 2 3 Miles

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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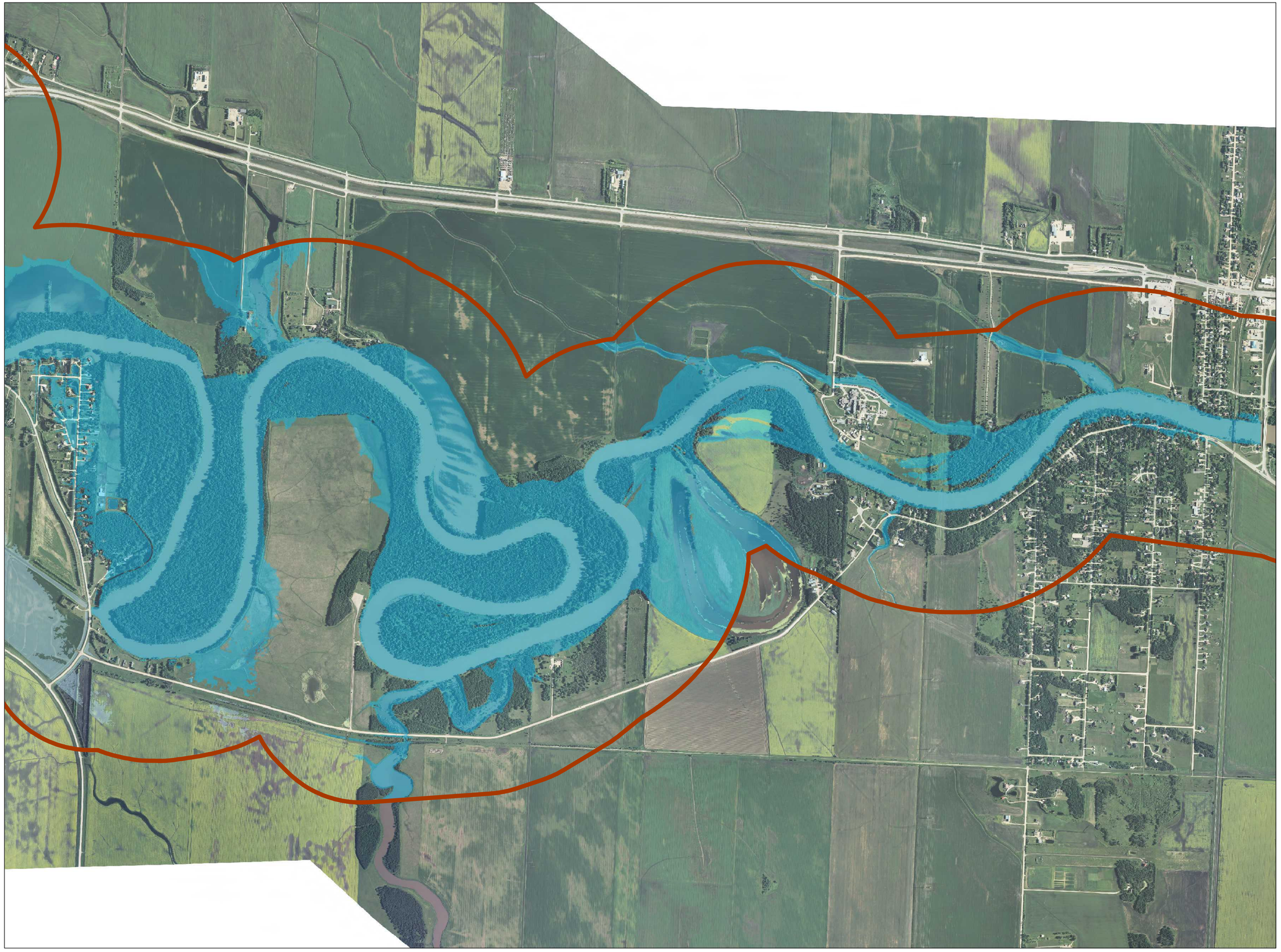
Baie St Paul to Headingley
Flood Extent:
KGS 23,100 cfs

Map 5 of 5

Flood level based on
the 2015 KGS Water Surface Profile



LiDAR Extent
Assiniboine River Dike

2015 KGS

Area affected by 23100 cfs
Uncertainty in area affected by 23100 cfs

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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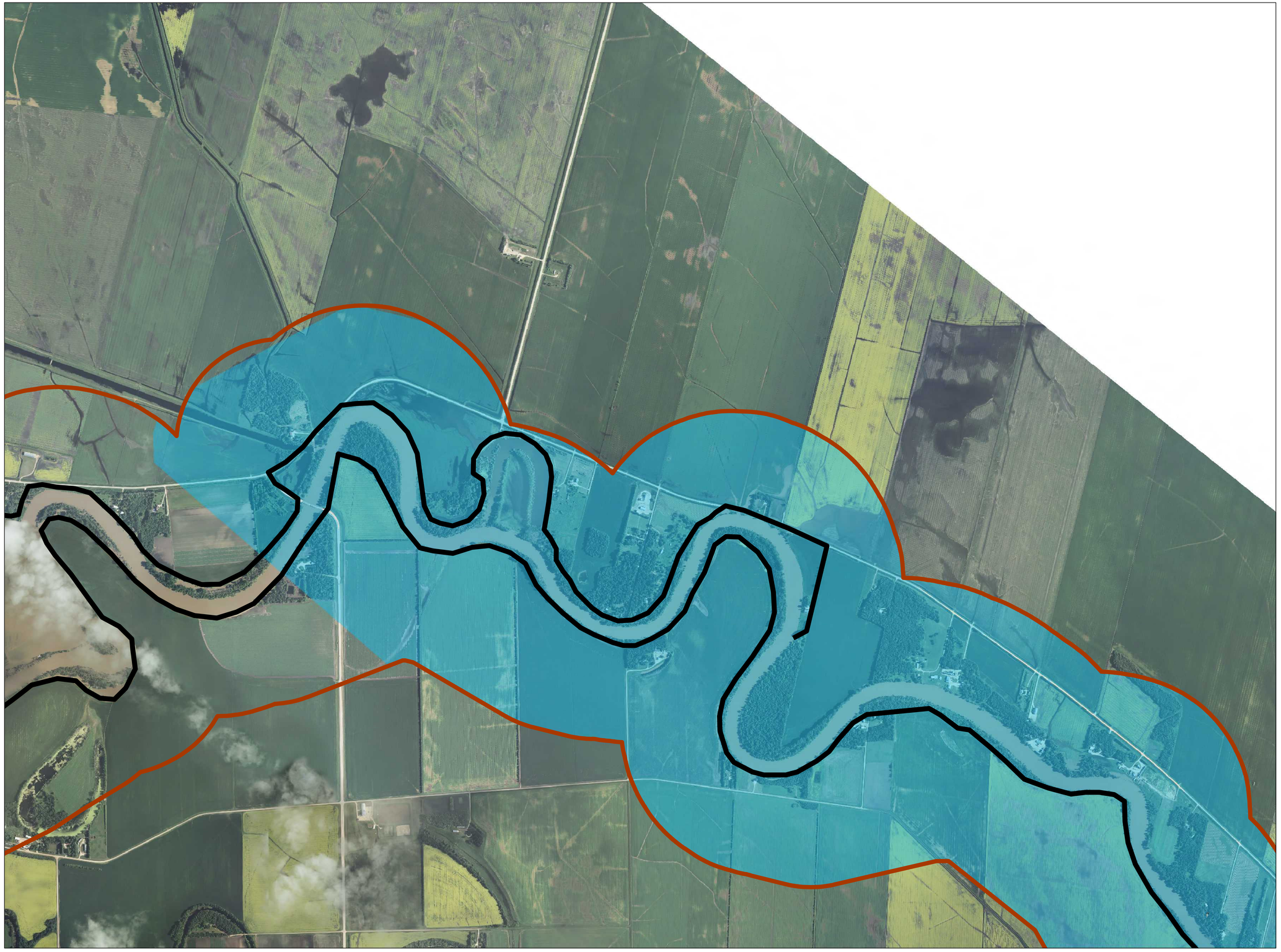
Baie St Paul to Headingley
Flood Extent:
KGS 28,000 cfs

Map 1 of 5

Flood level based on
the 2015 KGS Water Surface Profile



LiDAR Extent
Assiniboine River Dike
2015 KGS
Area affected by 28000 cfs
Uncertainty in area affected by 28000 cfs

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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Baie St Paul to Headingley
Flood Extent:
KGS 28,000 cfs

Map 2 of 5

Flood level based on
the 2015 KGS Water Surface Profile

LiDAR Extent
Assiniboine River Dike
2015 KGS
Area affected by 28000 cfs
Uncertainty in area affected by 28000 cfs

0 0.5 1 2 3 Kilometers
0 0.5 1 2 3 Miles

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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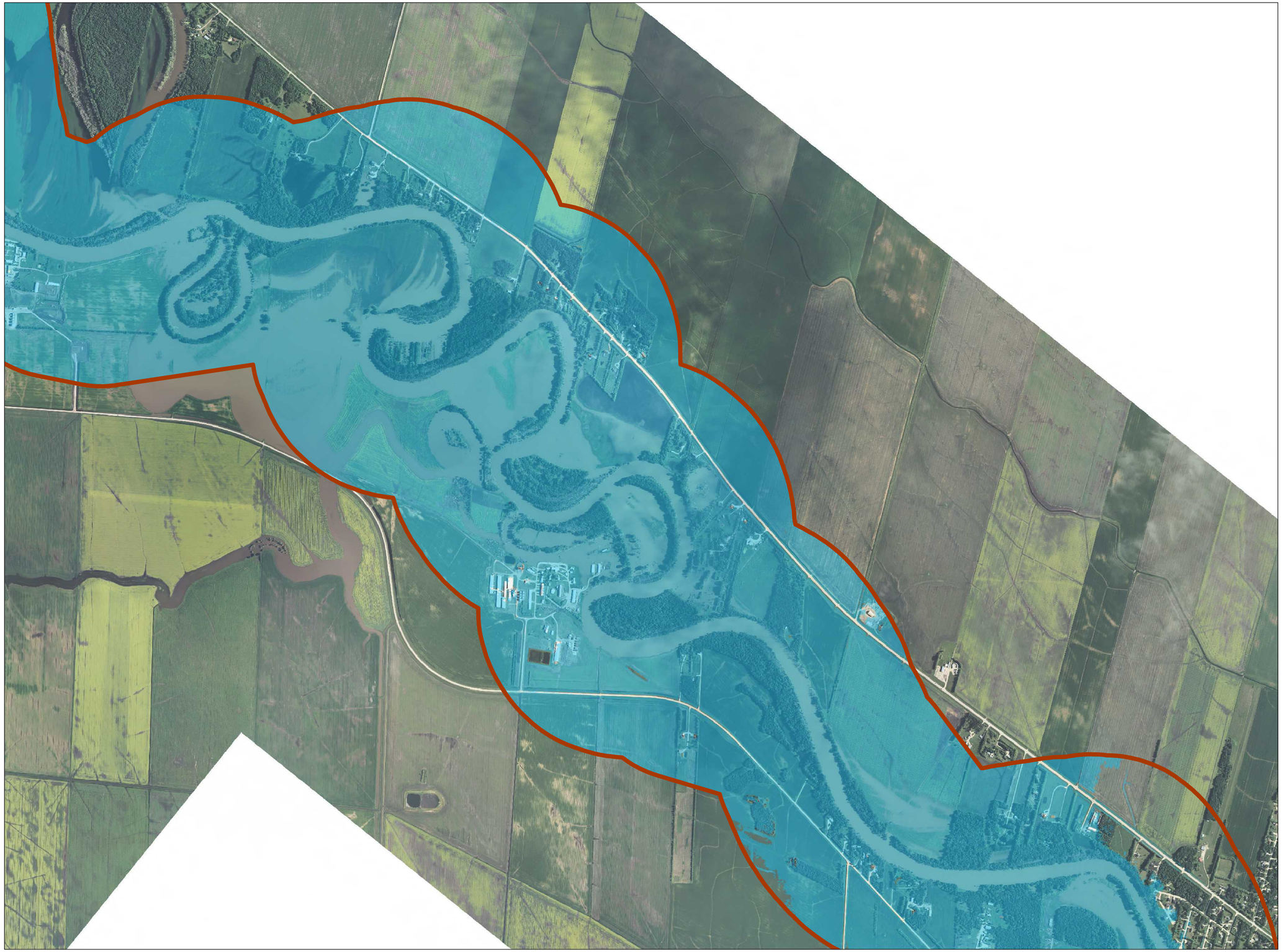
Baie St Paul to Headingley
Flood Extent:
KGS 28,000 cfs

Map 3 of 5

Flood level based on
the 2015 KGS Water Surface Profile



For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

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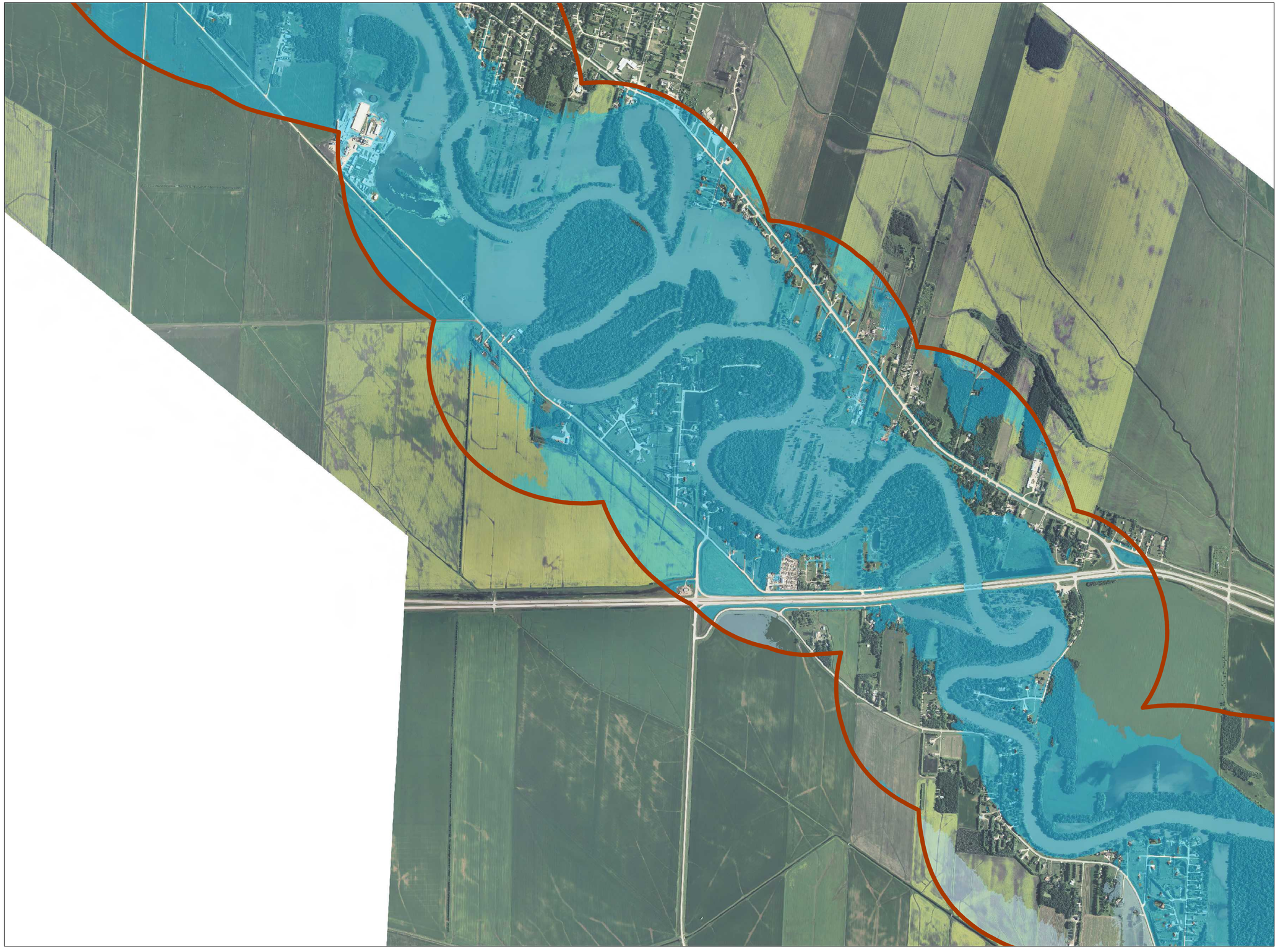
Feb 4, 2015

Baie St Paul to Headingley
Flood Extent:
KGS 28,000 cfs

Map 4 of 5

Flood level based on
the 2015 KGS Water Surface Profile



LiDAR Extent
Assiniboine River Dike

2015 KGS

Area affected by 28000 cfs
Uncertainty in area affected by 28000 cfs

0 0.5 1 2 3 Kilometers
0 0.5 1 2 3 Miles

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

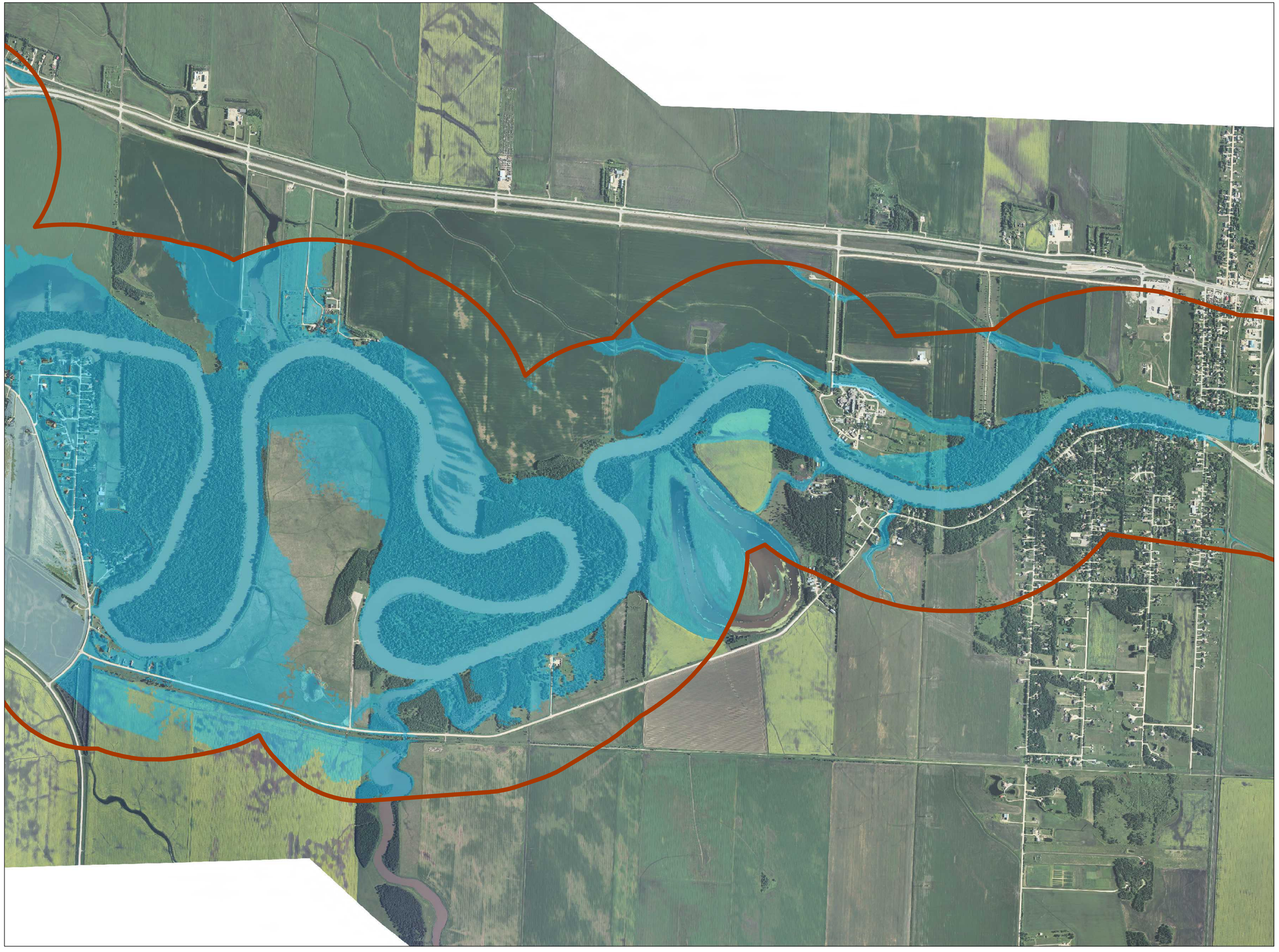
Manitoba
Infrastructure and Transportation

Water Management, Planning and Standards
Feb 4, 2015

Baie St Paul to Headingley
Flood Extent:
KGS 28,000 cfs

Map 5 of 5

Flood level based on
the 2015 KGS Water Surface Profile



LiDAR Extent
Assiniboine River Dike
2015 KGS
Area affected by 28000 cfs
Uncertainty in area affected by 28000 cfs

For Discussion Purposes Only
Not to Scale

Source:
KGS 2015 Water Surface Profile
2007 Assiniboine LiDAR
2014 Assiniboine Flood Imagery

Manitoba
Infrastructure and Transportation

Water Management, Planning and Standards
Feb 4, 2015

APPENDIX K

SUMMARY TABLE OF ENVIRONMENTAL MITIGATION COSTS

APPENDIX K - Summary List of Flood Mitigation Alternatives & Estimated Level of Effort for Environmental Mitigation

Study Area	Concepts	Options Considered	Capacity of Options	Environmental Mitigation	
				Estimated Level of Effort	Comments
Upper Assiniboine	Diking along Upper Assiniboine River	Shellmouth to Brandon / St. Lazare	Varies	Minor	New diking system. Raising of water levels. Oxbows & small streams cut-off from river.
	"Buyout" Scheme along Upper Assiniboine River	Shellmouth to St. Lazare	3,000 cfs	Negligible	No changes to flooded area.
	Developing Storage on the Assiniboine River and Tributaries	Alexander Dam	1,600,000 dam^3	Major	Potential for significant environmental impacts. Flooded area greater than 1500 ha.
		Holland Dam	880,000 dam^3	Major	Potential for significant environmental impacts. Flooded area greater than 1500 ha.
		Victor Dam	170,000 dam^3	Major	Potential for significant environmental impacts. Flooded area greater than 1500 ha.
		Zelena Dam	250,000 dam^3	Major	Potential for significant environmental impacts. Flooded area greater than 1500 ha.
	Mitigation Alternatives in the City of Brandon	Raise Dikes	40,500 cfs	Negligible	Upgrades to existing diking system.
Souris River	Developing Storage on the Souris River	High Souris Dam	40,000 dam^3	Major	Potential for significant environmental impacts. Flooded area about 1500 ha.
		Nesbitt Dam	410,000 dam^3	Major	Potential for significant environmental impacts. Flooded area greater than 1500 ha.
	Assessment of Diking Options	Raise dikes in Melita, Souris, and Wawanesa	Varies	Negligible	Upgrades to existing diking system
Lower Assiniboine	Improvement Schemes for the Lower Assiniboine Dikes	Upgrade Existing Dikes	18,000-35,000 cfs	Negligible	Upgrades to existing diking system
		Major Setbacks	18,000-35,000 cfs	Moderate	Flooding of land previously protected by dikes. New ROW requirements.
		Combination of Upgrades and Setbacks	18,000-35,000 cfs	Minor	"Average effects" between the two options above.
	Improvement Schemes Downstream of the Assiniboine Dikes	Expansion to Headingley	18,000-35,000 cfs	Minor	New diking system and ROW requirements. Potential seepage issues.
		Combination of individual flood proofing, neighbourhood diking and purchase of vulnerable properties.	18,000-35,000 cfs	Negligible	No changes to flooded area. Seepage issues addressed separately.
	Individual & Community Flood Protection	Portage to Headingley	23,100 cfs	Major	Flooding of land previously protected by dikes upstream of Baie St. Paul
	Control Structure at Hoop and Holler Bend on Assiniboine River	"Wasteway" only	3,900 cfs	Moderate	Deliberate flooding of land. Interbasin transfer. Increased flows D/S on La Salle R.
		Control Structure with "Wasteway"	3,900 cfs	Moderate	Deliberate flooding of land. Interbasin transfer. Increased flows D/S on La Salle R.
		Control Structure with Diversion Channel	3,900 cfs	Minor	Interbasin transfer. Increased flows D/S on La Salle R.
		Control Structure with Diversion Channel and La Salle R. Expansion	>3,900 cfs	Major	Interbasin transfer. Dredging of River
Portage Diversion	Elimination of the Portage Diversion Failsafe	Raise Portage Diversion Dikes	18,000/25,000 cfs	Negligible	Improvements due to elimination of flooded area. Pre-existing dikes.
		Buyout	25,000 cfs	Negligible	No changes to flooded area.
		Dike Agricultural Land	25,000 cfs	Negligible	Reduction in flooded area. New diking system
		Diking Cram Creek	25,000 cfs	Minor	New dikes located in wetland complex. Reduction in flooded area.
	Increasing Capacity of Portage Diversion	Channel Widening	34,000 cfs	Minor	Increased flow. Larger footprint.
		Parallel Channel	34,000/46,000 cfs	Moderate	Increased flow. Double the infrastructure. Increased ROW requirements. FN near inlet.
		Retrofit	34,000 cfs	Minor	Increased flow, WL and velocities.
Dauphin Lake	Diversion Channel	Short Channel (30 m & 60 m wide)	4,000 cfs	Minor	New channel
		East Channel	4,000 cfs	Minor	New channel
		Tributary Channel	4,000 cfs	Moderate	New Channel & increased river flow
		West Channel	4,000 cfs	Moderate	Longer channel. Intercepts multiple streams.
	Mossy River Excavation	Excavate Channel by 0.3 m , 0.5 m, 1.0 m	Varies	Major	Dredging of river.
	Upstream Storage	33 to 156 Reservoirs/Dams	11,000 ha	Moderate	Potential for significant environmental impacts - but to lesser degree than large dams. High number of locations, but less than 1500 ha each.
Lake Winnipegosis	Lake Winnipegosis Diversion	Diversion to Cedar Lake	2,500-10,000 cfs	Moderate	Interbasin transfer. Potential for FN impacts around Cedar Lake. Changes to natural LWPGosis regime
		Diversion to Lake Winnipeg	10,000 cfs	Moderate	Interbasin transfer. Numerous stakeholders around LWPG. Changes to natural LWPGosis regime
	Lake Winnipegosis Outlet	Control Structure on Waterhen River	6,000 cfs	Major	Increased WL. New lake regulation. Changes to Waterhen River regime. FN around LWPGosis and on Waterhen.
	Storage on Swan Lake	Control Structure on Shoal River	N/A	Moderate	Increased WL. New lake regulation.
Lake Manitoba and Lake St. Martin	LMB & LSM Outlet Channels	Option A - Twinning the Fairford River	5,000 - 15,000 cfs	10%	Increased WL and flows on Fairford River and Lake Pineimuta. FN impacts.
		Option B - Channel South of Pinaymootang FN	5,000 - 15,000 cfs	5%	New channel.
		Option C1 - Channel less South of Pinaymootang FN	5,000-15,000 cfs	5%	New channel.
		Option D1 - Channel following Birch Creek	5,000-15,000 cfs	7%	New channel.
		Option E - Bypass Channel North of FRWCS	2,500 - 3,750 cfs	8%	Increased WL and flows on Fairford River and Lake Pineimuta. FN impacts.
		Option F - Expansion of Fairford R. & FRWCS (+dredging)	5,000 cfs	15%	Dredging of River.
Varies	Multitude of Small Dams	Total of 21 Dams	200,000 dam^3	Moderate	Potential for significant environmental impacts - but to lesser degree than large dams. High number of locations, but less than 1500 ha each.
	Wetlands Restoration	10, 20, 30% Restoration	73,000-219,000 ha	Negligible	

Notes: Estimated Level of Effort for Environmental Mitigation
Negligible - assumed part of contingency
Minor - assumed 5% of direct costs
Moderate - assumed 10% of direct costs
Major - assumed 15% of direct costs

APPENDIX L

RADAR SATELLITE IMAGERY

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N W E S

Rat Creek
242
Westbourne
Longburn
Rignold
Rat Creek
Yellowhead Route
16
632
Macdonald
227
Long Creek
House Creek
Big Lake
Long Creek
Delta Game Bird Refuge
Delta Beach
Cadham Bay
Delta Marsh
Fulton
Oakland
240
Simpson Bay
Delta Marsh
Portage Creek
Portage Creek
Bluebill Bay
Long Lake Drain
High Bluff
624

Brandon
Winnipeg
Portage la Prairie

DRAFT

1 0 1 2 3 4
Kilometres

SCALE: 1:100,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

RADARSAT IMAGERY
MAY 12, 2011

PRELIMINARY
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JULY 2015 | FIGURE L-1 | REV: A

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Delta Beach
Bluebill Bay
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Simpson Bay
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Portage Creek
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Yellowhead Route
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Brandon
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Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

RADARSAT IMAGERY
MAY 18, 2011

JULY 2015 | FIGURE L-2 | REV: A

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11"x17" PLOT SCALE 1:1

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N W E S

Rat Creek
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Macdonald
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Rat Creek
Yellowhead Route
Delta Game Bird Refuge
Delta Beach
Cadham Bay
Fulton
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Delta Marsh
Simpson Bay
Delta Marsh
Portage Creek
Portage Creek
Bluebill Bay
Long Lake Drain
High Bluff

242 242 227 227 240 16 632 624

Brandon
Winnipeg
Portage la Prairie

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SCALE: 1:100,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

RADARSAT IMAGERY
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JULY 2015 | FIGURE L-3 | REV: A

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Long Creek
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Portage Creek
Rignold
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Long Lake Drain
High Bluff
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Brandon
Winnipeg
Portage la Prairie

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Kilometres

SCALE: 1:100,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

RADARSAT IMAGERY
MAY 22, 2011

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Delta Game Bird Refuge
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Cadham Bay
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Portage Creek
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Brandon
Winnipeg
Portage la Prairie

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Kilometres

SCALE: 1:100,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

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Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

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MAY 25, 2011

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JULY 2015 | FIGURE L-5 | REV: A

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APPENDIX M

POTENTIAL EFFECTS OF WETLAND RESTORATION ON ASSINIBOINE RIVER FLOODS

POTENTIAL EFFECTS OF WETLAND RESTORATION ON ASSINIBOINE RIVER FLOODS

Background

A recent report on a detailed study of the effects of drainage in the Smith Creek watershed, conducted by the Centre for Hydrology, University of Saskatchewan (CHSask, 2014) has suggested that drainage of wetlands since 1958 has greatly increased runoff volumes and peak flows. Smith Creek is a tributary of the Assiniboine River, which flows out of Saskatchewan and joins the Assiniboine River between Shellmouth Dam and PTH 16. Considerable drainage of wetlands is known to have occurred in most other sub-basins of the Assiniboine River watershed. This has raised the issue of whether restoration of wetlands could be a viable flood mitigation measure for the Assiniboine River. A brief study, as described herein, has been conducted to estimate the effect that wetland restoration could have on Assiniboine River flood magnitudes. Restoration of wetlands, for purposes of this study, consists of reversing some of the wetland drainage, as well as development or enhancement of additional wetlands using gated culverts or other structural measures. The extent of such possible works is very uncertain since the characteristics of wetlands are poorly defined for the Assiniboine Basin as a whole and the feasibility, environmental impacts, landowner compliance and costs of mitigation works would need to be considered. Nevertheless, an estimate of the potential impacts of wetland drainage on runoff and on the flow hydrographs of the Assiniboine River are needed as a basis for a benefit-cost study of wetland mitigation. Results from the CHSask study have been used to make rough estimates of possible decreases in runoff volume due to wetland restoration. The main objective of this brief study was to determine what effect certain changes in runoff would have on flood hydrographs of the Assiniboine River. Many assumptions and approximations were made due to lack of sufficient information on wetlands for the many sub-watersheds of the Assiniboine Basin other than Smith Creek, and due to the severe time limitations for this study.

Report on Drainage Effects at Smith Creek

A report entitled “Improving and Testing the Prairie Hydrologic Model at Smith Creek Research Basin” by Pomeroy et al. (2014) was officially released by the Centre for Hydrology, University of Saskatchewan, in early June, 2014. While the report deals primarily with development of a prairie hydrologic model (PHM), which can simulate the effects of drainage on runoff, it also reports significant findings on the effects of drainage on runoff volumes and peak flows for Smith Creek. Simulations of runoff changes are based on the effects of wetland drainage during the period 1958 to 2008, the years where information on wetlands was collected using various means including a detailed LiDAR survey in 2008. During this period, the extent of wetlands was estimated to have decreased from 24% to 11% of the watershed area. The year 2008 was assumed to represent the ‘present condition’ since it was the last year that detailed information on wetlands was available. The model was run with watershed data for the 6 year period from 2007 to 2013 and the effects on runoff volume and peak flows were noted. Readers are referred to the Pomeroy et al. report for details of this intensive 5-year study. It is not the purpose of the current study document herein to evaluate the reliability of the CHSask model results; rather, the objective is to determine what the findings imply for Assiniboine River flood magnitudes. This requires the extrapolation of the Smith Creek results to the many other sub-basins of the Assiniboine River and Souris River and the conversion of expected changes in runoff to stream flow hydrographs of the Assiniboine River main stem. The first step is to understand the significant findings of the Smith Creek study, which are briefly summarized in the following sections.

Effects of Wetland Drainage and Restoration on Smith Creek Runoff

Prairie Hydrologic Model simulations on Smith Creek indicated an average 55% increase in annual runoff volumes for the 6-year period of study, based on wetland changes from 1958 to 2008. The increase was greatest for years with low runoff, ranging from 200% to 300% increases. The increase in runoff volume for the largest flood on record (2011) was computed at 42% based on wetland changes between 1958 and 2008 and was estimated to increase a further 28%, if complete drainage of wetlands after 2008 were to occur. Information pertaining to the effects of wetland changes on Smith Creek peak flows was more difficult to simulate and was less reliable. Therefore, it will not be further considered in this report.

The report on Smith Creek also presented percentage decreases in runoff volume, which might occur due to restoration of wetlands. These were considerably smaller than the percent increases due to wetland drainage. However, the absolute value of the changes in runoff (rather than the percent changes) were very similar for the cases of drainage and restoration of wetlands. For example, if drainage increased runoff volume from 500 dam^3 to 800 dam^3 (405 to 650 ac-ft), the percentage increase would be 300/500 or 60%. However, the decrease due to restoration was computed as 300/800 or 37.5%.

Since this brief study is about the effects of wetland restoration, the starting point in terms of wetlands is assumed to be the 2008 wetland state. The percentage reductions due to wetland restoration are, therefore, very significant since any changes in runoff would be divided by runoff for the 2008 wetland condition.

The simulations showed that restoration from 2008 to 1958 wetland conditions resulted in an average runoff decrease of 21% to 60% for moderate to low flow years. The decrease for the 2011 flood was 29.4% going from the 2008 wetland state back to 1958 and 44.7% going from complete drainage back to 1958 conditions. While the effects of wetland storage available for runoff reduction vary with antecedent moisture conditions, the Smith Creek results suggest that, for major floods preceded by the relatively empty natural wetlands of the 1958 era, wetland restoration of a 2008 condition could reduce runoff by about 30%. The reduction could be much less for flood events preceded by major floods in the antecedent year or two, which would have filled the wetlands existing in 1958, leaving little or no space to store additional water. The summer flood of 2012 is an example of such a condition. Wetland restoration to the 1958 condition produced virtually no reduction in runoff for 2012 since the extreme flood of 2011 would have filled the wetland existing in 1958, leaving no room to store water in 2012.

Transferring Smith Creek Results to Other Parts of Assiniboine Watershed

The Smith Creek watershed may be somewhat unique in that it had very extensive wetlands as of 1958. This presented the opportunity for landowners to greatly increase the area of productive lands by means of drainage. Even many deep wetlands have been drained using heavy equipment, which most farmers presently own. Many other sub-watersheds of the Assiniboine River and Souris River have fewer wetlands. This raised the question as to how applicable the Smith Creek results were to other portions of the Assiniboine watershed. Perhaps the percentage reduction on runoff due to wetland restoration could be considerably less in other areas. The pertinent question seems to be whether there is sufficient opportunity in terms of wetlands that could be restored or enhanced to achieve certain targets of runoff reduction. The opportunity would depend on whether there were enough wetlands with which to work. A map for the prairie region showing the natural non-contributing drainage areas produced by the former Prairie Farm Rehabilitation Administration / Agriculture and Agri-Food Canada (PFRA and AAFC, 2008) was used to estimate the percentage of drainage consisting of wetland for

each tributary of the Assiniboine and Souris rivers. The three foot average depth is a very approximate estimate as no inventory of the millions of prairie wetlands in terms of their size and depth was available. This estimate was based on discussions with a number of farmers in western Manitoba and was considered a reasonably achievable target. The available storage volume was then estimated based on an average incremental depth of three feet due to restoration works. The calculations for each tributary are shown on Table 1. The purpose of this exercise was to determine, very approximately, whether each sub-watershed had sufficient storage opportunity to accommodate as much as 30% of the 2011 runoff volume, the 30% being taken as the upper limit for large floods based on the Smith Creek study. The data in Table 1 shows that for each tributary the storage volume opportunity due to wetland restoration (Column 8) was larger than 30% of the 2011 runoff volume (Column 12).

The remaining issue is that of uncertainties in the Smith Creek estimate of percent runoff decrease due to wetland restoration and uncertainties in the estimates of potential wetland restoration volume opportunities in the many sub-watersheds of the Assiniboine and Souris River watersheds. An actual program of wetland restoration would also be subject to the feasibility of construction works, environmental issues, agreements and approvals from landowners and costs. Such factors could vary significantly from one watershed to another. Due to the many uncertainties, it was decided to use a range of wetland restoration volumes for each sub-watershed, expressed as a target percentage reduction in runoff volume. This would enable a sensitivity analysis of restoration volumes versus effect on Assiniboine River flood hydrographs. It would also provide useful information for a benefit-cost analysis of various degrees of wetland restoration.

The wetland restoration volumes chosen for examination of effects on floods were those equivalent to 10%, 20% and 30% reductions in runoff volume for large floods with near normal antecedent wetland conditions.

Study Framework

Most of the Assiniboine River watershed lies outside of Manitoba. This presents particular difficulties with regard to data and operation of flood control works in outside jurisdictions that affect river flows in Manitoba. The portion of the Souris River lying upstream of the Manitoba-North Dakota boundary, i.e. upstream of the river gauge at Westhope, presents particular difficulties due to international issues and since it is highly controlled by reservoirs such as Rafferty, Alameda, Boundary, Lake Darling and the Fish and Wildlife Refuges near Upham, N. Dakota. As a consequence, this portion of the Souris River was excluded from the study; and flow hydrographs for the Souris River at Westhope were used without change. All other portions of the Assiniboine watershed from the headwaters to Portage la Prairie were included. The vast Qu'Appelle River watershed was treated as one unit, i.e. there was no attempt made to estimate changes in runoff and flows from its many tributaries in Saskatchewan. This was necessary due to lack of sufficient data and the time constraints of this study.

The focus of this brief study was on the potential effect of wetland restoration on three large Assiniboine River floods such as occurred in 1976, 1995 and 2011. Since floods differ with respect to causes and spatial distributions of runoff, it was deemed that this would provide a better perspective of restoration effects than the use of a single flood. The 1976 snowmelt flood produced record high peak flows (at the time) on the Assiniboine River and Souris River with peaks flows at Brandon and at Wawanesa being nearly equal. The 1995 Flood was very severe in the upper Assiniboine River, while the 2011 Flood produced new record high flows from St. Lazare to Portage la Prairie, as well as on the Souris River. The 2011 Flood was extended in

time and produced a much higher volume of flow than that of 1976, even though peak flows at Portage la Prairie were similar. The return periods of peak flows for the floods at Portage, based on frequency curves developed by MIT and adopted for the current study, were approximately 140, 20 and 140 years for the 1976, 1995 and 2011 floods, respectively.

Use of data for these recorded floods would facilitate the calculations and help relate the findings to some real flood events. Tributary hydrographs for the Assiniboine River and Souris River would need to be reduced based on each of the three degrees of wetland restoration, and the resultant hydrographs would need to be flood routed to obtain the effects on main stem flood hydrographs. The upper Assiniboine watershed, including the portion upstream of Shellmouth Dam, had extensive wetlands in its natural state; and, therefore, it was of interest to determine possible effects of wetland storage on Shellmouth Reservoir inflows, operations and outflows. Accordingly, inflows were adjusted and routed through the Reservoir. Outflows were then routed down the Assiniboine River to Portage la Prairie using recorded tributary hydrographs and hydrographs reduced by 10%, 20% and 30% due to wetland restoration. Souris River flows were routed from Westhope to Wawanesa, both with unadjusted and adjusted tributary flows. Routed flows for Wawanesa were then input to the Assiniboine River flood routings.

It is important to note that the 1976, 1995 and 2011 floods were chosen to represent different flood types in terms of the climatic forcing factors rather than as exact recorded events for the purpose of this study. While the routed Assiniboine flows based on no wetland restoration closely resemble the recorded events at most river points, there may be significant differences due to operation of the Shellmouth Dam for this study. Operations for this study deviated from actual operations during those years. These differences are most manifest in the routed flows of the Assiniboine River near Russell. The focus of this study is on the differences on the Assiniboine and Souris River flow hydrographs (see Annex A) resulting from various reductions in runoff due to wetland restoration, which required that Shellmouth Dam operation be based on rigid application of a single set of operating rules.

Conversion of Runoff Reductions to Stream Flow

As previously noted, observed tributary hydrographs were reduced by 10%, 20% and 30% to account for various degrees of wetland restoration. For each hydrograph, it was assumed that the daily discharge would be reduced by 10%, 20% and 30%. This resulted in an equivalent 10%, 20% and 30% reduction to the total runoff volume for each hydrograph. However, restoration of wetlands would not only reduce runoff volumes, but would also delay runoff. The Prairie Cascade Model (PCM) developed by K. Shook of CHSask shows that runoff moves rapidly from one wetland to the next for a highly drained condition, thus reaching the outlet of the watershed earlier than for the pre-drainage condition. Wetland restoration would slow runoff as water is held back in storage. The effect of wetland storage on the shape of a tributary hydrograph is difficult to produce without the aid of a complex model such as PCM. It would take much data and time to apply the model to other sub-watersheds. A simplification was adopted in that the tributary hydrographs for the 10% runoff reduction condition were simply delayed by one day. Those for the 20% runoff reduction were delayed by two days and for the 30% reduction were delayed by three days. Admittedly, these adjustments in timing are crude but were considered to be reasonable estimates. There was insufficient time and modelling capability to accurately modify hydrograph shapes for this study.

Some larger areas of ungauged flow are difficult to index to gauged streams due to their unique response to moisture inputs, which are not well represented by the limited tributary data in the area. One such area is that located between Brandon and Portage la Prairie downstream of the

gauges at Brandon and Wawanesa. This area is treated as a single sub-basin in the Assiniboine River flood routing model. Flows from this area were first determined by flood routings of Brandon and Wawanesa flows to the gauge at Holland (PTH 34) and noting the difference between routed and recorded flows at Holland. Flows were similarly computed for a large ungauged area around the Shellmouth Reservoir, downstream of the gauges at Kamsack and Inglis. Hydrographs for both areas were thus computed for the 1976, 1995 and 2011 spring floods.

Shellmouth Reservoir Flood Routings and Dam Operation

Inflow hydrographs for Shellmouth Reservoir were obtained by computing daily changes in reservoir storage, converting these volumes to equivalent daily discharges and adding the reservoir outflows. The results were compared to inflows computed prior to this study with generally good agreement. The computed inflow hydrographs were multiplied in turn by 0.9, 0.8 and 0.7 to produce hydrographs for the range of runoff reductions as discussed previously.

Shellmouth inflows were routed through the reservoir to determine outflows for the three wetland restoration scenarios. This presented some difficulty, since operation modes used for the recorded floods of the 1970's and 1990's were no longer in use. Furthermore, the modes of operation varied significantly for these floods. For the sake of consistency in evaluation of wetland restoration effects, the following operating rules were employed for Shellmouth Dam:

1. Winter drawdown level for all years with high spring runoff potential is 422.6 m (1,386 feet).
2. Outflow is set at 14 m^3/s (500 cfs) until reservoir reaches a level of 425.3 m (1,395 feet).
3. When reaching 425.3 m (1,395 feet), outflow is raised to 40 m^3/s (1,500 cfs) unless inflow is less than 140 m^3/s (5,000 cfs) and declining, in which case outflow is left at 14 m^3/s (500 cfs).
4. When reservoir level reaches 426.7 m (1,400 feet), and inflow is 285 m^3/s (10,000 cfs) or higher and still rising, increase conduit flow to 140 m^3/s (5,000 cfs). If inflow at this point is less than 210 m^3/s (7,500 cfs), maintain a 40 m^3/s (1,500 cfs) outflow.
5. When reservoir has declined to 428.7 m (1,406.5 feet), reduce outflow to match inflow, but with a minimum of 40 m^3/s (1,500 cfs).
6. When reservoir has declined to 427.5 m (1,402.5 feet), set outflow equal to inflow, but do not exceed 40 m^3/s (1,500 cfs) unless reservoir rises above 428.7 m (1,406.5 feet).

The aggressive operation, where outflows were increased to 140 m^3/s (5,000 cfs) soon after a water excess was evident, follow the operation mode of 2014 and was based on the desire of downstream landowners to have flooding occur earlier rather than later. In an actual flood situation, operations may be guided also by other downstream concerns; but for purposes of this study, such complexities were avoided. Based on these rules of operation, the inflows were routed through the reservoir for each of the three flood years based on inflows for no wetland restoration and for the three restoration scenarios discussed previously.

Muskingum Flood Routings for Souris and Assiniboine

Souris River routings using the Muskingum method were done for 1976, 1995 and 2011 for each of the four sets of tributary hydrographs, namely the case with no wetland restoration and the cases of 10%, 20% and 30% runoff reduction due to wetland restoration. Recorded flows at Westhope were used for all of these routings due to no restoration being considered in North Dakota and further upstream. Routed hydrographs for Wawanesa were then input to Assiniboine River routings from Shellmouth to Portage la Prairie. Trial Muskingum routings were

done beforehand with recorded data to determine the routing parameters and the appropriate multiplying factors for ungauged areas, which are indexed to appropriate nearby streams.

The output hydrographs from the flood routings were plotted for the following locations and are attached in Annex A:

- Lake of the Prairies levels (Shellmouth Reservoir).
- Assiniboine River flows near Russell.
- Assiniboine River flows near Miniota.
- Assiniboine River flows at Brandon.
- Souris River flows at Wawanesa.
- Assiniboine River flows at Portage la Prairie.

Each plot depicts flow hydrographs for the four conditions, namely for no wetland restoration and for three degrees of wetland restoration.

The bankfull capacities also shown on the figures relate mainly to the flows at which flooding of valley agricultural lands begins. A serious flood threat to low lying parks in the City of Brandon begins at about 340 m^3/s (12,000 cfs) and developments near 18th Street are protected to flows near 1,020 m^3/s (36,000 cfs). At Portage la Prairie, the channel capacity shown is for areas somewhat upstream of the Portage Reservoir, near Long Plains Reserve. Significant problems with operation of flood control infrastructure at Portage la Prairie begin at flows above 1,220 m^3/s (43,000 cfs).

Summary and Discussion of Results
A summary of the reductions in peak flows due to wetland restoration, for several locations along the Assiniboine River and Souris River is provided in Table 2.

The results summary and the hydrograph plots show modest but perhaps significant main stem flow reductions due to wetland restoration. The effect is relatively small on the Souris River due to the assumption that no wetland restoration would occur on the large basin area upstream of Westhope, North Dakota. The results of this study suggest that wetland restoration could be an effective way to reduce flood hydrographs on the Assiniboine River for various types of floods. However, the reduction in flows based on even a 30% reduction in runoff would not be sufficient to prevent agricultural flooding for major floods such as those similar to the 1976, 1995 and 2011 floods. It would very likely have the potential to prevent minor flooding for smaller floods, but such events are of lesser significance and would provide less justification for large expenses on wetland restoration.

The reduction of 55 m^3/s to 85 m^3/s (2,000 cfs to 3,000 cfs) for even the 10% runoff reduction scenario could be very significant for the City of Brandon and for the management of flows at Portage la Prairie, especially for major floods which threaten infrastructure in these areas. Benefits for the Portage Diversion capacity and lower levels on Lake Manitoba and the Assiniboine River from Portage la Prairie to Winnipeg could be very significant also. Such benefits would increase significantly for greater degrees of wetland restoration such as those resulting in 20% or 30% reductions in runoff. However, such levels of wetland restoration would be difficult and costly to achieve, so the benefit-cost ratios might not be favourable.

This study was very brief and required many assumptions and simplifications. A more detailed assessment would be needed to enable a more reliable benefit-cost analysis before an

extensive wetland restoration program might be considered. This study, followed by a preliminary b/c analysis, should at least point to the advisability of conducting a more detailed analysis.

References:

Pomeroy, J.W., Shook, K., Fang, X., Dumanski, S., Westbrook, C. and Brown, T., 2014, Improving and Testing the Prairie Hydrologic Model at Smith Creek Research Basin, Centre for Hydrology, University of Saskatchewan.

Prairie Farm Rehabilitation Administration (PFRA) / Agriculture and Agri-Food Canada (AAFC), March 31, 2008, PFRA Watershed Project – Areas of Non-Contributing Drainage.

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TABLES

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Table 1
Wetland Volume Estimates for 2011 Flood

	1	2	3	4	5	6	7	8	9	10		11		12	
		Wetland Conditions in 2008				2011	2011	2011	2011	Fractional Runoff Reduction due Wetland Storage during 2011 Flood					
Watershed	Gross Drainage Area* (acres)	2008 Wetland % of D. Area from Map*	Wetlands Area (acres)	Average Depth Increase (ft)	Maximum Wetland Volume (ac-ft)	Antecedent Climatic Condition Wet = 0.9, Dry = 0.1	Wetland Storage Already Utilized at Beginning of Flood (ac-ft)	Potential Restoration Volume (ac-ft)	Observed Runoff Volume (ac-ft)	Low Volume		Medium Volume		High Volume	
										10%	Ac-Ft	20%	Ac-Ft	30%	Ac-Ft
U/S Kamsack	3,212,160	15	481,824	3	1,445,472	0.3	433,642	1,011,830	1,018,947	0.10	101,895	0.20	203,789	0.30	305,684
Shell River	487,040	10	48,704	3	146,112	0.3	43,834	102,278	194,921	0.10	19,492	0.20	38,984	0.30	58,476
Basin 3	620,160	15	93,024	3	279,072	0.3	83,722	195,350	222,068	0.10	22,207	0.20	44,414	0.30	66,620
Conjuring	17,408	10	1,741	3	5,222	0.3	1,567	3,656	8,791	0.10	879	0.20	1,758	0.30	2,637
Smith	487,040	24	116,890	3	350,669	0.3	105,201	245,468	53,449	0.10	5,345	0.20	10,690	0.30	16,035
Silver	112,640	15	16,896	3	50,688	0.3	15,206	35,482	59,851	0.10	5,985	0.20	11,970	0.30	17,955
Cutarm	189,440	20	37,888	3	113,664	0.3	34,099	79,565	57,442	0.10	5,744	0.20	11,488	0.30	17,233
Welby	12,576,000	15	1,886,400	3	5,659,200	0.3	1,697,760	3,961,440	1,491,335	0.10	149,134	0.20	298,267	0.30	447,401
Birdtail	272,000	14	38,080	3	114,240	0.3	34,272	79,968	172,831	0.10	17,283	0.20	34,566	0.30	51,849
Arrow	149,760	16	23,962	3	71,885	0.3	21,565	50,319	65,360	0.10	6,536	0.20	13,072	0.30	19,608
Gopher	74,240	15	11,136	3	33,408	0.3	10,022	23,386	36,642	0.10	3,664	0.20	7,328	0.30	10,993
Oak	289,280	20	57,856	3	173,568	0.3	52,070	121,498	80,900	0.10	8,090	0.20	16,180	0.30	24,270
Little Sask	960,640	15	144,096	3	432,288	0.3	129,686	302,602	364,913	0.10	36,491	0.20	72,983	0.30	109,474
Brandon-PG	1,304,320	10	130,432	3	391,296	0.3	117,389	273,907	584,530	0.10	58,453	0.20	116,906	0.30	175,359
Waskada	24,320	12	2,918	3	8,755	0.3	2,627	6,129	42,864	0.10	4,286	0.20	8,573	0.30	12,859
Antler	793,600	17	134,912	3	404,736	0.3	121,421	283,315	177,761	0.10	17,776	0.20	35,552	0.30	53,328
Gainsborough	302,720	17	51,462	3	154,387	0.3	46,316	108,071	99,688	0.10	9,969	0.20	19,938	0.30	29,906
Graham	156,800	17	26,656	3	79,968	0.3	23,990	55,978	55,376	0.10	5,538	0.20	11,075	0.30	16,613
Medora	73,600	16	11,776	3	35,328	0.3	10,598	24,730	40,438	0.10	4,044	0.20	8,088	0.30	12,131
Jackson	140,800	17	23,936	3	71,808	0.3	21,542	50,266	33,226	0.10	3,323	0.20	6,645	0.30	9,968
Stony	138,880	18	24,998	3	74,995	0.3	22,499	52,497	41,533	0.10	4,153	0.20	8,307	0.30	12,460
Plum	1,657,600	17	281,792	3	845,376	0.3	253,613	591,763	448,338	0.10	44,834	0.20	89,668	0.30	134,501
Elgin	89,600	14	12,544	3	37,632	0.3	11,290	26,342	59,421	0.10	5,942	0.20	11,884	0.30	17,826
							Total for entire area	**7,685,839**	**7,685,839**		**541,063**		**1,082,125**		**1,623,188**
							Total for Manitoba only	**1,404,386**	**4,338,878**		**206,367**		**412,733**		**619,100**

Note: * Data based PFRA 2008
Multiply acres by 0.4047 to convert to hectares, multiply feet by 0.3048 to convert to meters, multiply acre-feet by 1.23 to convert to cubic decameters.

Table 2
Summary of Computed Reductions in Peak Flows (cfs) and Percentages due to Wetland Restoration

1976 Type Flood	**Runoff Reduced by 10%**		**Runoff Reduced by 20%**		**Runoff Reduced by 30%**	
	CFS	%	CFS	%	CFS	%
Russell	151	7.5	254	12.6	454	22.5
Miniota	1600	9.7	3200	19.4	4800	29.1
Brandon	2025	9.8	4051	19.5	6076	29.3
Wawanesa	1543	6.1	3157	12.6	4806	19.1
Portage	3997	7.6	8103	15.5	12300	23.5
1995 Type Flood	**Runoff Reduced by 10%**		**Runoff Reduced by 20%**		**Runoff Reduced by 30%**	
	CFS	%	CFS	%	CFS	%
Russell	2256	18.3	6303	51.1	6452	52.3
Miniota	2182	11.3	4003	20.8	8241	42.8
Brandon	2126	11.1	4483	23.3	6213	32.3
Wawanesa	345	4.3	710	8.9	1100	13.9
Portage	2233	9.4	4631	19.5	6654	28.1
2011 Type Flood	**Runoff Reduced by 10%**		**Runoff Reduced by 20%**		**Runoff Reduced by 30%**	
	CFS	%	CFS	%	CFS	%
Russell	2137	13.8	4709	30.5	6562	42.5
Miniota	3848	10.9	8584	27.5	11489	36.9
Brandon	4095	11.5	8754	24.7	13054	36.8
Wawanesa	598	3.0	676	3.4	847	4.3
Portage	4302	8.1	9984	18.8	15084	28.5

Note: Divide cubic feet per second by 35.31 to convert to cubic meters per second.

ANNEX A

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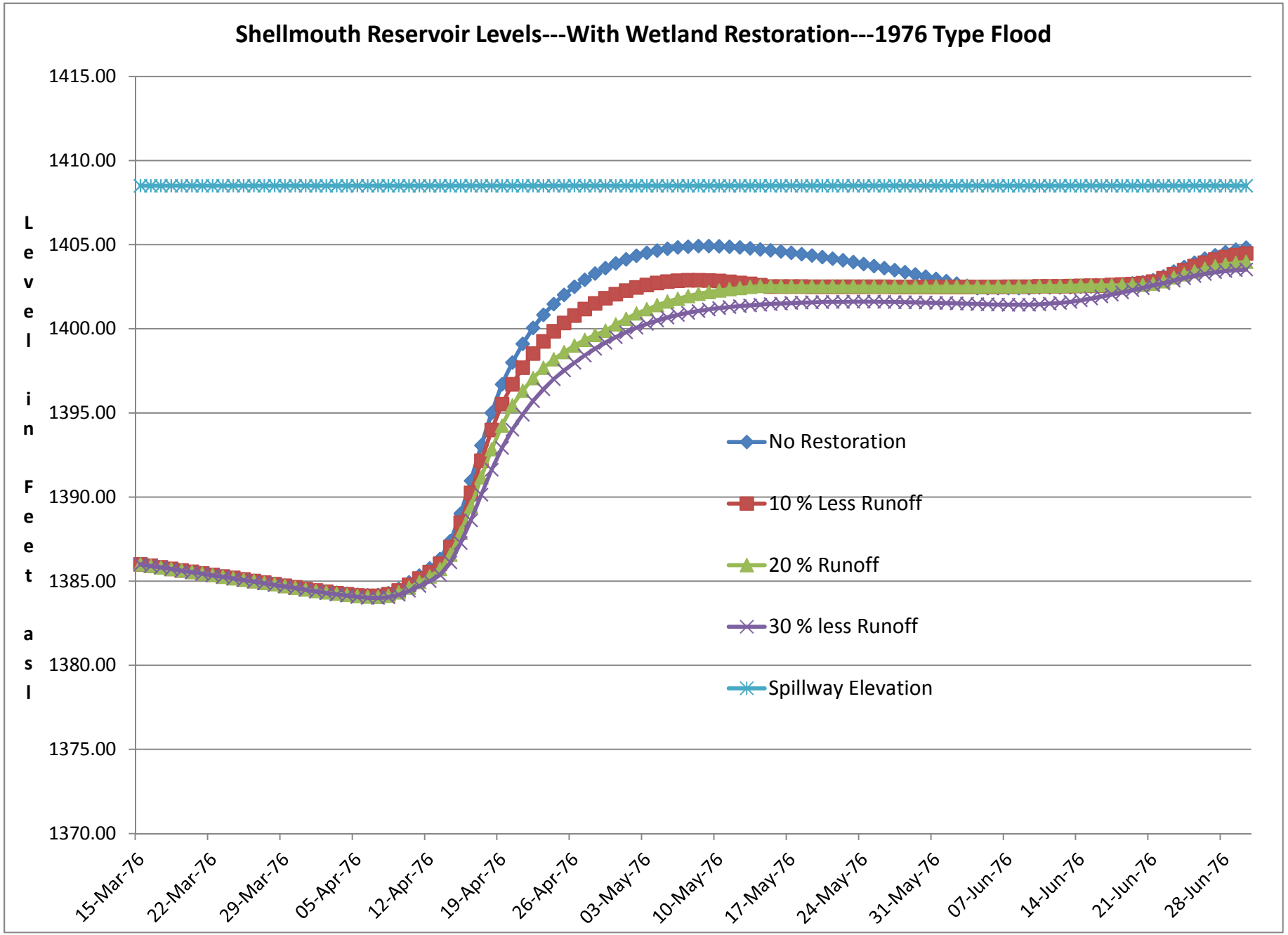
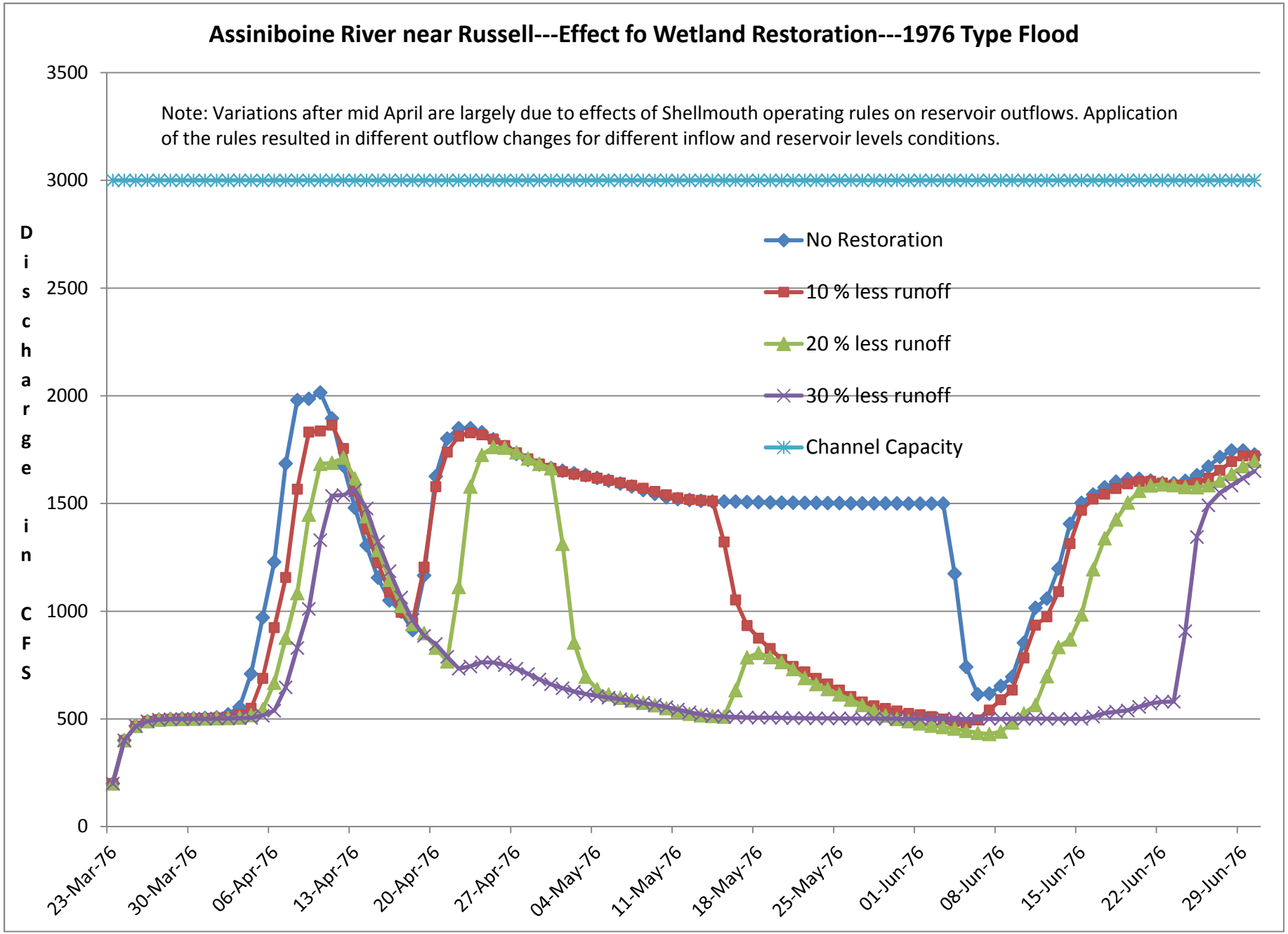
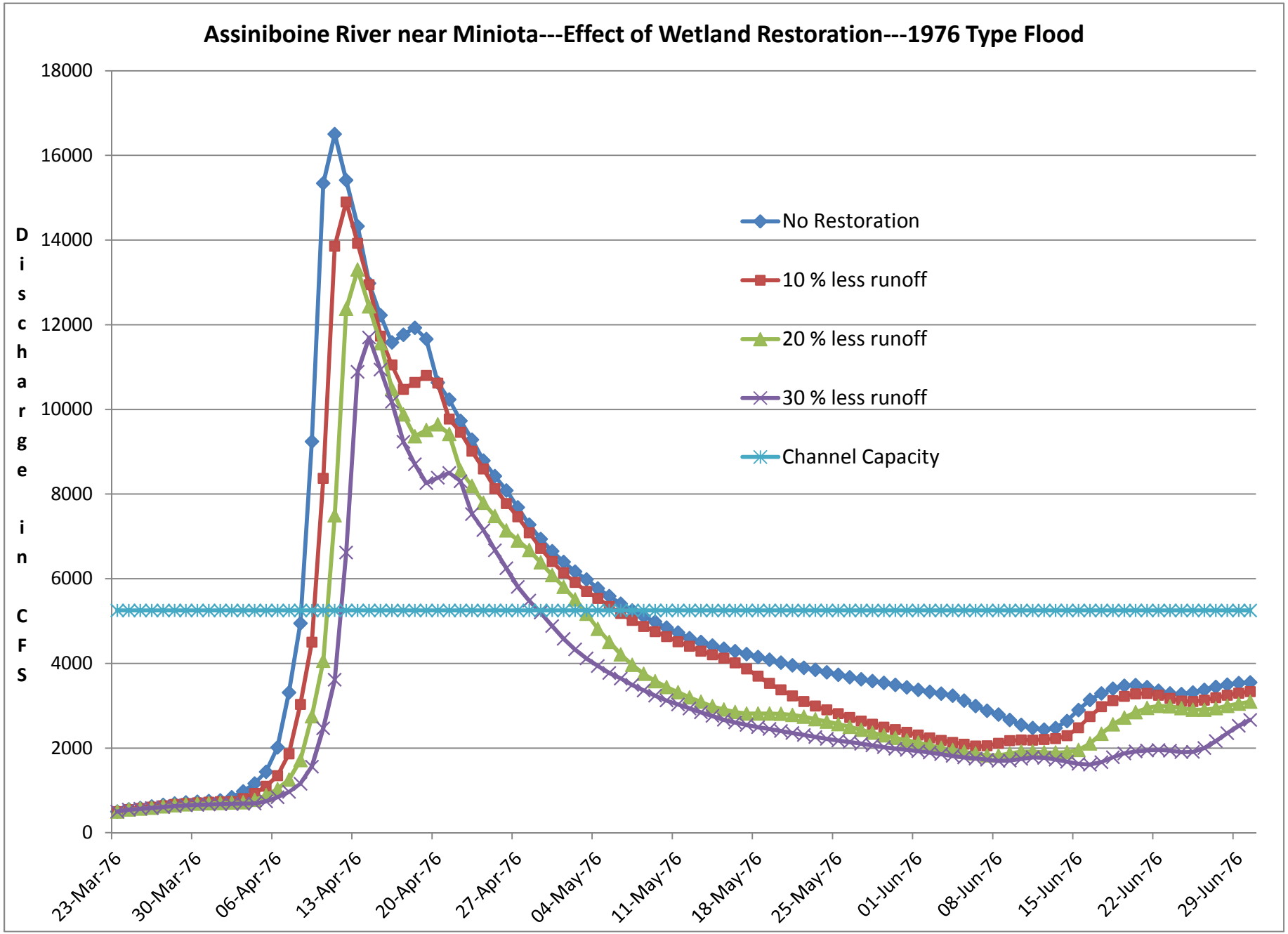
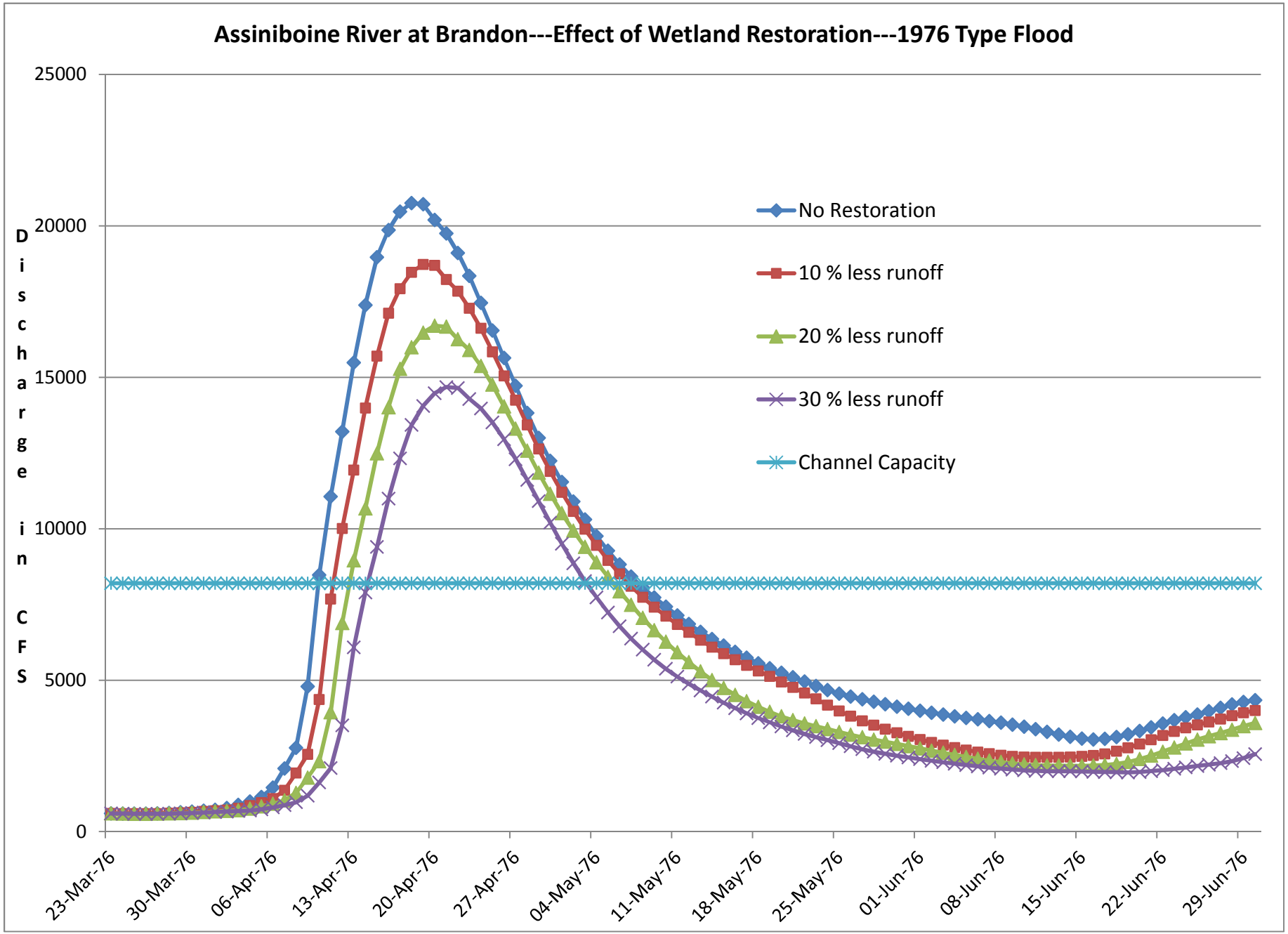
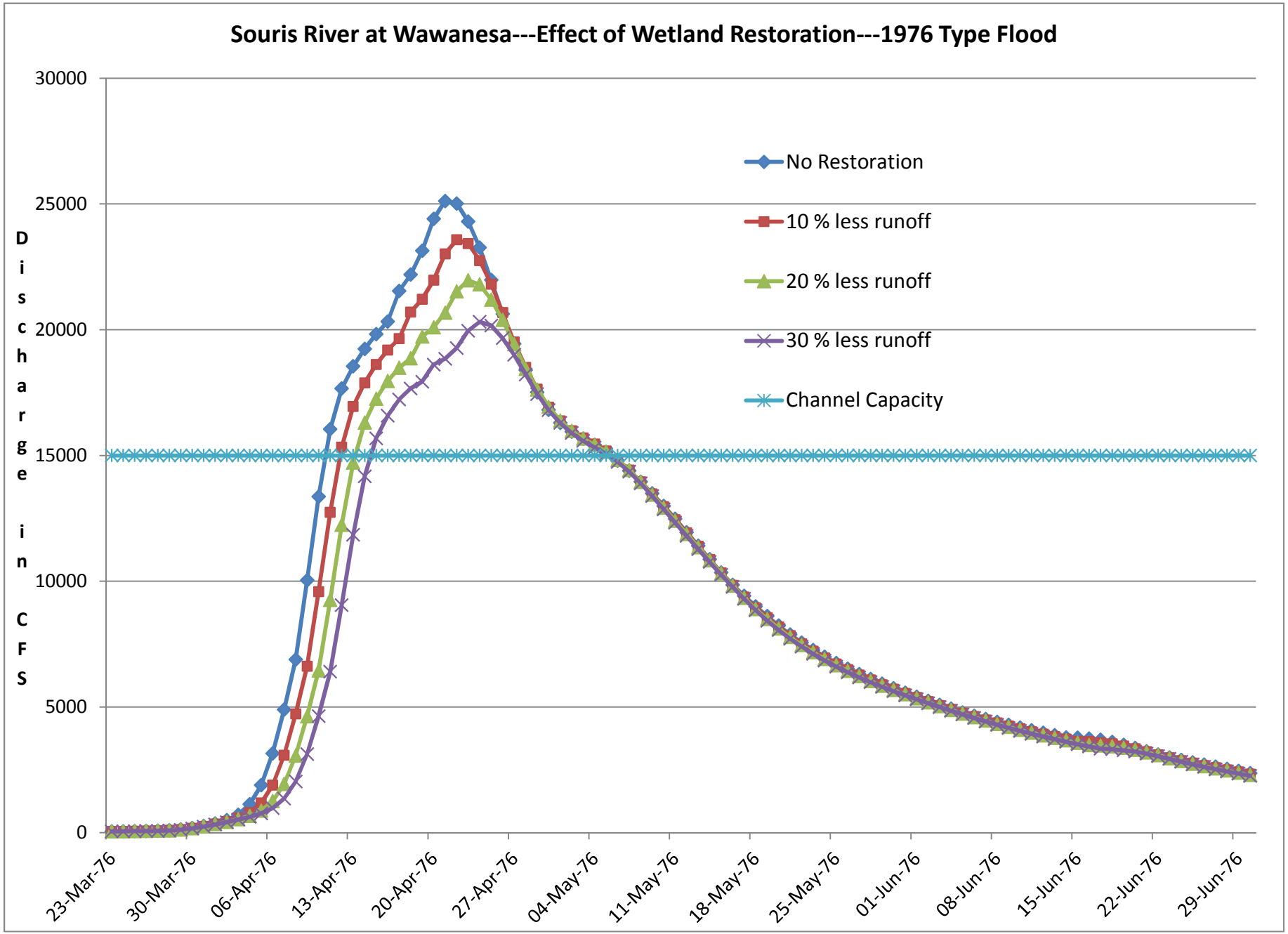
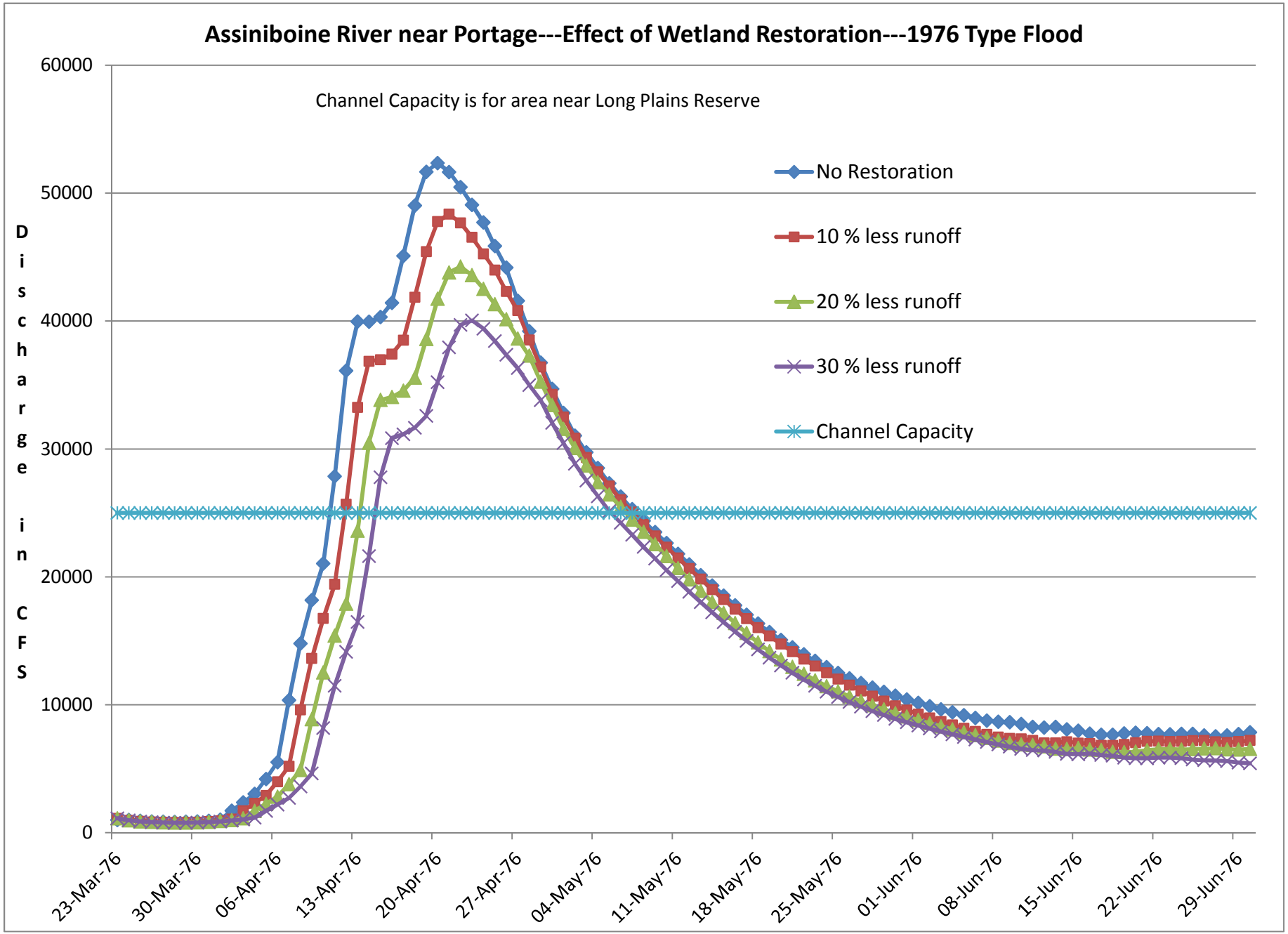

Shellmouth Reservoir Levels---With Wetland Restoration---1976 Type Flood
Level in Feet asl
1415.00
1410.00
1405.00
1400.00
1395.00
1390.00
1385.00
1380.00
1375.00
1370.00
15-Mar-76
22-Mar-76
29-Mar-76
05-Apr-76
12-Apr-76
19-Apr-76
26-Apr-76
03-May-76
10-May-76
17-May-76
24-May-76
31-May-76
07-Jun-76
14-Jun-76
21-Jun-76
28-Jun-76
No Restoration
10 % Less Runoff
20 % Runoff
30 % less Runoff
Spillway Elevation

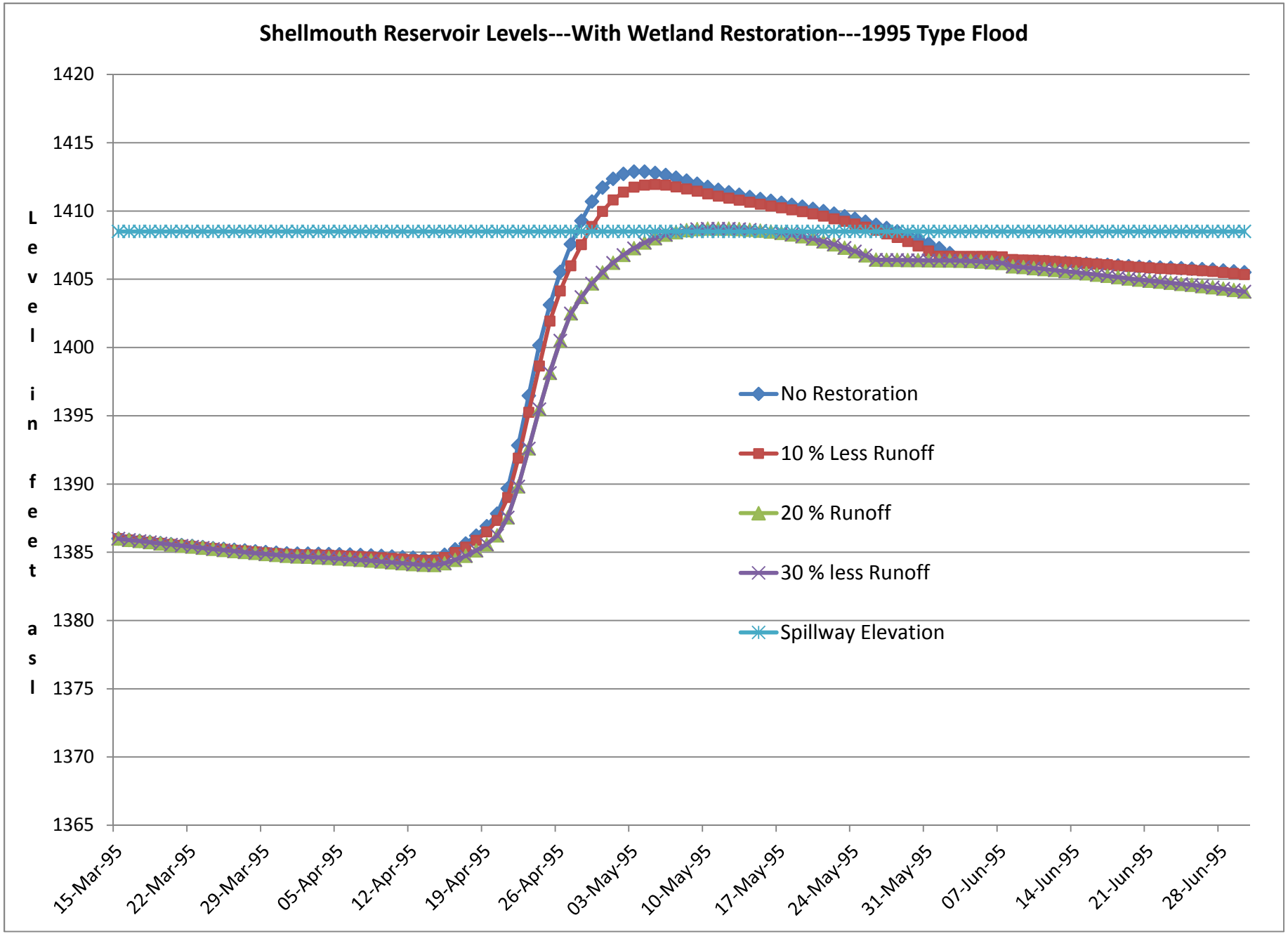

Assiniboine River near Russell---Effect fo Wetland Restoration---1976 Type Flood
Note: Variations after mid April are largely due to effects of Shellmouth operating rules on reservoir outflows. Application of the rules resulted in different outflow changes for different inflow and reservoir levels conditions.
Discharge in CFS
3500
3000
2500
2000
1500
1000
500
0
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity
23-Mar-76
30-Mar-76
06-Apr-76
13-Apr-76
20-Apr-76
27-Apr-76
04-May-76
11-May-76
18-May-76
25-May-76
01-Jun-76
08-Jun-76
15-Jun-76
22-Jun-76
29-Jun-76

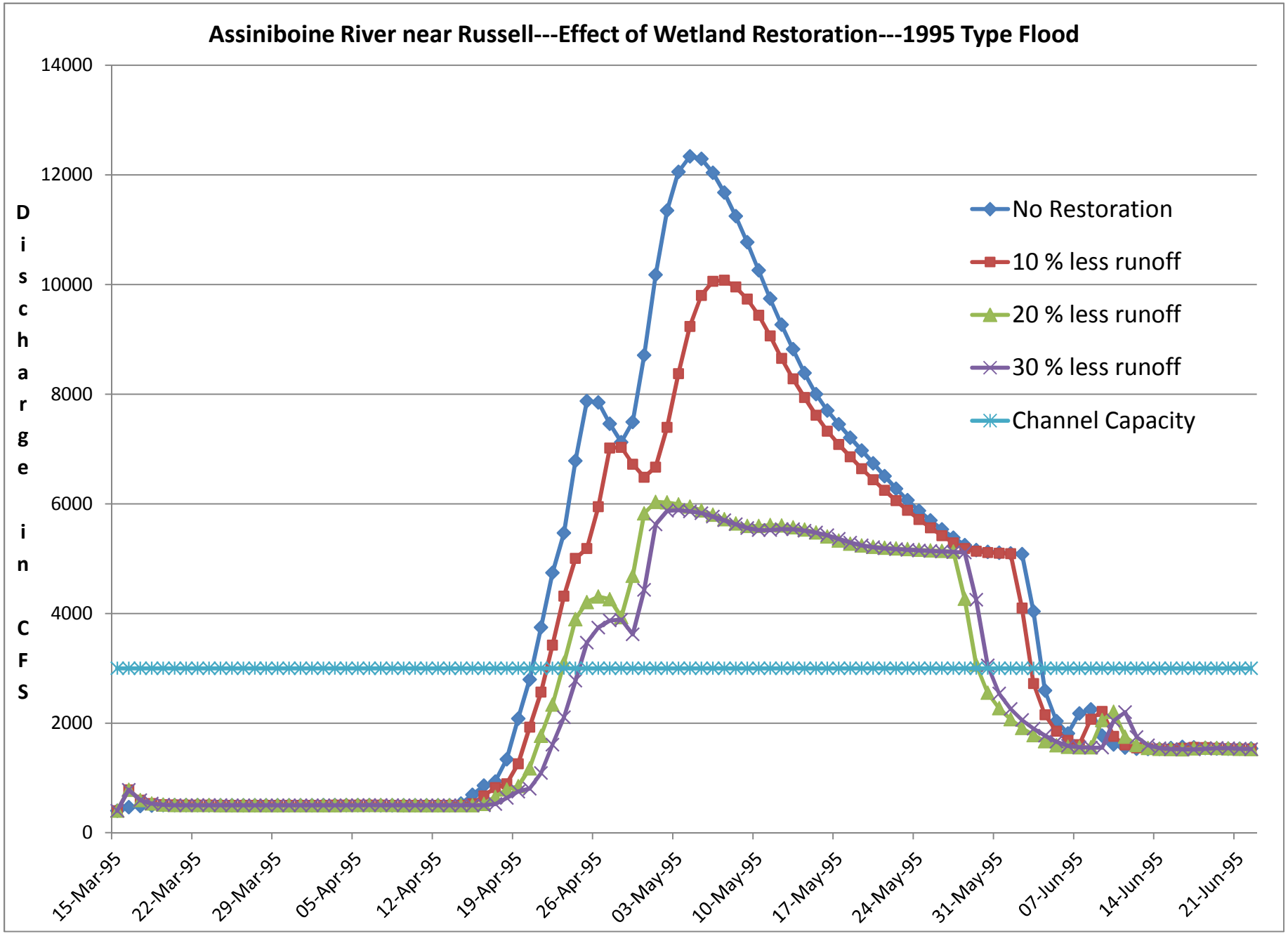

Assiniboine River near Miniota---Effect of Wetland Restoration---1976 Type Flood
Discharge in CFS
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity
23-Mar-76
30-Mar-76
06-Apr-76
13-Apr-76
20-Apr-76
27-Apr-76
04-May-76
11-May-76
18-May-76
25-May-76
01-Jun-76
08-Jun-76
15-Jun-76
22-Jun-76
29-Jun-76

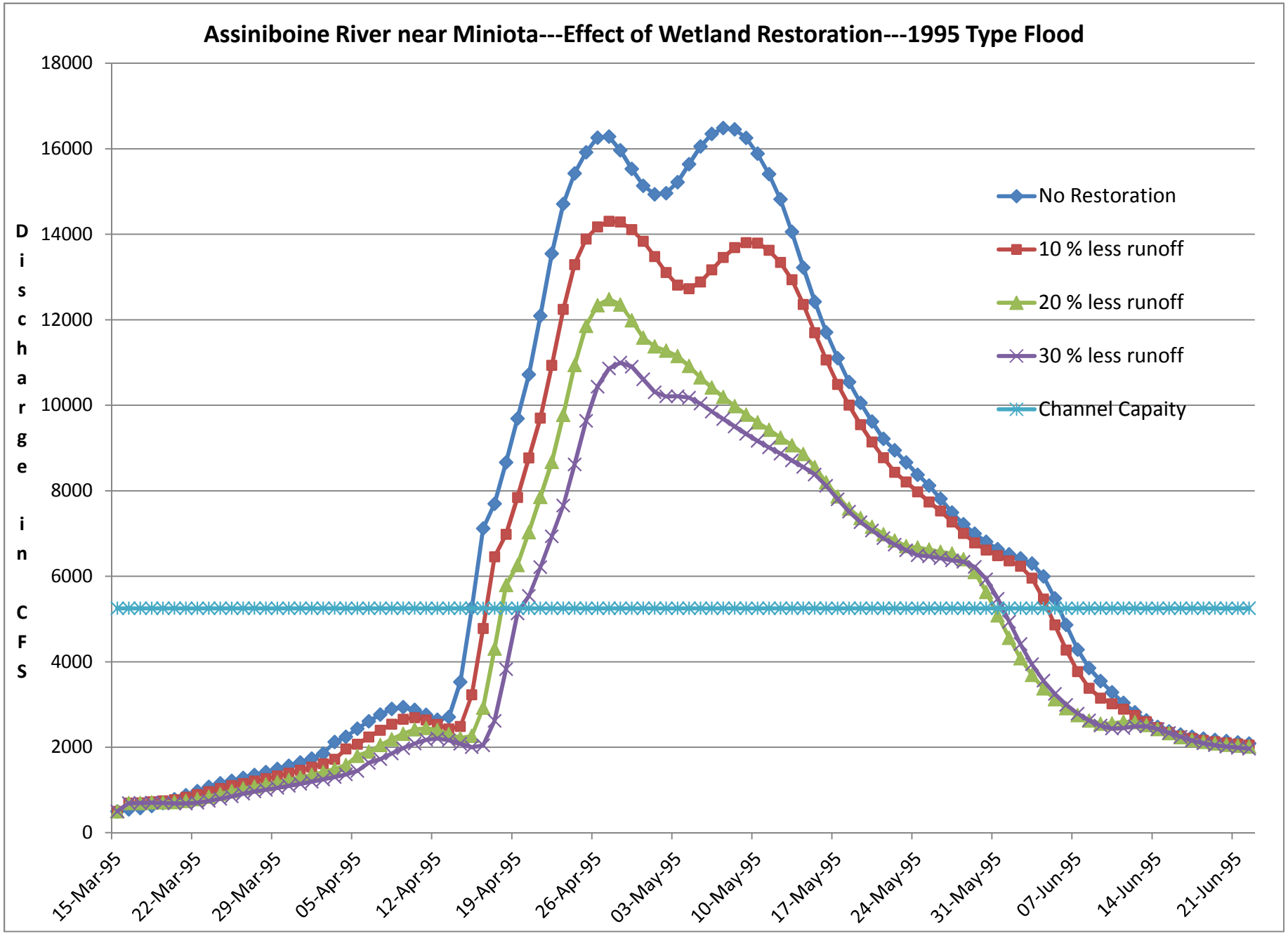

Assiniboine River at Brandon---Effect of Wetland Restoration---1976 Type Flood
Discharge in CFS
25000
20000
15000
10000
5000
0
23-Mar-76
30-Mar-76
06-Apr-76
13-Apr-76
20-Apr-76
27-Apr-76
04-May-76
11-May-76
18-May-76
25-May-76
01-Jun-76
08-Jun-76
15-Jun-76
22-Jun-76
29-Jun-76
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity

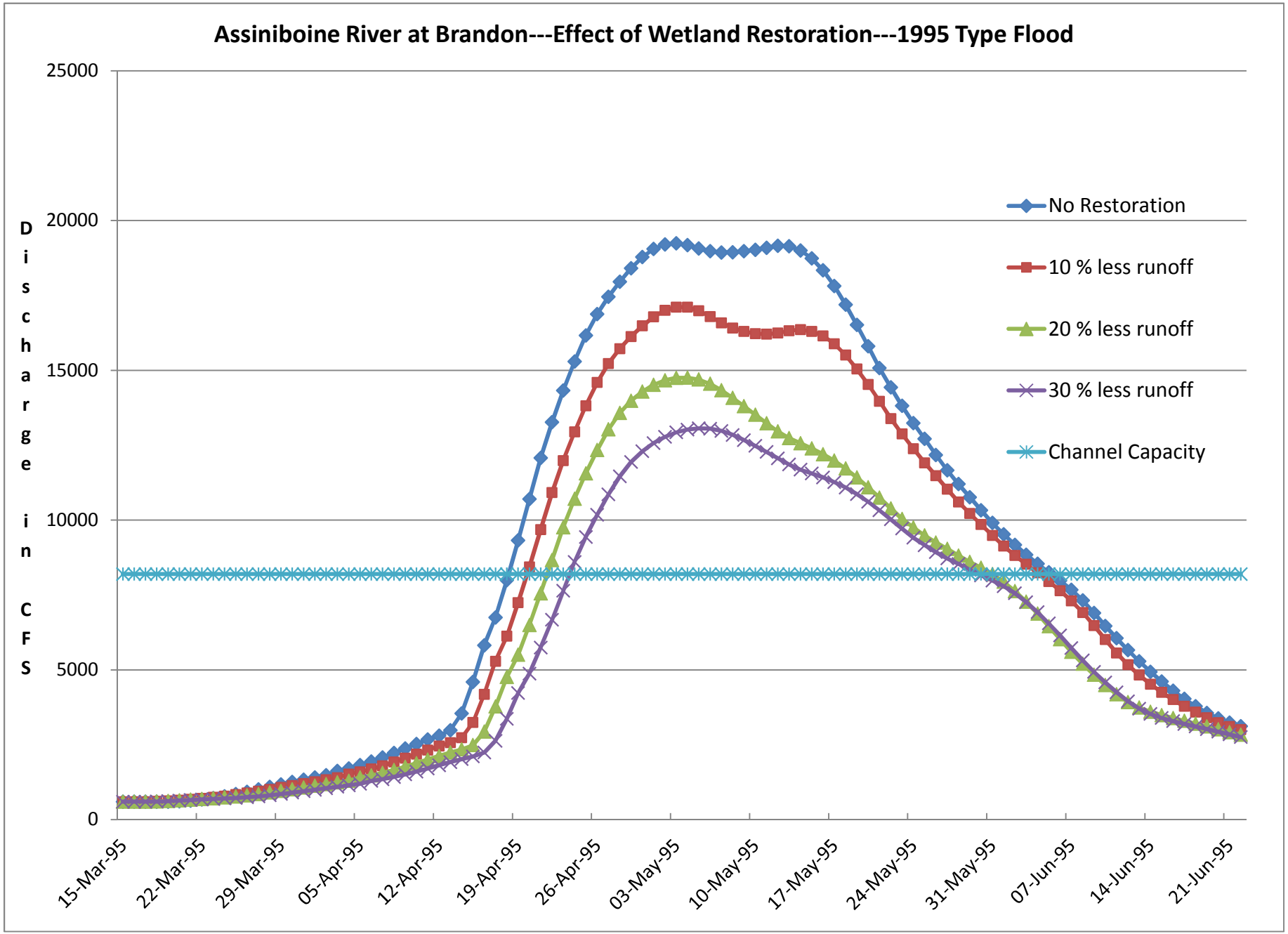

Souris River at Wawanesa---Effect of Wetland Restoration---1976 Type Flood
Discharge in CFS
30000
25000
20000
15000
10000
5000
0
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity
23-Mar-76
30-Mar-76
06-Apr-76
13-Apr-76
20-Apr-76
27-Apr-76
04-May-76
11-May-76
18-May-76
25-May-76
01-Jun-76
08-Jun-76
15-Jun-76
22-Jun-76
29-Jun-76

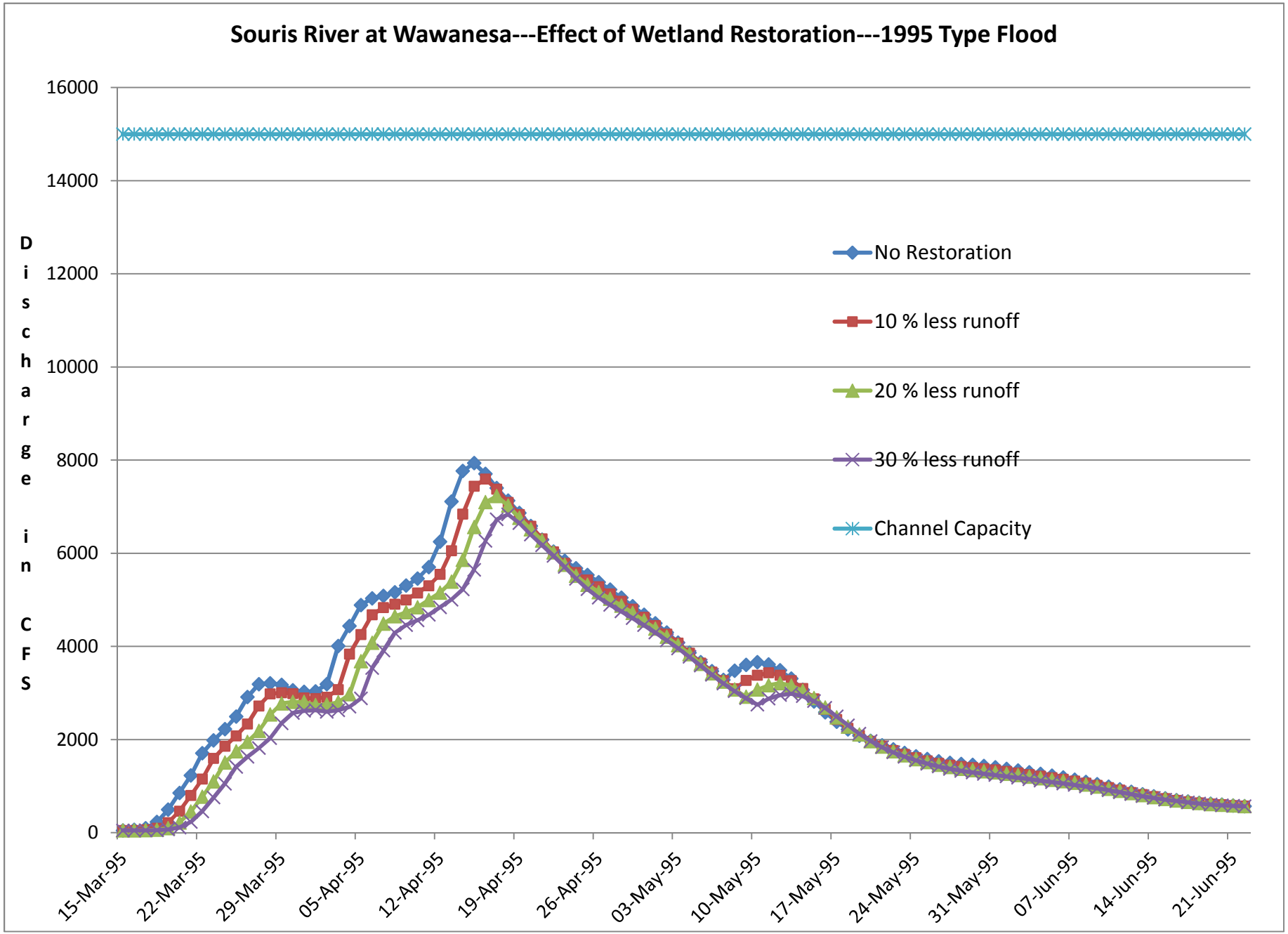

Assiniboine River near Portage---Effect of Wetland Restoration---1976 Type Flood
Channel Capacity is for area near Long Plains Reserve
Discharge in CFS
60000
50000
40000
30000
20000
10000
0
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity
23-Mar-76
30-Mar-76
06-Apr-76
13-Apr-76
20-Apr-76
27-Apr-76
04-May-76
11-May-76
18-May-76
25-May-76
01-Jun-76
08-Jun-76
15-Jun-76
22-Jun-76
29-Jun-76

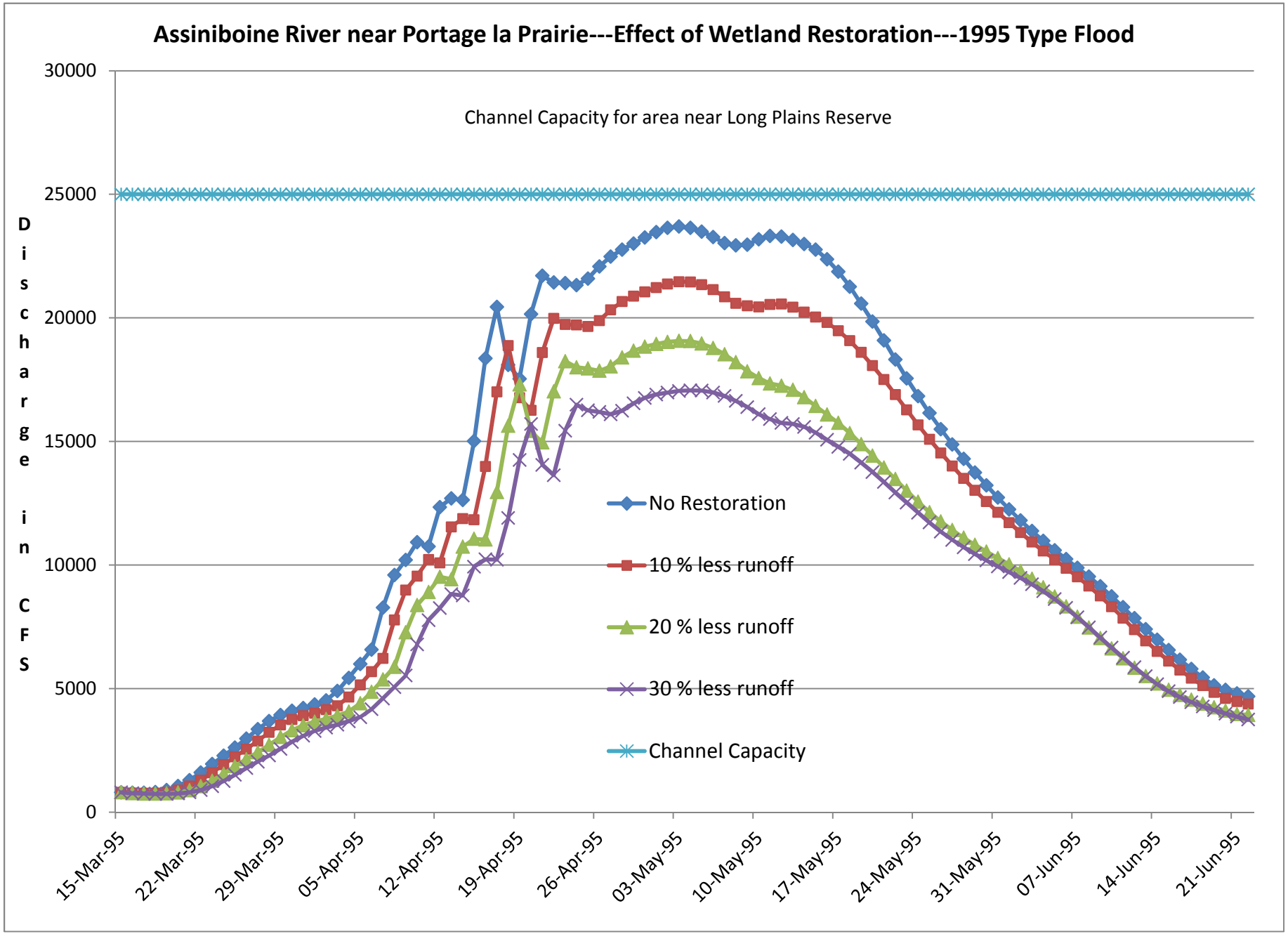

Shellmouth Reservoir Levels---With Wetland Restoration---1995 Type Flood
Level in feet asl
1420
1415
1410
1405
1400
1395
1390
1385
1380
1375
1370
1365
15-Mar-95
22-Mar-95
29-Mar-95
05-Apr-95
12-Apr-95
19-Apr-95
26-Apr-95
03-May-95
10-May-95
17-May-95
24-May-95
31-May-95
07-Jun-95
14-Jun-95
21-Jun-95
28-Jun-95
No Restoration
10 % Less Runoff
20 % Runoff
30 % less Runoff
Spillway Elevation


Assiniboine River near Russell---Effect of Wetland Restoration---1995 Type Flood
Discharge in CFS
14000
12000
10000
8000
6000
4000
2000
0
15-Mar-95
22-Mar-95
29-Mar-95
05-Apr-95
12-Apr-95
19-Apr-95
26-Apr-95
03-May-95
10-May-95
17-May-95
24-May-95
31-May-95
07-Jun-95
14-Jun-95
21-Jun-95
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity


Assiniboine River near Miniota---Effect of Wetland Restoration---1995 Type Flood
Discharge in CFS
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
15-Mar-95
22-Mar-95
29-Mar-95
05-Apr-95
12-Apr-95
19-Apr-95
26-Apr-95
03-May-95
10-May-95
17-May-95
24-May-95
31-May-95
07-Jun-95
14-Jun-95
21-Jun-95
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capaity


Assiniboine River at Brandon---Effect of Wetland Restoration---1995 Type Flood
Discharge in CFS
25000
20000
15000
10000
5000
0
15-Mar-95
22-Mar-95
29-Mar-95
05-Apr-95
12-Apr-95
19-Apr-95
26-Apr-95
03-May-95
10-May-95
17-May-95
24-May-95
31-May-95
07-Jun-95
14-Jun-95
21-Jun-95
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity


Souris River at Wawanesa---Effect of Wetland Restoration---1995 Type Flood
Discharge in CFS
16000
14000
12000
10000
8000
6000
4000
2000
0
15-Mar-95
22-Mar-95
29-Mar-95
05-Apr-95
12-Apr-95
19-Apr-95
26-Apr-95
03-May-95
10-May-95
17-May-95
24-May-95
31-May-95
07-Jun-95
14-Jun-95
21-Jun-95
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity


Assiniboine River near Portage la Prairie---Effect of Wetland Restoration---1995 Type Flood
Channel Capacity for area near Long Plains Reserve
Discharge in CFS
30000
25000
20000
15000
10000
5000
0
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity
15-Mar-95
22-Mar-95
29-Mar-95
05-Apr-95
12-Apr-95
19-Apr-95
26-Apr-95
03-May-95
10-May-95
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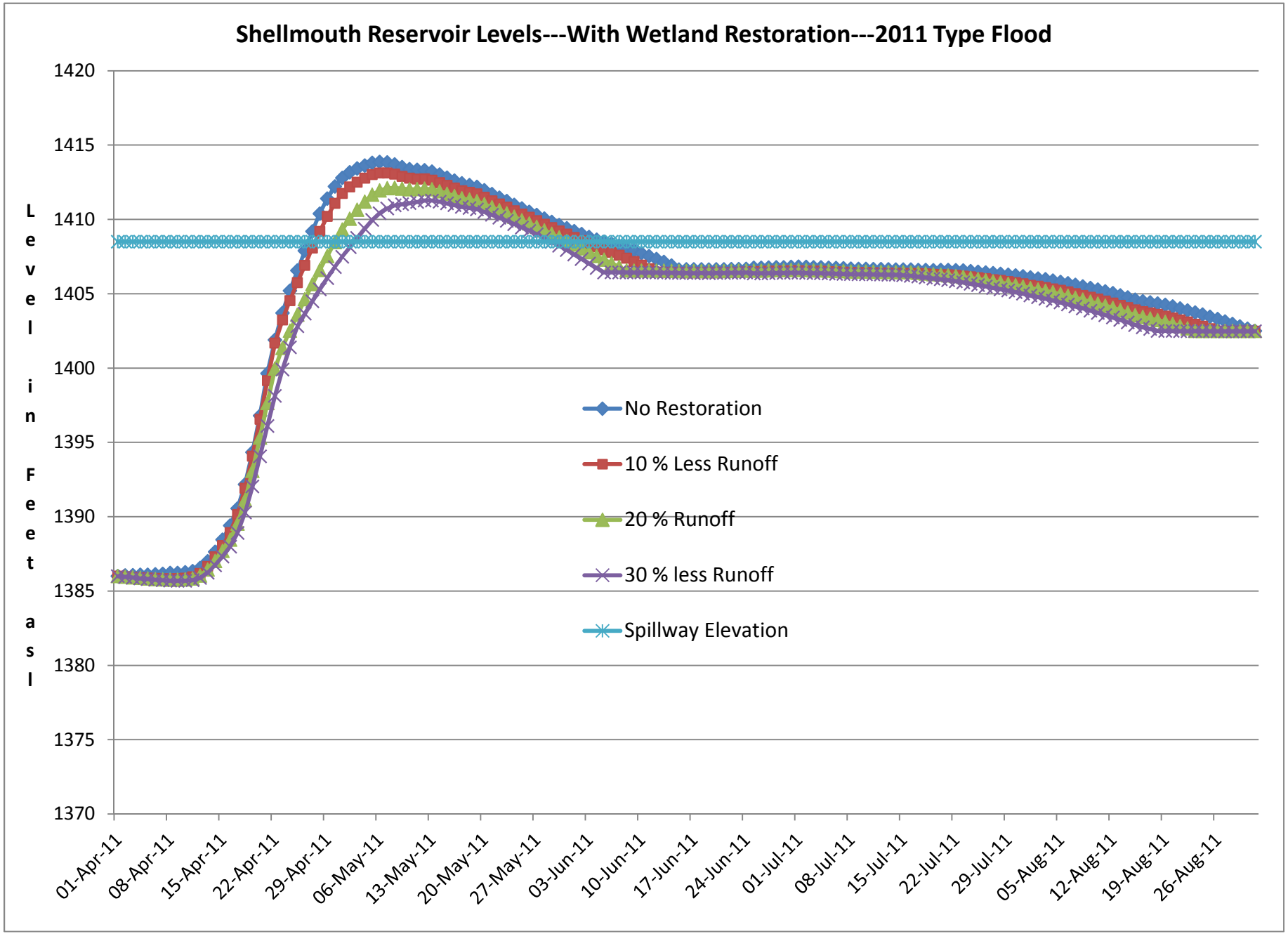
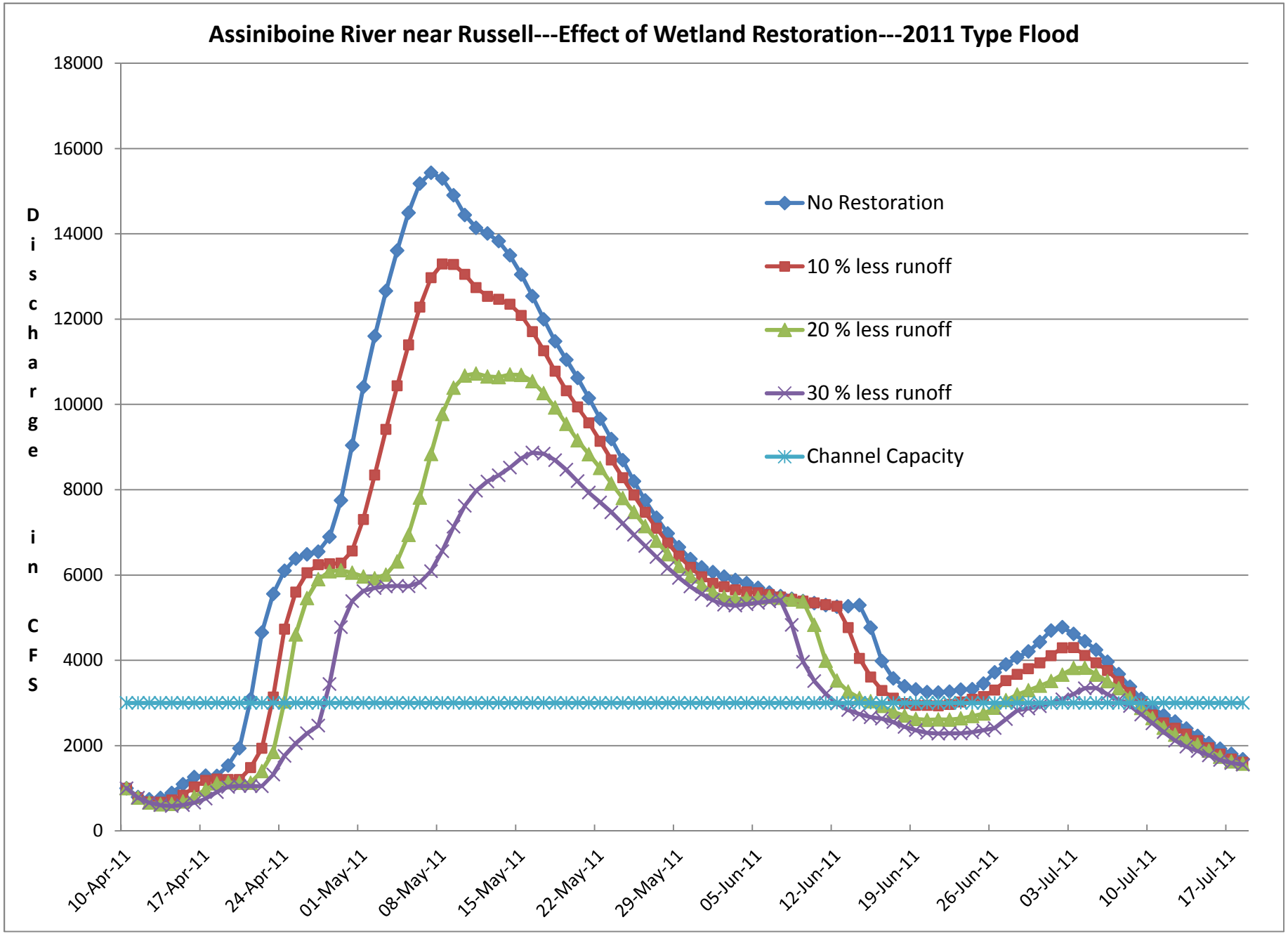
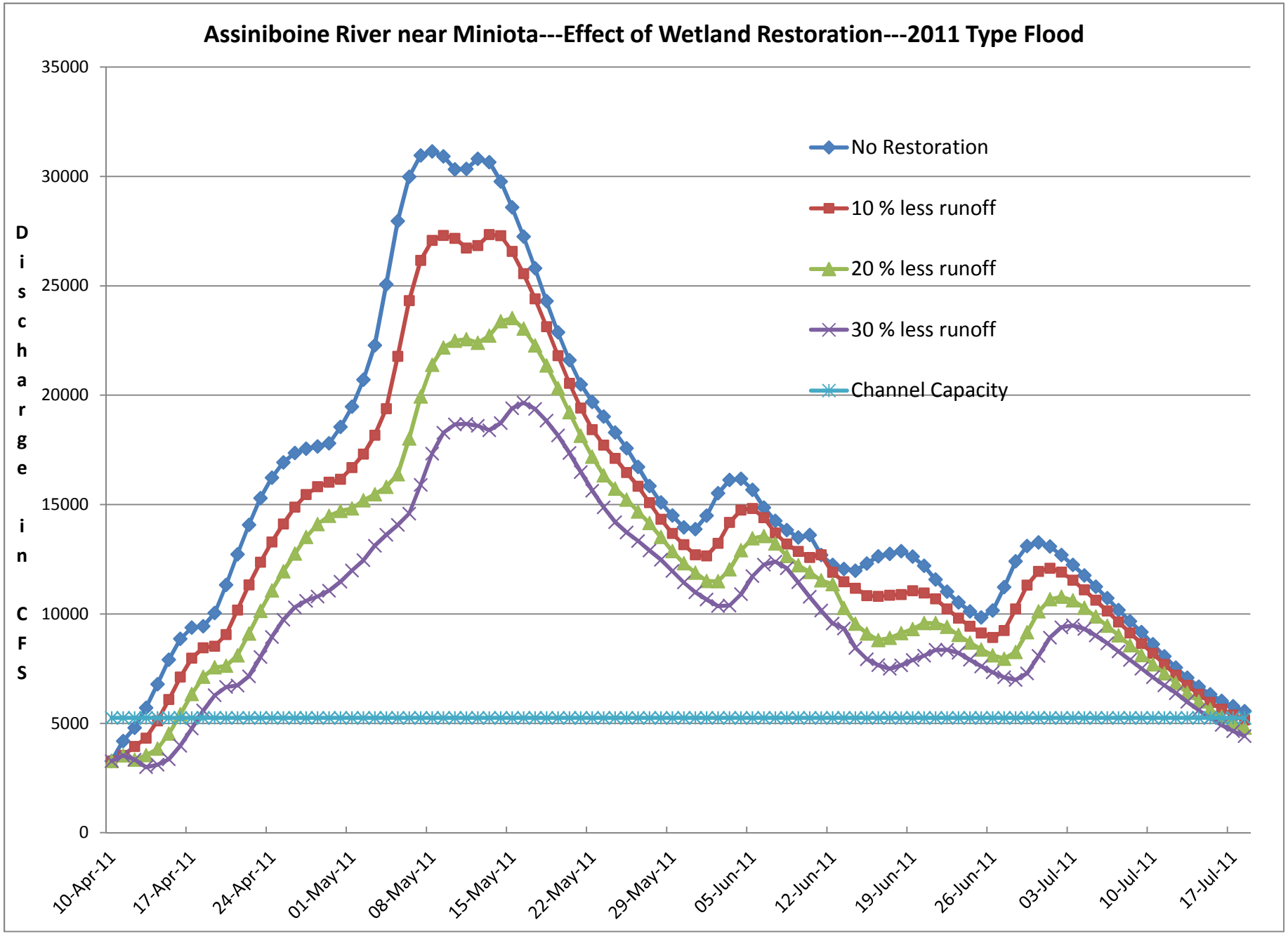
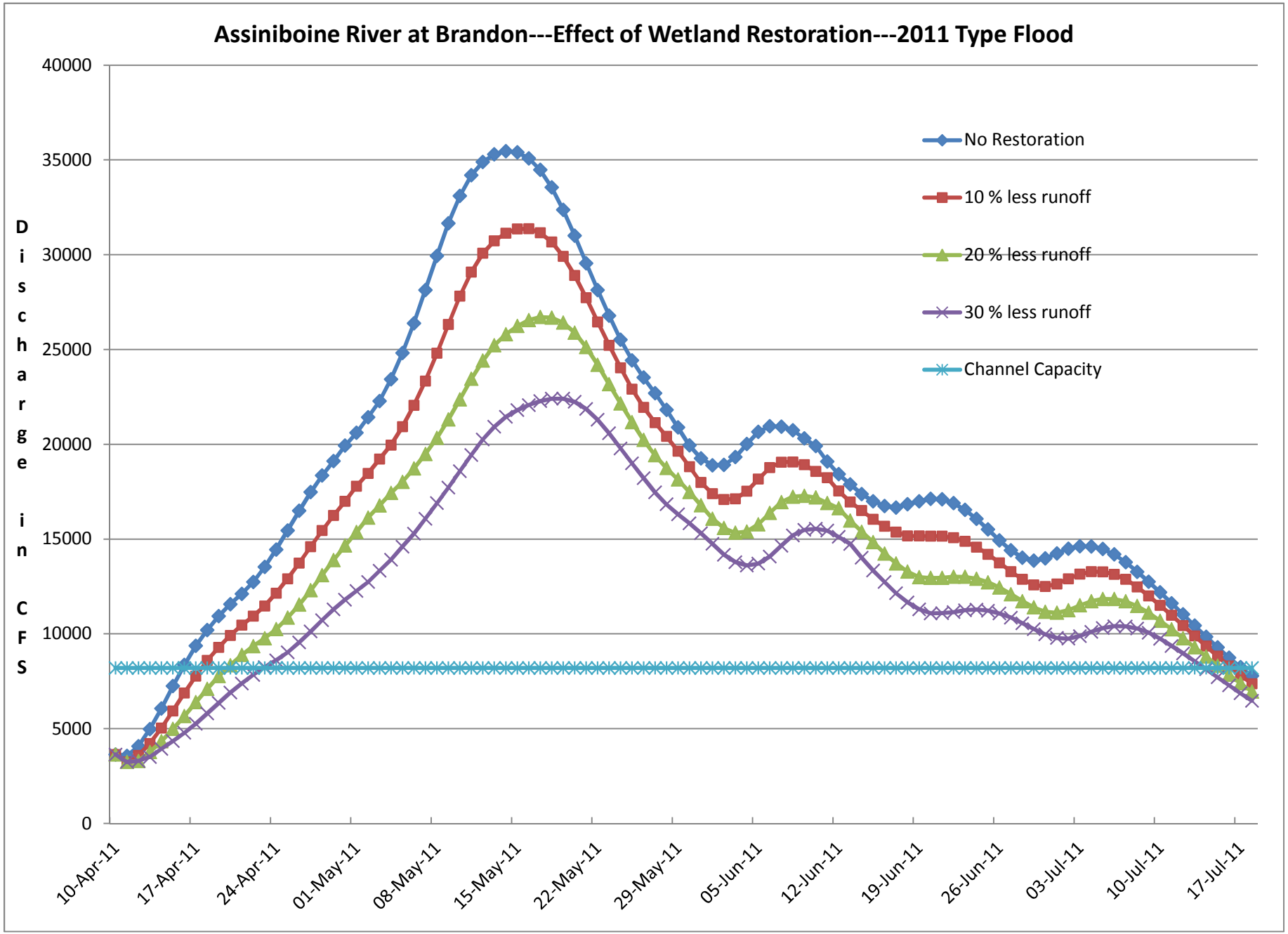
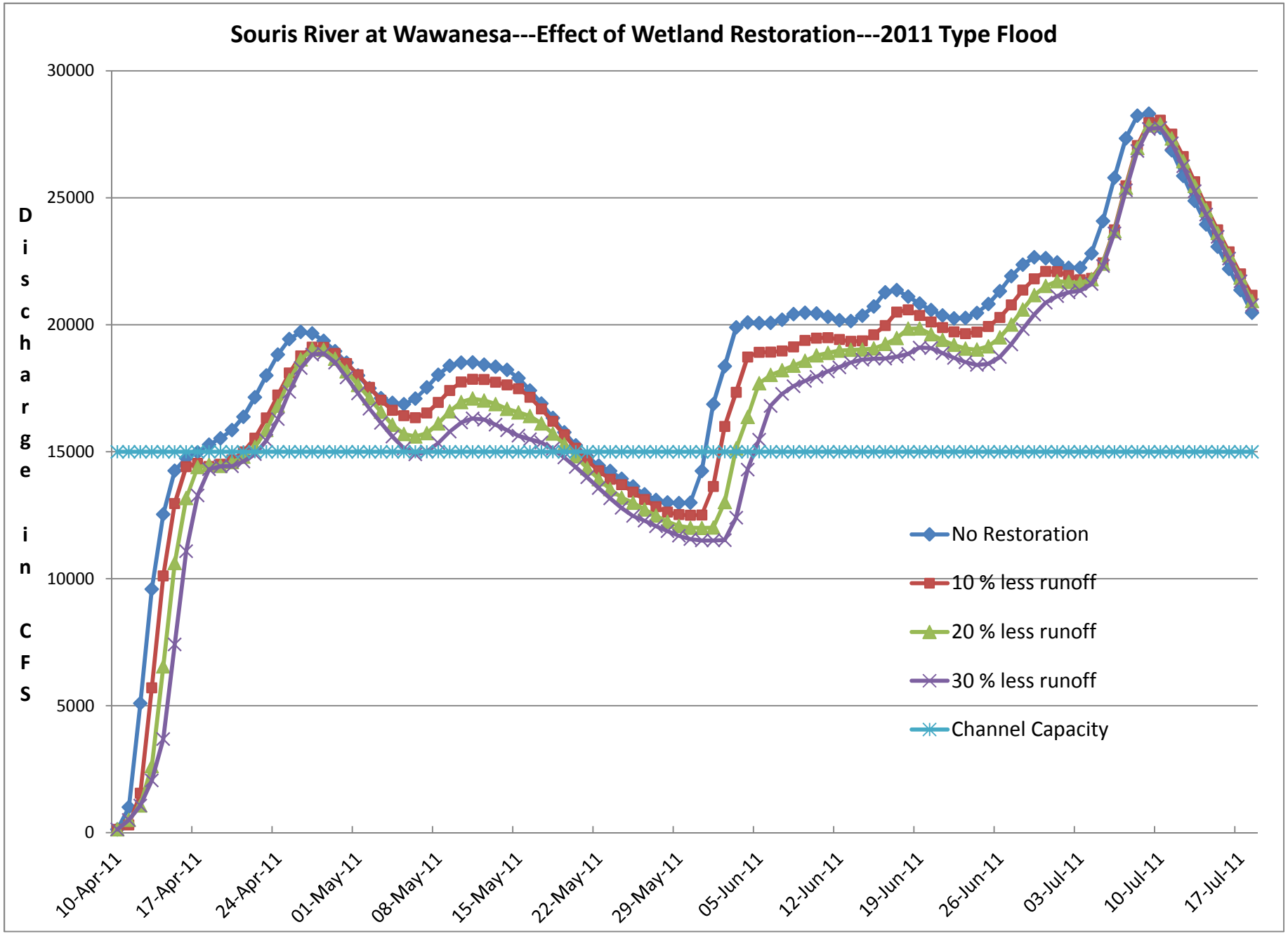
14-Jun-95
21-Jun-95

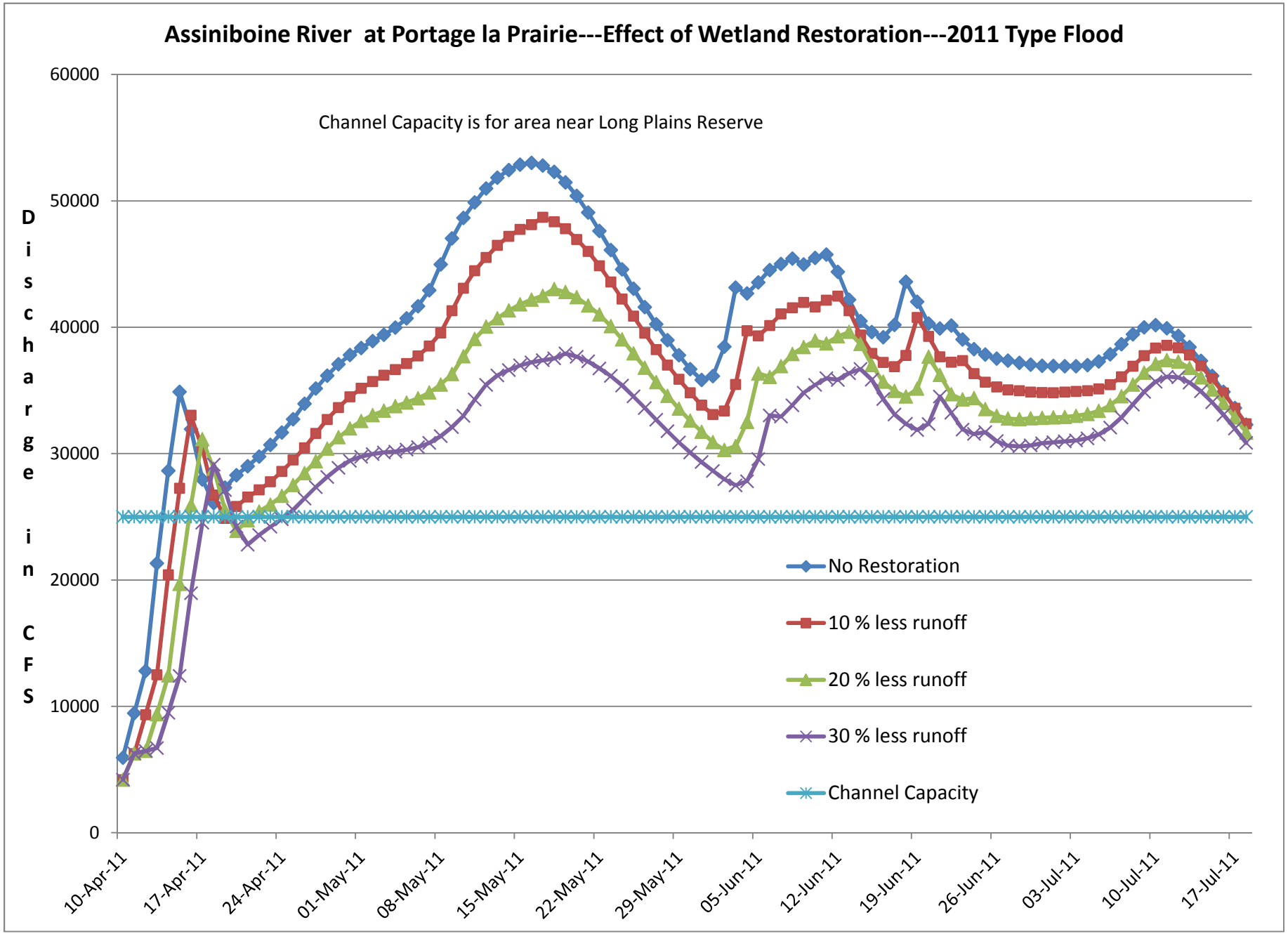

Shellmouth Reservoir Levels---With Wetland Restoration---2011 Type Flood
Level in Feet asl
1420
1415
1410
1405
1400
1395
1390
1385
1380
1375
1370
01-Apr-11
08-Apr-11
15-Apr-11
22-Apr-11
29-Apr-11
06-May-11
13-May-11
20-May-11
27-May-11
03-Jun-11
10-Jun-11
17-Jun-11
24-Jun-11
01-Jul-11
08-Jul-11
15-Jul-11
22-Jul-11
29-Jul-11
05-Aug-11
12-Aug-11
19-Aug-11
26-Aug-11
No Restoration
10 % Less Runoff
20 % Runoff
30 % less Runoff
Spillway Elevation


Assiniboine River near Russell---Effect of Wetland Restoration---2011 Type Flood
Discharge in CFS
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity
10-Apr-11
17-Apr-11
24-Apr-11
01-May-11
08-May-11
15-May-11
22-May-11
29-May-11
05-Jun-11
12-Jun-11
19-Jun-11
26-Jun-11
03-Jul-11
10-Jul-11
17-Jul-11


Assiniboine River near Miniota---Effect of Wetland Restoration---2011 Type Flood
Discharge in CFS
35000
30000
25000
20000
15000
10000
5000
0
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity
10-Apr-11
17-Apr-11
24-Apr-11
01-May-11
08-May-11
15-May-11
22-May-11
29-May-11
05-Jun-11
12-Jun-11
19-Jun-11
26-Jun-11
03-Jul-11
10-Jul-11
17-Jul-11


Assiniboine River at Brandon---Effect of Wetland Restoration---2011 Type Flood
Discharge in CFS
40000
35000
30000
25000
20000
15000
10000
5000
0
10-Apr-11
17-Apr-11
24-Apr-11
01-May-11
08-May-11
15-May-11
22-May-11
29-May-11
05-Jun-11
12-Jun-11
19-Jun-11
26-Jun-11
03-Jul-11
10-Jul-11
17-Jul-11
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity


Souris River at Wawanesa---Effect of Wetland Restoration---2011 Type Flood
Discharge in CFS
30000
25000
20000
15000
10000
5000
0
10-Apr-11
17-Apr-11
24-Apr-11
01-May-11
08-May-11
15-May-11
22-May-11
29-May-11
05-Jun-11
12-Jun-11
19-Jun-11
26-Jun-11
03-Jul-11
10-Jul-11
17-Jul-11
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity


Assiniboine River at Portage la Prairie---Effect of Wetland Restoration---2011 Type Flood
Channel Capacity is for area near Long Plains Reserve
Discharge in CFS
60000
50000
40000
30000
20000
10000
0
10-Apr-11
17-Apr-11
24-Apr-11
01-May-11
08-May-11
15-May-11
22-May-11
29-May-11
05-Jun-11
12-Jun-11
19-Jun-11
26-Jun-11
03-Jul-11
10-Jul-11
17-Jul-11
No Restoration
10 % less runoff
20 % less runoff
30 % less runoff
Channel Capacity

APPENDIX N

EFFECTS OF SASKATCHEWAN DRAINAGE PRACTICES

APPENDIX N

EFFECTS OF SASKATCHEWAN DRAINAGE PRACTICES

1.0 SCOPE OF WORK

The scope of this work was to undertake a simple analysis of several hydrometric gauging stations in an attempt to capture the influence of increased Saskatchewan drainage practices. Years before and after the best estimate of the change in drainage practices were analyzed. The analysis was limited to examining impacts on annual peak flows and runoff volumes. The analysis of streamflow data for trends to capture the influence of artificial manmade drainage is complicated because of the non-stationary impact of climate and land management practices. The analysis was limited a literature review, a direct station comparison, mass curve and trend analysis. This analysis and findings are documented herein.

2.0 STUDY AREA

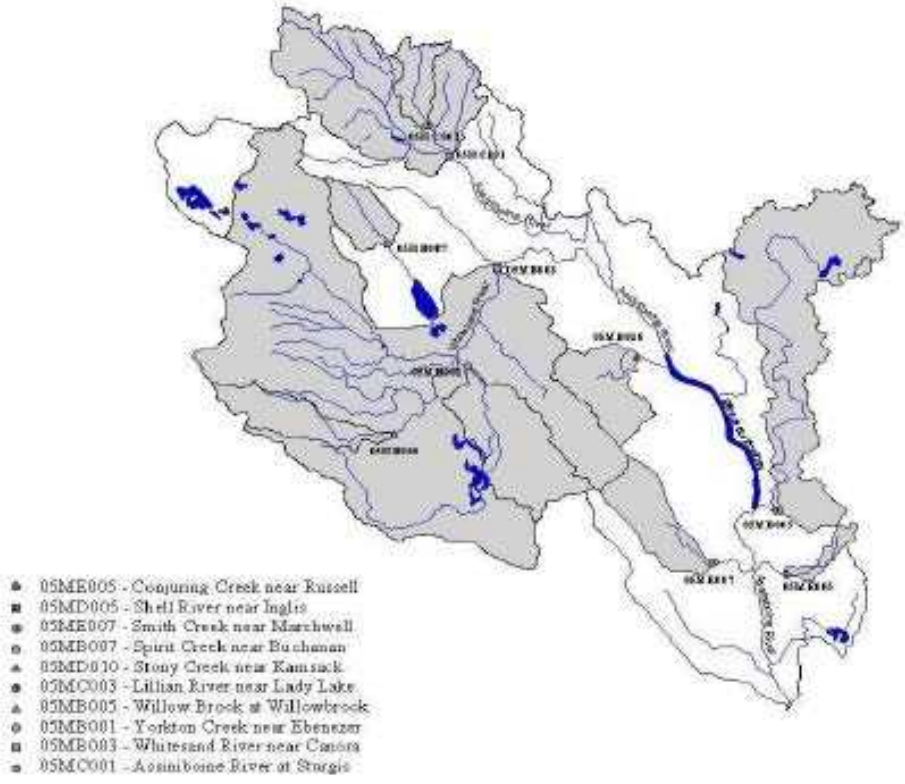
The Upper Assiniboine River Basin was selected for the study. It is one of the most controversial areas regarding the issue of interprovincial drainage and has been a long standing and very contentious issue between Manitoba and Saskatchewan. The issue of interprovincial drainage associated with agricultural development and practices in the Upper Assiniboine River Basin (UARB) following the 1995 Flood was the main issue leading to the Manitoba, Saskatchewan, and Canada comprehensive study of water management issues in the UARB (Manitoba et al., 2000).

The reason for re-examining the Upper Assiniboine River Basin was that there are now an additional 15 years of record streamflow data and the basin has experienced recently extreme floods in 2011 and 2014. The long term hydrometric data should capture the influences of Saskatchewan changing drainage practices. The data includes a relatively wet period in the mid-1950s. This allows a comparison of pre and post drainage conditions during wet climatic conditions. Some of hydrometric gauge stations examined in this study are indicated in Table 1. A map showing the location of the stations is on Figure 1.

TABLE 1
HYDROMETRIC GAUGING STATION INFORMATION

Station Number	Station Name	Period of Record	Gross Drainage Area in km^2	Gross Drainage Area in km^3
05MD004	Assiniboine River near Kamsack	1944 - 2014	13055	3842
05MD005	Shell River near Inglis	1948 - 2014	1968	938
05MB003	Whitesand River near Canora	1943 -2014	8828	1477
05MC001	Assiniboine River near Sturgis	1944 -2014	1939	1166
05JM015	Cutarm Creek near Spy Hill	1941 -2014	849	236
05ME007	Smith Creek near Marchwell	1974 - 2014	445	86
05ME005	Conjuring Creek near Russell	1959 - 2014	70	53
05ME001	Assiniboine River near Russell	1912 - 2014	19352	6707
05MB001	Yorkton Creek near Ebenezer	1941 - 2013	2317	410
05MD010	Stony Creek near Kamsack	1971 - 2013	445	90

FIGURE 1
LOCATION MAP OF HYDROMETRIC STATIONS
(SOURCE URBS DRAINAGE AND FLOOD CONTROL COMMITTEE REPORT)



3.0 LITERATURE REVIEW

A literature review was undertaken to assist understanding the knowledge of the impact of drainage practices on runoff response and it application to the Assiniboine River Basin. Its purpose was to identity approaches that could be applied in this study or modified and the relevancy of the finding to the Assiniboine River Basin. Three relevant studies were identified with are cited as follows,

3.1 UPPER ASSINIBOINE RIVER BASIN STUDY (Manitoba Conservation Et Al., 2000a,)

During the Upper Assiniboine River Basin study (2000), an analysis of streamflow data was undertaken to determine if land use practices may have a significant effect on the hydrologic response of the watershed. The analysis included: a comparison of analogue years, mass curve analysis of peak flows and runoff volumes, the derivation of mass curves, analysis of stationarity, analysis of trend, sub-watershed travel times, and, regression analysis (split sample). The study noted that "*The hydrologic analysis of streamflow records in the Basin indicates that runoff volumes have been increasing. The Committee concluded that changes in Basin land cover, including loss of wooded area, loss of wetlands, improved drainage network, and increased road development, have resulted in increased runoff volumes over what would have occurred naturally. However, it is not possible to determine the hydrologic effect of individual land cover changes on runoff by examining only recorded streamflow data*."

3.2 ON THE CHANGES IN LONG TERM STREAMFLOW REGIMES IN THE PRAIRIES (Ehsanzadehab et al., 2014)

This study investigated the integrated impacts of precipitation change/variability and landscape changes, specifically wetland drainage practices, on streamflow regimes in wetland dominated landscapes in the Assiniboine and Saskatchewan River Basins of the North American Prairies. Precipitation and streamflow metrics were examined for gradual (trend type) and abrupt (shift type) changes using the modified Mann-Kendall trend test and a Bayesian change point detection methodology. Results of statistical analyses indicate that precipitation metrics did not experience statistically significant increasing or decreasing changes and there was no statistical evidence of streamflow regime change over the study area except for one of the smaller watersheds. The absence of widespread streamflow and precipitation change suggests that

wetland drainage did not lead to detectable changes in streamflow metrics over most of the Canadian portion of the Prairies from 1967 to 2007.

3.3 THE IMPACT OF CLIMATIC VARIABILITY AND CHANGE IN THE HYDROCLIMATOLOGY OF LAKE WINNIPEG WATERSHED
(Ehsanzadehab Et Al, 2011)

The total annual streamflow, precipitation, runoff ratio and daily maximum streamflow in the two major tributaries of the Assiniboine River and Red River were analysed for a range of nonstationary behaviours. Each of these rivers has been gauged for more than 90 years. The methods used included a nonparametric Mann–Kendall test modified to account for diverse memory properties (i.e. short term versus long term) and a Bayesian change point detection model to identify possible segments of time series with inconsistent nonstationary behaviour.

The authors found that “There is no evidence of trend type non-stationarity behaviour in the hydroclimatology of the Assiniboine River watershed. Annual runoff in this watershed, however, is characterised with abrupt changes echoing a sequence of wet and dry episodes each lasting over a decade. These wet and dry epochs are not detectable in precipitation inputs, indicating again that precipitation–runoff relationship in the studied watersheds is highly nonlinear. The non-proportionality of watershed response behaviour to changes in climate inputs suggests that major or even minor precipitation changes in the North America Prairie landscapes may trigger certain thresholds, which govern the hill-slope runoff and wetland fill-spill processes and lead to significant changes in the areas contributing to runoff at the outlet of the watershed. The sudden expansion (contraction) of contributing areas, in turn, superimposes abrupt changes on the significant/insignificant trends in runoff caused by precipitation anomalies. For example, the depression-dominated landscapes of the North America may experience a hydrologic drought although a climatological drought is not in effect. It is concluded that streamflows of the North American glaciated Prairies are particularly vulnerable to minor climate variability and change. The significance of detected trends under a Long-term persistence (LTP) hypothesis in the Red River watershed suggests that the observed non-stationarity is not solely due to natural climate variability and it may be attributed, in part, to climate change. The lack of a long-term trend of the annual flow of the Assiniboine River indicates that long-term cumulative changes in the watershed, including wetland drainage and changes of farming practices have not had a net detectable influence on the flow. This result is intriguing because modelling studies have

suggested that wetland drainage in particular leads to increased runoff. The reasons for the absence of identifiable effects are not clear and can probably be best established through more detailed studies of smaller sub-watersheds".

The most commonly used statistical test to investigate the presence of trends in hydrometeorological time series is the nonparametric Mann–Kendall (MK) test (Mann, 1945; Kendall, 1975). The nonparametric statistical tests, with parametric tests, are thought to be more suitable for nonnormally distributed and censored data, which are frequently encountered in hydrometeorological time series. However, as the scope of this study was to undertake a simple analysis of available flow data in an attempt to capture the influence of increased Saskatchewan drainage practices approaches similar to those in the UARB Study applied.

The finding of the UARB study and Ehsanzadehab appear contradictory. The URAB Study found that runoff volumes have been increasing in the Upper Assiniboine River Basin. Whereas, Ehsanzadehab's research indicates the lack of a long-term trend of the annual flow of the Assiniboine River indicates that long-term cumulative changes in the watershed, including wetland drainage and changes of farming practices, have not had a net detectable influence on the flow. This could be because of the size of the two areas examined. It is likely, the response of drainage practice may be more pronounced and detectable on small watersheds that larger watersheds. Impacts may diminish with increasing basin size because of other downstream influences.

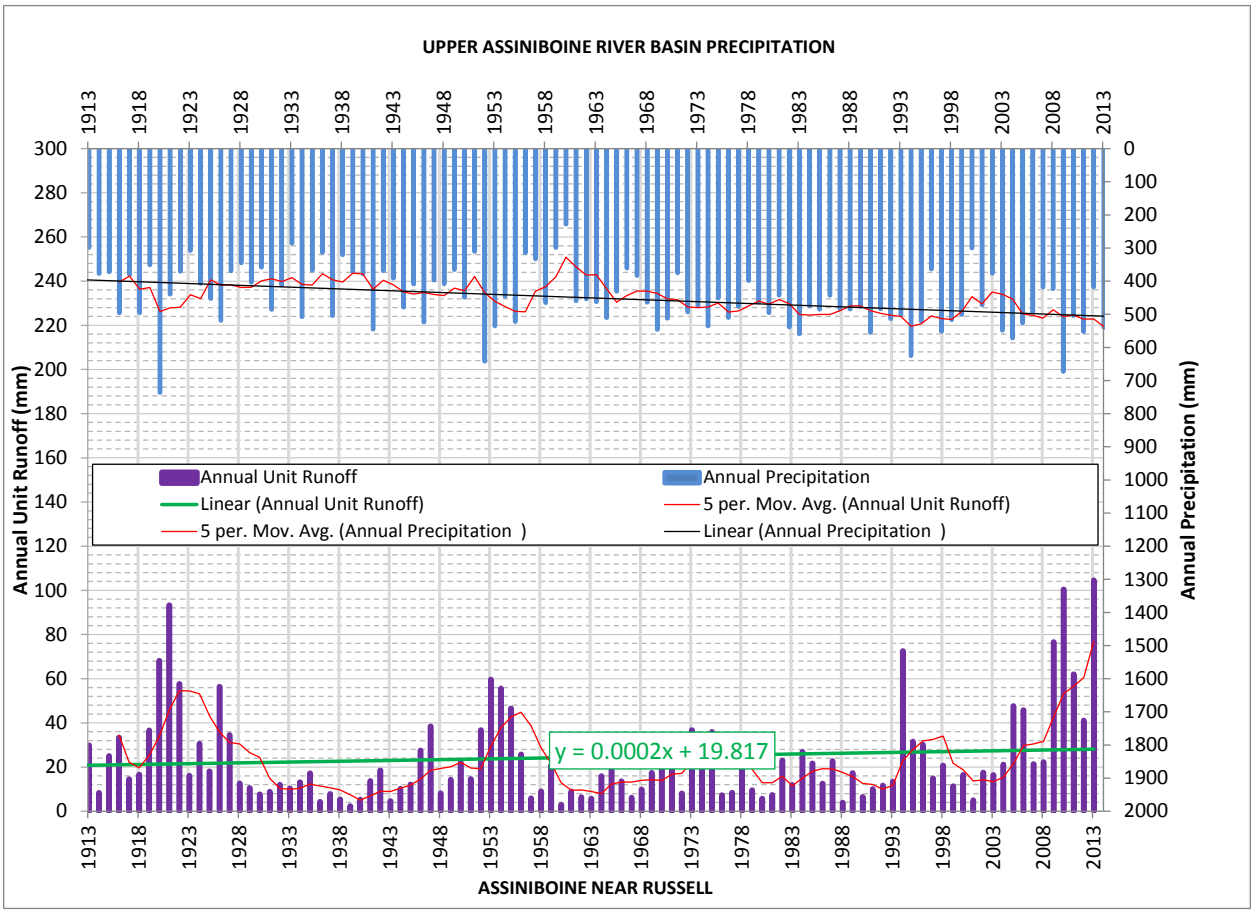
4.0 ASSINIBOINE RIVER NEAR RUSSELL TREND ANALYSIS

The Assiniboine River near Russell was streamflow records dating back to 1913. The Assiniboine River near Russell has been regulated by Shellmouth Dam and Reservoir since 1970. The annual streamflow were adjusted for the effects of Shellmouth Dam. These flows are referred to as "unregulated flows". During the period 1913 to 2014 the Upper Assiniboine River has experienced a number of wet cycles as illustrated on Figure 2 occurring in the 1920s, mid-1950s, mid-1990s and 2010s. As shown on Figure 2, the annual runoff volumes near Russell show a slight increasing trend overtime.

It is important to note that the initial hydrologic conditions influence any trend analysis. If the streamflow records started in the 1930s, a very dry hydrologic period the trend would be a lot

greater. To assess the impact of climate on the annual runoff volumes overtime, an estimate of the basin total annual precipitation was computed by simply averaging the available daily recorded precipitation in the UARB and accumulating it for the year. The initial years had a very sparse climate network. The precipitation records for the UARB indicate an increasing trend in the total annual precipitation as shown on Figure 2. Examination of Brandon annual total precipitation data also indicates an increasing trend overtime, however, much less. This also appears consistent with Statistics Canada precipitation tends over a 62 year period (1948 to 2009) (http://www.statcan.gc.ca/pub/16-002-x/2011003/part-partie3-eng.htm).

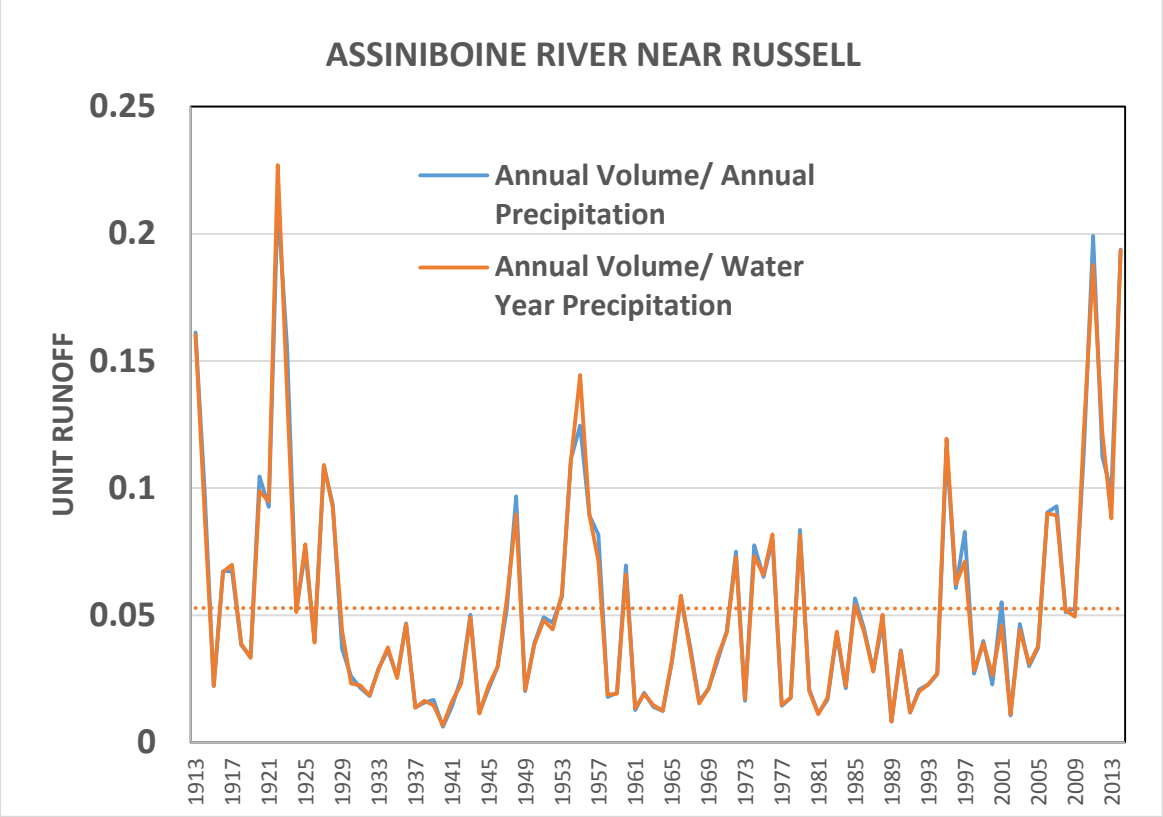
FIGURE 2
ANNUAL RUNOFF VOLUME ASSINIBOINE RIVER NEAR RUSSELL AND UASB ANNUAL PRECIPITATION



To determine if any long-term trends could be detected, the annual volumes for the entire period of record were used in producing the mass curve for the Assiniboine River near Russell. The annual volumes were normalized by dividing the annual volume by the gross drainage area to

obtain a unit runoff for the basin. Using a unit runoff allows comparison with other stations. To assess the impact of increasing basin precipitation on runoff, the unit runoff was divided by the total annual precipitation. Two scenarios were examined: one used the total annual precipitation, the other the water year precipitation (October 1 to September 30). Based on the scenarios there does not appear to be an increase in runoff at Russell overtime when increased precipitation is considered.

FIGURE 3
ASSINIBOINE RIVER NEAR RUSSELL UNIT RUNOFF VOLUMES

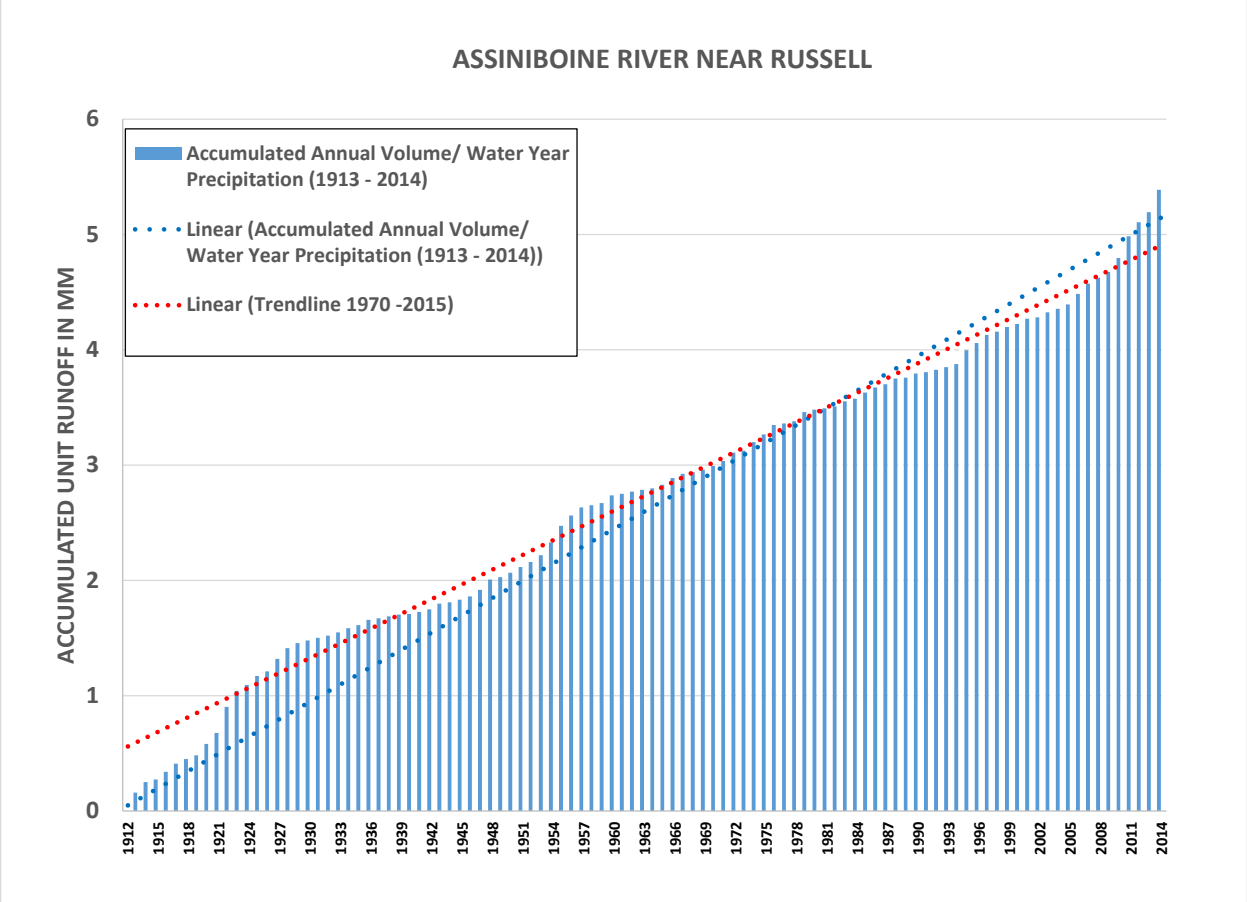


The mass curve for the Assiniboine River near Russell is shown on Figure 4. Two trend lines are shown on the figure: one for the entire period 1913 to 2014 and a second for the period 1970 to 2014. The period 1970 was considered to be representative of current drainage activities. The trend line for the period 1913 to 2014 does not show any significant trend indicating increased runoff volumes due to increased drainage practices since the 1970s. The trend line for period 1970 to 2014 actually shows a decline versus the 1912 to 2014 period. This

is not explainable, as an increasing trend in runoff would be expected because of increased drainage practices in the latter half of the twentieth century.

The mass curve indicates the influence of the climatic conditions. Whenever, there was an abnormally wet period there was a significant increase in the slope of the mass curve indicating significant increases in the annual runoff volumes. This likely due to the triggering of certain thresholds, which govern the hill-slope runoff and wetland fill-spill processes and lead to significant changes in the areas contributing to runoff at the outlet of the watershed. This response appears consistent though out the period of record: for the pre and post drainage periods. This is consistent with Ehsanzadehab et al., 2011 findings that long-term cumulative changes in the watershed, including wetland drainage and changes of farming practices have not had a net detectable influence on the flow.

FIGURE 4
MASS CURVE OF ASSINIBOINE RIVER NEAR RUSSELL UNIT RUNOFF

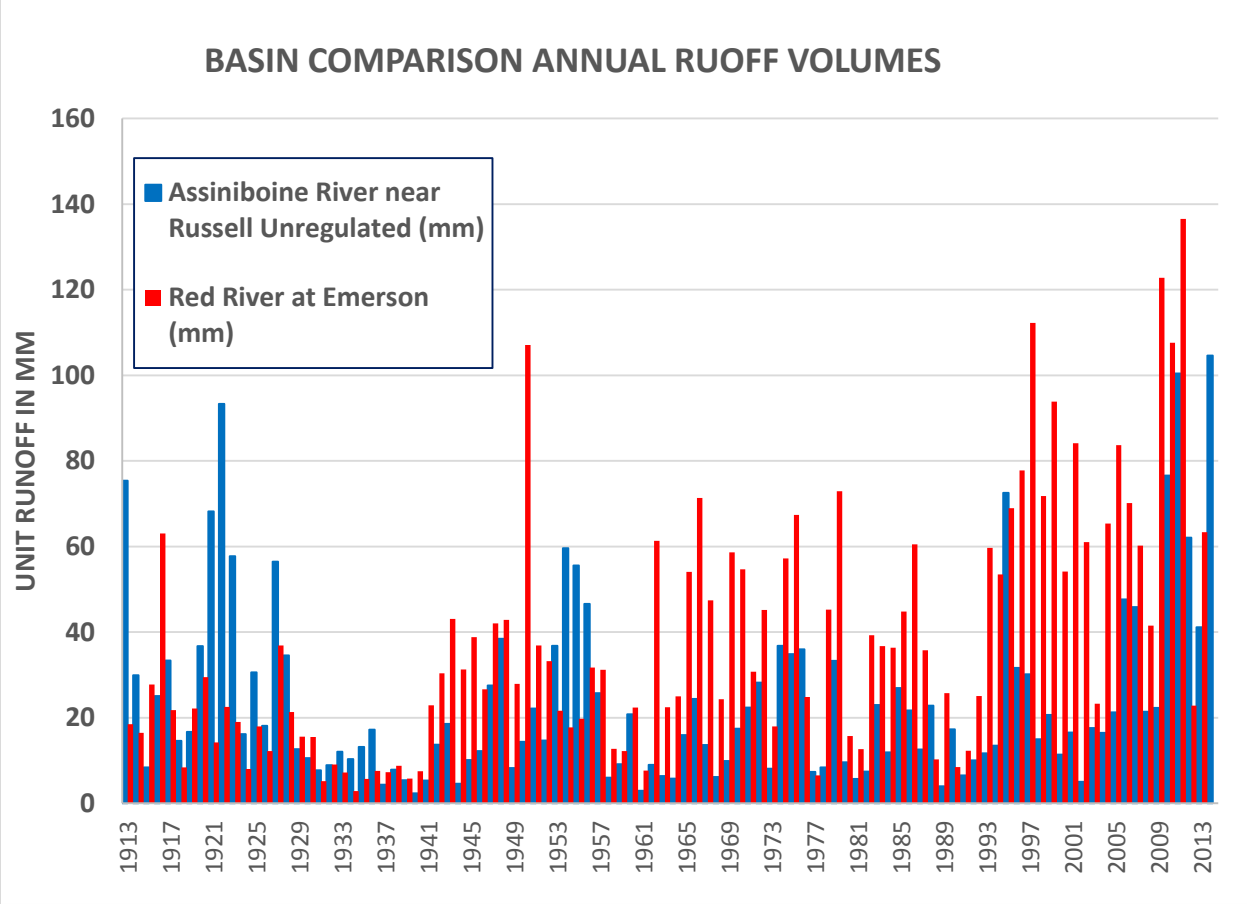


5.0 DIRECT RED AND UARB BASIN COMPARISON

It was considered worthwhile to compare a basin with extensive drainage to the UARB. The Red River Basin was selected as it has a high degree of drainage with the initial drainage occurring in the late 1880s with settlement in the basin. “Manitoba, Minnesota, North Dakota, and South Dakota all share a similar history in terms of when drainage activity peaked. The late 1800’s and early 1900’s saw a proliferation of drainage activity within the RRB, due, in part, to the construction of the railway, which brought great numbers of new settlers to the Basin. As more settlers arrived, the demand for land increased. Legislation, encouraging land drainage for agricultural and economic development, was passed in the three states and Manitoba “(Red River Basin Commission, 2000). Also, the Red River basin has very little streamflow regulation.

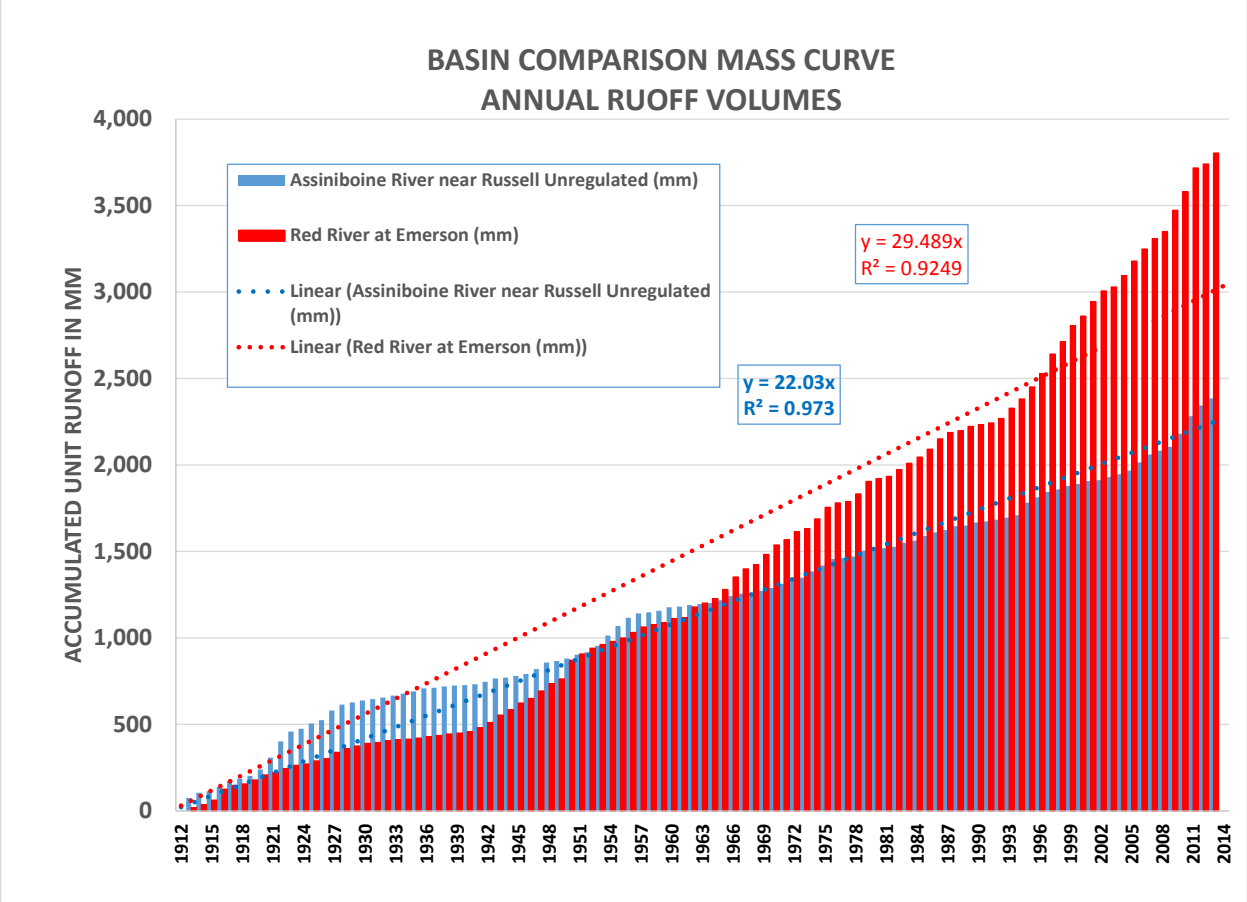
The Red River at Emerson was selected as it has long-term streamflow records starting in 1912. This provides a long-term hydrologic period of record equivalent to the Assiniboine River near Russell. The annual unit runoff for the Red River at Emerson was determined by dividing the annual volume by the gross drainage area. This allows direct comparison with the Assiniboine River near Russell unit runoff. Figure 5 shows the annual unit runoff volumes for the Red River at Emerson and the Assiniboine River near Russell. The Red River‘s annual runoff is about 60% greater than the UARB. This could be in part due to the clay soils in the Red River Basin. Also, the Red River Basin experiences greater average annual precipitation.

FIGURE 5
COMPARISON OF UNIT ANNUAL RUNOFF RED RIVER AND UPPER ASSINIBOINE RIVER BASINS



The mass curve comparing the Red and Assiniboine River Basins indicates a greater stationarity of the Upper Assiniboine River Basin. It is difficult to compare the Red and Upper Assiniboine River Basins because of differences in climatic conditions, soils and land use in the respective basins.

FIGURE 6
MASS CURVE COMPARISON OF ANNUAL RUNOFF RED RIVER AND UPPER ASSINIBOINE RIVER BASINS



6.0 SHELLMOUTH DAM CATCHMENT COMPARISON

Shellmouth Dam became operational in 1971. Inflows to the dam have been computed since. The purpose of the Shellmouth Dam catchment comparison was to identify those areas in its contributing drainage area experiencing increased runoff and see if the increase could be quantified.

"The drainage area of Shellmouth Dam encompasses about 19350 km^2, with about 79 per cent in Saskatchewan and 21 per cent in Manitoba. It is situated within two major physiographic regions — the Saskatchewan Plains and the Saskatchewan Uplands. The former is comprised of the Assiniboine River Plain and the Quill Lake Plain physiographic units; the latter is represented by the Duck Mountain Upland, the Porcupine Hills upland and the Touchwood Hills

Upland. The topography is gently to moderately undulating with the higher relief evident in the northeast portion of the Basin. The northern headwater reaches of the Basin lie within the Boreal Plains Ecozone, an area characterized by groves of brush, wooded bluffs, steeper flow gradients and reasonably well defined drainage patterns. To the south, the lower and largest portion of the Study area falls within the Prairie Ecozone, a much flatter terrain with less brush and fewer trees. During the retreat of the most recent glacier, some 10,000 years ago, a condition known as ablation moraine or dead-ice moraine predominated in this area of the Prairie resulting in a hummocky landscape, an area pock-marked with a myriad of dimpled depressions, sloughs and potholes" (Manitoba et al., 2000). If drainage was having a noticeable effect on runoff volumes it was felt that it should be evident in the portion of the catchment situated in the Prairie Ecozone.

To examine this hypothesis the Shellmouth Dam gross drainage area was subdivided into six catchments based on the available hydrometric station data. The six catchments are described in Table 2.

TABLE 2
SHELLMOUTH DAM CATCHMENTS

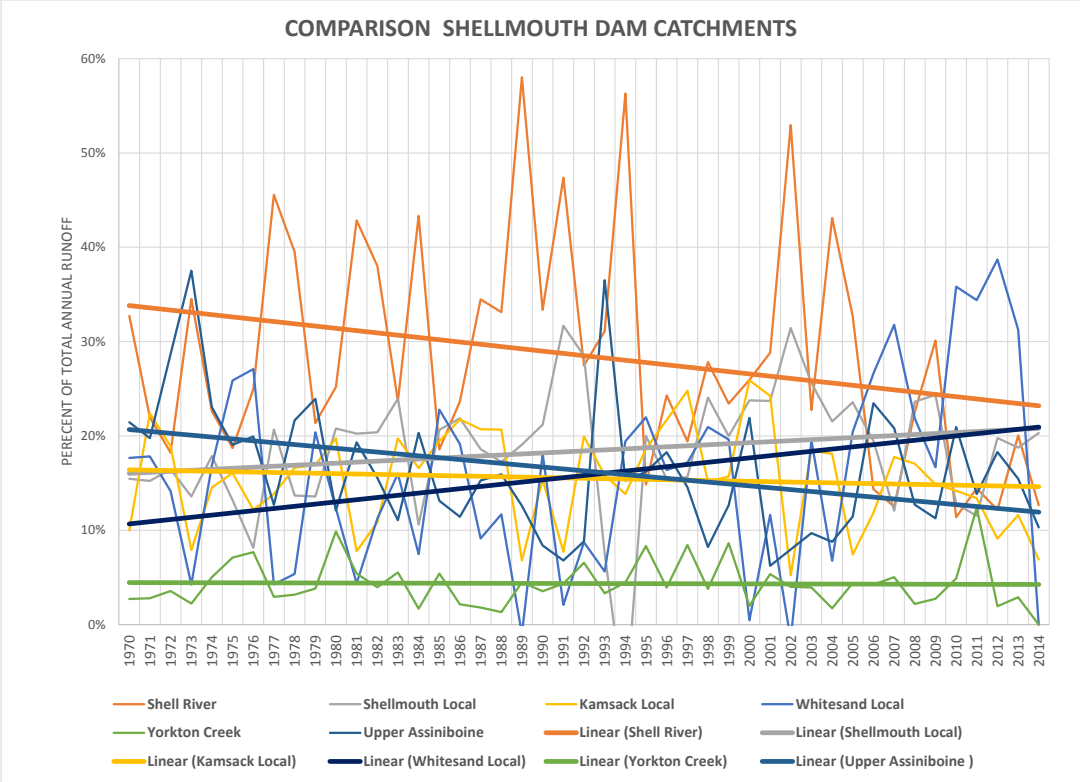
Catchment Name	Description	Gross Drainage Area in km^2
Shell River	Used Shell River near Inglis	1967.7
Shellmouth Local	Equal to Shellmouth Dam inflow minus Shell River near Inglis and Assiniboine River near Kamsack	4328.6
Kamsack Local	Equal to Assiniboine River near Kamsack minus Whitesand River near Canora and Assiniboine River near Sturgis	2287.4
Upper Assiniboine Local	Used Assiniboine River near Sturgis	1939.4
Whitesand Local	Equal to Whitesand River near Canora minus Yorkton Creek near Ebenezer	6510.9
Yorkton Creek	Used Yorkton Creek near Ebenezer	2317.0

The local runoff for each catchment was computed for the open water period (April to October). The open water period was selected as it corresponds with the index stations operating period

used in the analysis. Each catchment's annual open water runoff volume was calculated as described in Table 2. The annual open water runoff for each catchment was divided by total inflow to Shellmouth Dam to determine its percentage contribution. Dividing by the Shellmouth Dam inflow attempted to normalize the variance in the annual runoff volume due to variations in the annual climatic conditions such as melt rate and precipitation.

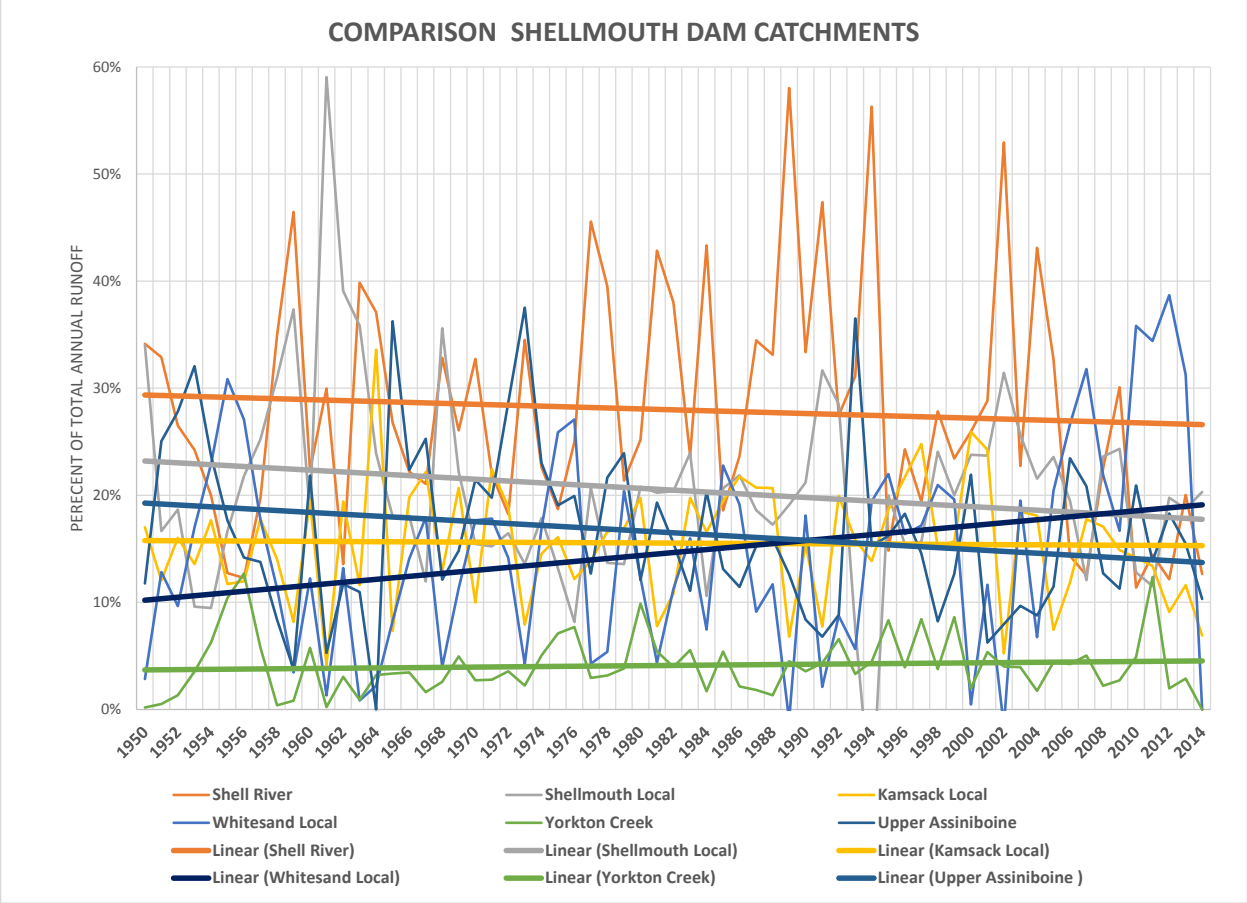
As shown on Figure 7, there is considerable variation in the annual contributions of the various catchments. From the trend lines on Figure 7, it appears that overtime the three catchments associated in the Boreal Plains Ecozone: the Shell River, Upper Assiniboine River and Kamsack Local are contributing a smaller proportion of the total inflow to Shellmouth Dam. The Kamsack Local catchment shows the least decrease with time as it is partially in the Prairie Ecozone. The contribution from Yorkton Creek which is in the Prairie Ecozone shows no significant change overtime. The Shellmouth and Whitesand Local catchments are contributing a higher proportion of the total inflow to Shellmouth Dam overtime. These two catchments are in the Prairie Ecozone where drainage has been prevalent.

FIGURE 7
TREND LINES OF SHELLMOUTH DAM CATCHMENT CONTRIBUTION, 1970 TO 2014



To ensure the trend analysis was not impacted by the wet period currently being experienced in the Upper Assiniboine River Basin, the trend analysis was extended back to 1950s. This was an attempt to capture the historic wet period of the 1950s which predates the period of major drainage activity in the Basin. Figure 8 shows the trend lines obtained for the six catchments for the period 1950 to 2014. Similar trends were observed with the extended data. The exception being the Shellmouth Local, which now has a declining trend over time. The Whitesand Local still exhibits an increasing runoff trend.

FIGURE 8
TREND LINES OF SHELLMOUTH DAM CATCHMENT CONTRIBUTION, 1947 TO 2014



7.0 UARB RUNOFF VOLUME AND PEAK FLOW COMPARISON

The drainage of wetlands may have a significant effect on the hydrologic response of a watershed. The current research by J. Pomeroy and it application to the Smith Creek watershed indicates that the annual stream flow volume and peak daily discharge have a remarkably

strong sensitivity to wetland drainage. The research indicated that wetland drainage had a strong impact on floods due to snowmelt (2011) and rainfall (2012). Overall, Smith Creek total flow volumes over six years simulated (2007-2013) increased 55% due to drainage of wetlands from the current (2008) state, and decreased 26% with restoration to the 1958 state. The increase was largest for years with low runoff, ranging from 200% to 300% increases. The increase in runoff volume was the largest for the flood of record (2011). For the 2011 Flood, the annual flow volume and the peak daily discharge were estimated to increase from 57,317 to 81,227 dam^3 (42%) and from 19.5 to 27.5 m^3/s (41%), respectively, due to wetland drainage that had already occurred in the Smith Creek watershed. If wetland drainage has had such a profound effect on the runoff response of a watershed it should be evident by examining the streamflow records.

To determine if drainage practice has had an impact, a mass curve analysis of annual runoff peaks and open water runoff volumes was undertaken. The analysis used the annual period for peak flows as summer rainfall events are starting to become prominent. Examples are 2010 and 2014.

TABLE 3
1990 LAND COVER UPPER ASSINIBOINE RIVER BASIN (MANITOBA ET AL., 2000A)

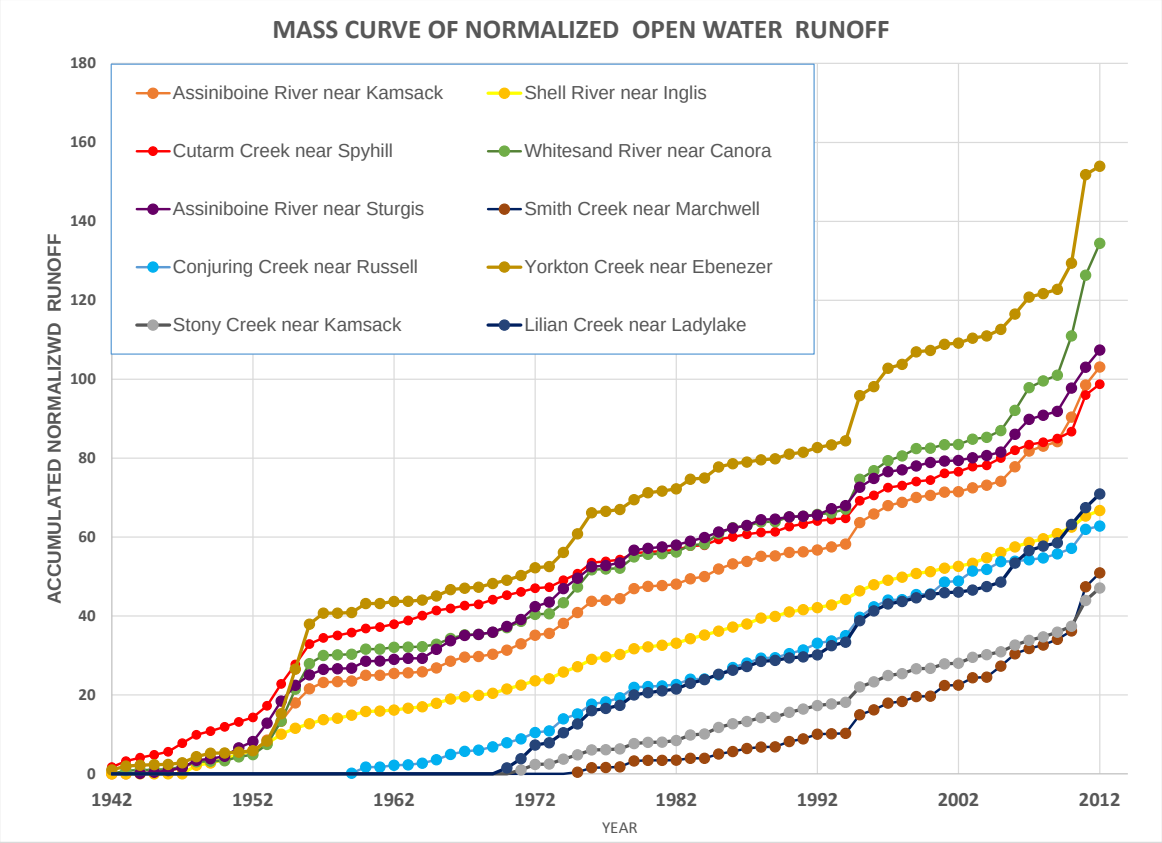
Watershed	Percent and Cover					
	Cropland	Grass	Shrubs	Trees	Wetland	Water
Conjuring Creek	47.3	19.8	0.0	26.9	6.0	0.0
Shell River	17.0	16.5	0.0	54.5	11.8	0.2
Smith Creek	79.1	6.0	4.3	4.3	6.3	0.0
Stony Creek	72.2	12.0	2.5	4.6	8.7	0.0
Spirit Creek	87.8	0.9	6.0	4.0	1.3	0.0
Lilian River	49.1	13.6	11.0	21.2	5.1	0.0
Willow Brook	40.0	26.0	16.0	12.0	6.0	0.0
Yorkton Creek	72.0	13.6	3.9	5.0	4.6	0.9
Whitesand River	63.9	18.6	6.9	5.5	4.4	0.7
Assiniboine River	41.0	11.0	9.0	31.0	7.0	1.0

7.1 RUNOFF VOLUME MASS CURVE ANALYSIS

Mass curves were derived based on recorded daily discharges for the period of record at the hydrometric gauging station. Smaller stream were selected as it was felt the impact of drainage should be more pronounced as opposed to the main stem or large tributaries of the Assiniboine River. However, due to the short period of record on the smaller tributaries, the analysis was expanded to include the larger tributaries, Yorkton Creek, Whitesand River, Shell River and the Sturgis station on the upstream reach of the Assiniboine River. These stations have long-term records dating back to the early 1940s, which may predate the period of major drainage activity in the Basin.

The runoff volumes were normalized by dividing the volume by its median value before accumulating the values for the period of record for each station. The mass curves for the open water runoff volumes are shown on Figure 9. For stations with long-term records such as Yorkton Creek, Cutarm Creek, the Whitesand River and the Assiniboine River near Sturgis the mass curves illustrate the impact of the wet period of the mid-1950s. It should be noted that for the Shell River, the trend is constant overtime, indicating little or no change in runoff response with time and a high degree of stationarity. As indicated by Table 3, the Shell River has only about 17% cropland cover. Conjuring Creek adjacent to the Shell River in Manitoba exhibits a similar trend in runoff response and stationarity. Smith Creek appears to exhibit the greatest increase in runoff response overtime and the highest degree of non-stationarity.

FIGURE 9
MASS CURVE OF NORMALIZED OPEN WATER RUNOFF VOLUMES



7.2 RUNOFF VOLUME DIRECT WATERSHED COMPARISON

A direct station comparison was made of the open water volumes for Smith Creek and the Shell River. The magnitude of the peak flow or volume in a given year is largely determined by the snowmelt conditions and these may not be the same at all stations. However, by comparing stations in the same area major climatic variations should be minimized. Smith Creek should represent a highly drained watershed, whereas, the Shell River should represent a relatively undrained watershed because of its stationarity. It should be noted that however, the response of a watershed may also be impacted by the clearing of trees and vegetation, development of road infrastructure, and alterations to land use practices. Generally, as more wetlands are drained and native parkland cover is converted to cultivation (with clearing of bush and breaking of grasslands) the hydrologic response of the area will be increasingly altered.

To compare Smith Creek and Shell River the unit runoff for the open water period was computed by dividing the open water runoff volume by the gross drainage area at the gauging station. The annual unit runoff shown on Figure 10 indicates an increasing trend overtime for Smith Creek whereas the Shell River is relatively constant. To attempt to remove climatic effects, the unit runoff for Smith Creek was divided by the unit runoff for the Shell River to obtain an annual runoff volume ratio. This ratio shown on Figure 11 has an increasing trend with time. Mass curves were derived based for the unit runoff volume and ratio. The mass curves for the open water unit runoff and runoff ratio are shown on Figure 12. The ratio definitely supports an increase in the unit runoff from Smith Creek watershed overtime.

FIGURE 10
SMITH CREEK AND SHELL RIVER UNIT RUNOFF COMPARISON

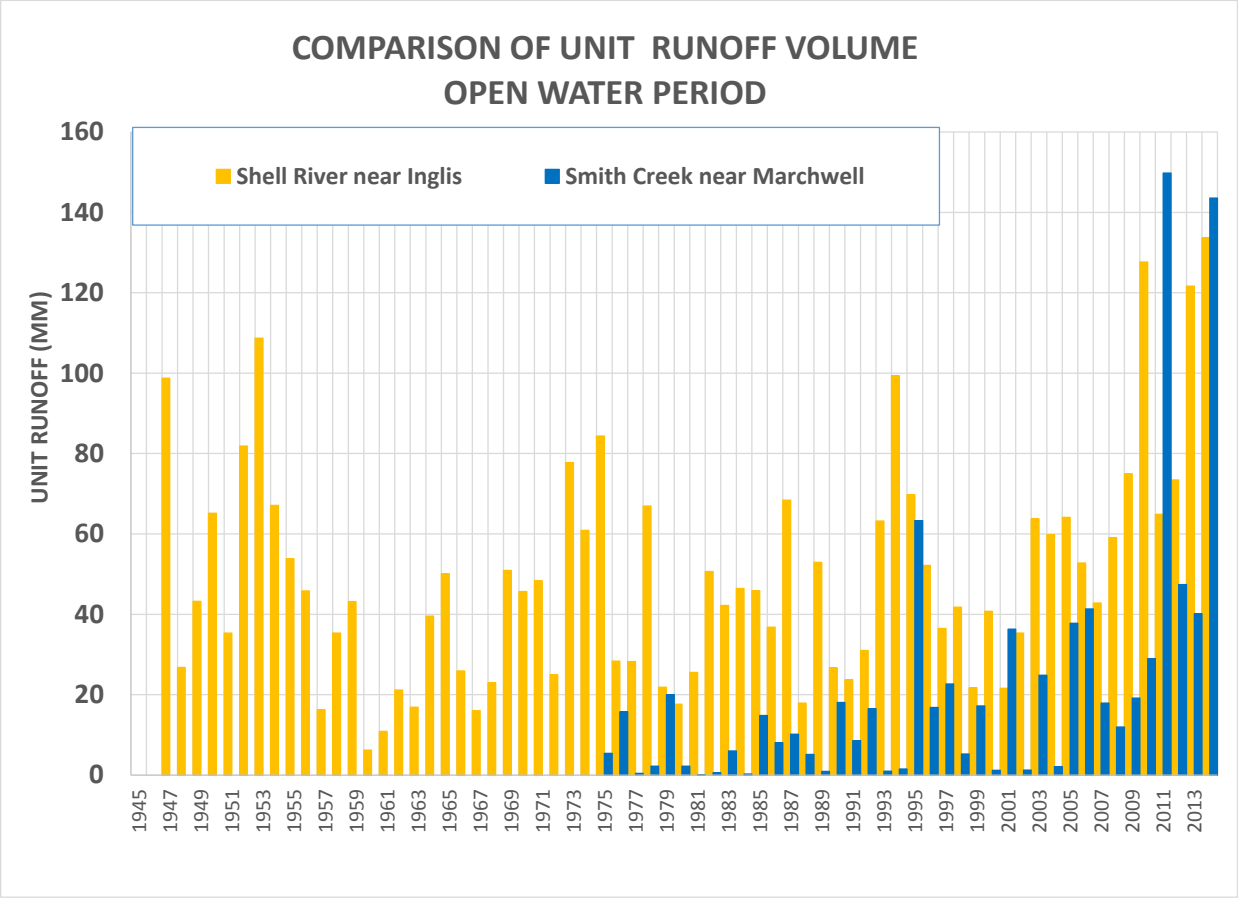


FIGURE 11
SMITH CREEK TO SHELL RIVER RUNOFF VOLUME RATIO

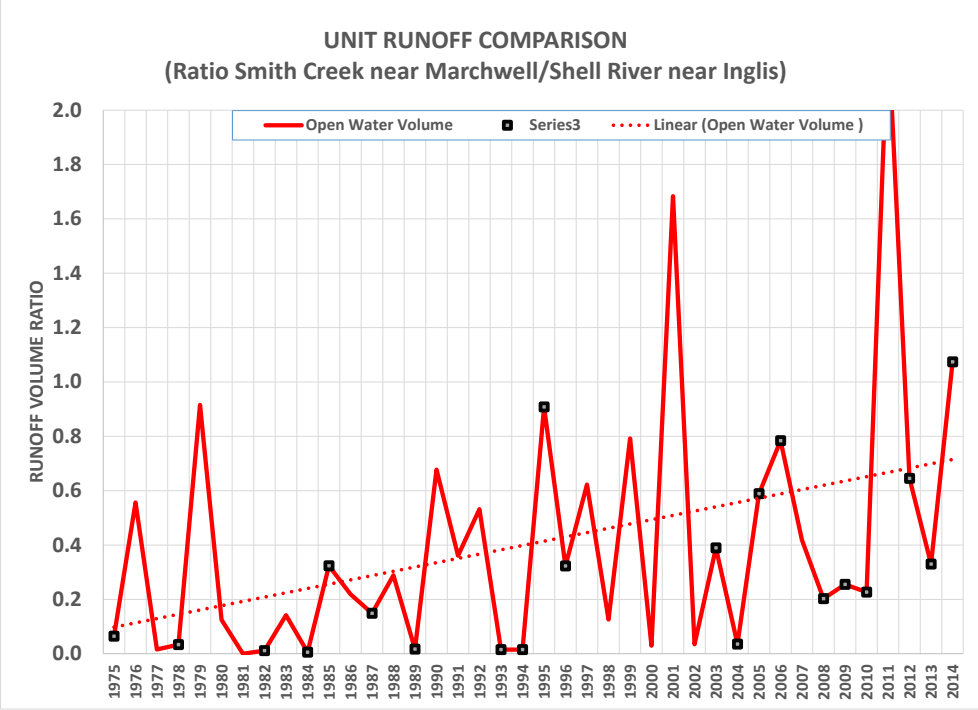
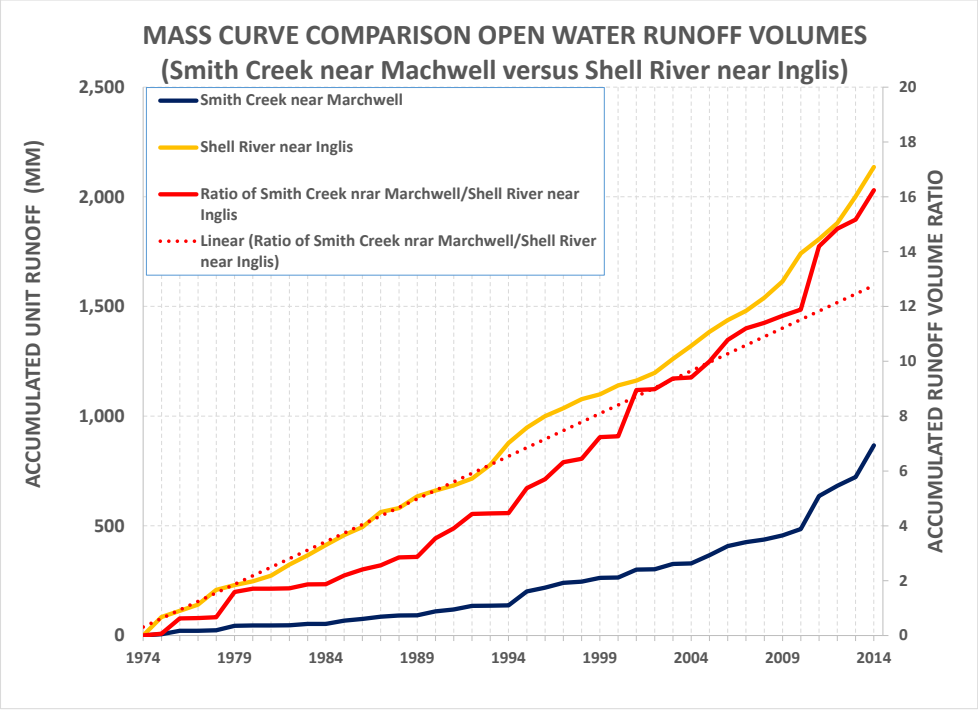


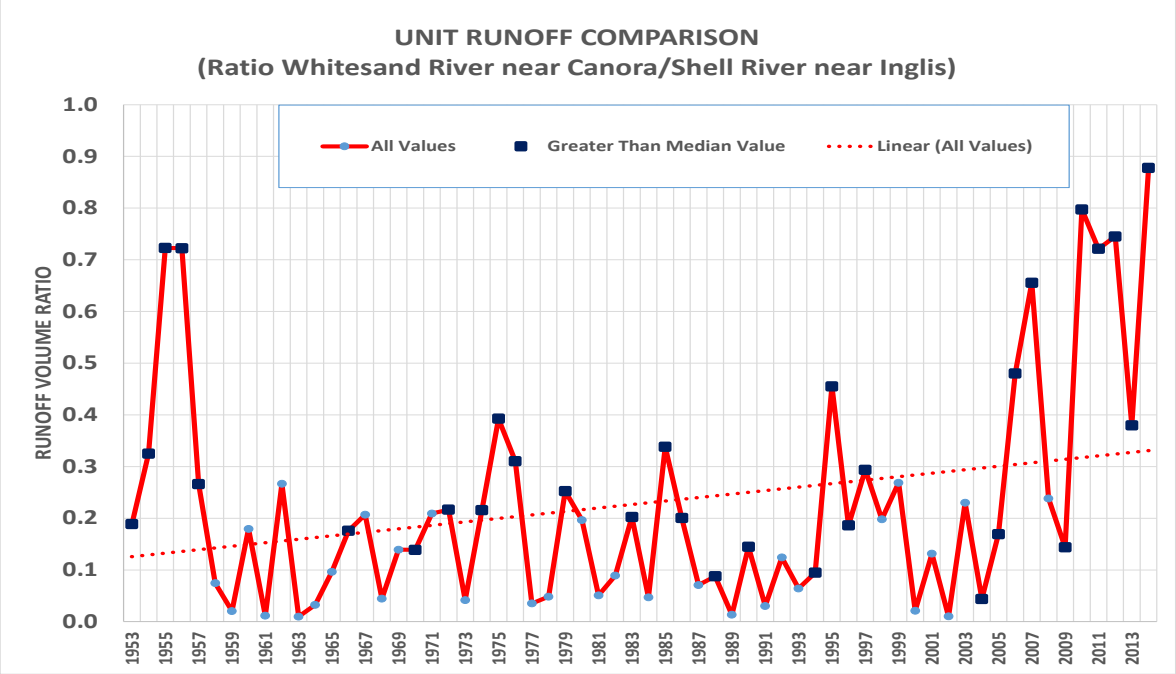
FIGURE 12
MASS CURVE COMPARISON UNIT RUNOFF SMITH CREEK NEAR MARCHWELL VERSUS SHELL RIVER NEAR INGLIS



To verify the approach and findings, the methodology was applied to the Whitesand River near Canora. The Whitesand River has a much larger gross drainage area. The Whitesand River has long- term streamflow records that should predate the period of major drainage activity in the Basin and includes the 1950s wet period. In the Shellmouth Dam catchment comparison, the Whitesand River was the only catchment exhibiting increasing runoff response with time.

The annual unit runoff ratio of the Whitesand River to Shell River is shown on Figure 13. The ratio for the entire period of record (195 to 2014) shows quite a bit of scatter. This was expected as the Whitesand River is in the Prairie Ecozone, whereas, the Shell River Boreal Plains Ecozone. The Prairie Ecozone should exhibit a much greater dynamic contributing drainage areas due to wetland storage effects which vary from year to year based on basin conditions. As the impact of increased runoff on flooding is a primary concern, the years with a ratio greater than median value were plotted on Figure 13. As shown on Figure 13, the scatter in the runoff ratio was not reduced when only years above the median are considered. Generally, the runoff ratio appears to exhibit a weak increasing trend overtime. This appears consistent with the Shellmouth Dam catchment comparison findings. The response of the 1950s is similar to that occurring now. The influence of the initial hydrologic conditions on the trend line is illustrated on Figure 13. If the data did not extend back to the 1950 the trend would have been much stronger.

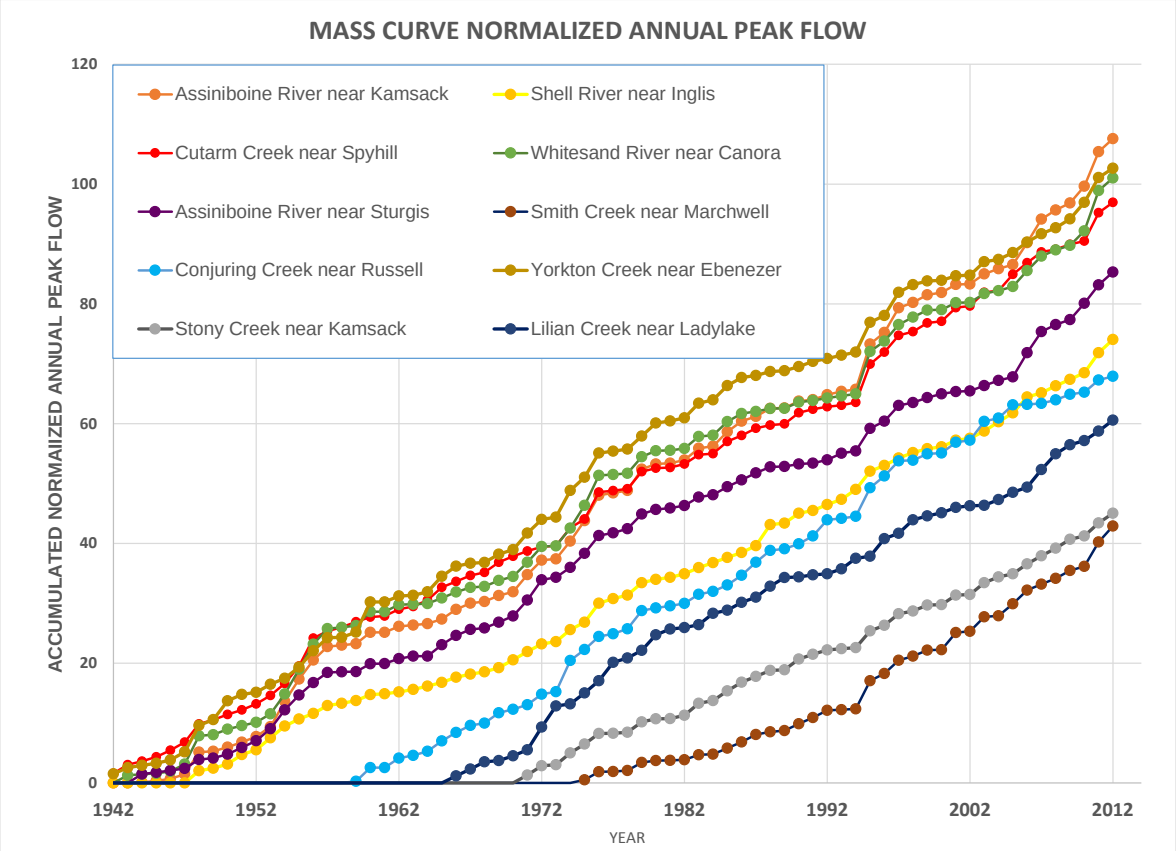
FIGURE 13
WHITESAND RIVER TO SHELL RIVER RUNOFF VOLUME RATIO



7.3 PEAK FLOW MASS CURVE ANALYSIS

The preceding described methodology was applied to the annual flood peaks for the same stations. The mass curves of the annual peak discharge are shown on Figure 14. The mass curves of annual peak flow show more stationarity than for the mass curves of runoff volumes. The Shell River still shows a high degree of stationarity. Comparisons were then done between Smith Creek and the Shell River and the Whitesand River and the Shell River.

FIGURE 14
MASS CURVE OF NORMALIZED ANNUAL PEAK FLOW



7.4 PEAK FLOW DIRECT WATERSHED COMPARISON

By comparing Smith Creek and the Whitesand River to the Shell River an attempt has been made to consider climatic effects. However, the response of a watershed may also be impacted by the clearing of trees and vegetation, development of road infrastructure, and alterations to land use practices which cannot be separated in a streamflow station comparison analysis.

Comparisons were made between Smith Creek, the baseline station, which exhibited a high degree of stationarity and Smith Creek and the Whitesand River both which are in the Prairie Ecozone with a fair degree of drainage. Ratios of annual peak flow were computed for Smith Creek versus the Shell River and the Whitesand River versus the Shell River. The annual ratios were plotted against time and trend lines superimposed in the plots. The annual ratios exhibited considerable variability but increasing trends with time. Smith Creek exhibited a greater

increasing trend than the Whitesand River. The variability decreased if only years above the median value are considered but the same trend exists. The difficulty is isolating and quantifying the impacts of drainage practices as other land management practices although not as predominate impact the runoff response of a watershed.

The peak flow ratio for Smith Creek, shown on Figure 15, has an increasing trend with time similar to its runoff volume ratio. However, the trend may be impacted by the initial hydrologic conditions as the data only starts in the 1970s a moderately wet period. The peak flow ratio for Smith Creek exhibits much more variability and a greater increasing trend than the volume ratio. The ratio appears to indicate that runoff response of the Smith Creek watershed is increasing overtime because of drainage.

The peak flow ratio for the Whitesand River, shown on Figure 16, indicates a decreasing trend with time unlike its runoff volume ratio. The peak flow ratio for the Whitesand River also exhibits much more variability than its runoff volume ratio.

FIGURE 15
SMITH CREEK TO SHELL RIVER PEAK FLOW RATIO

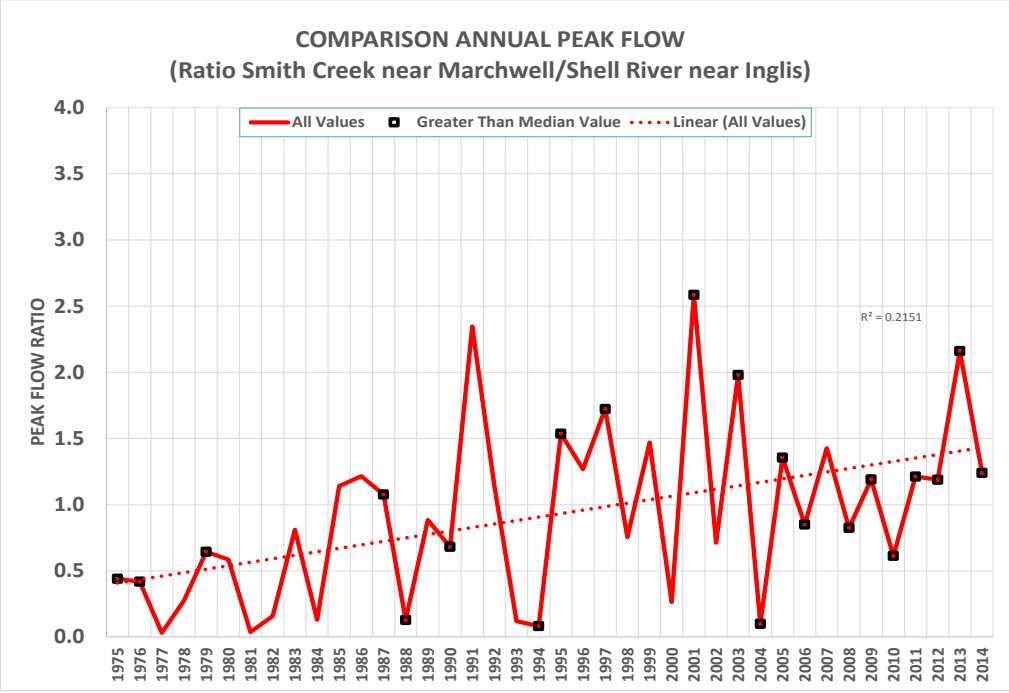
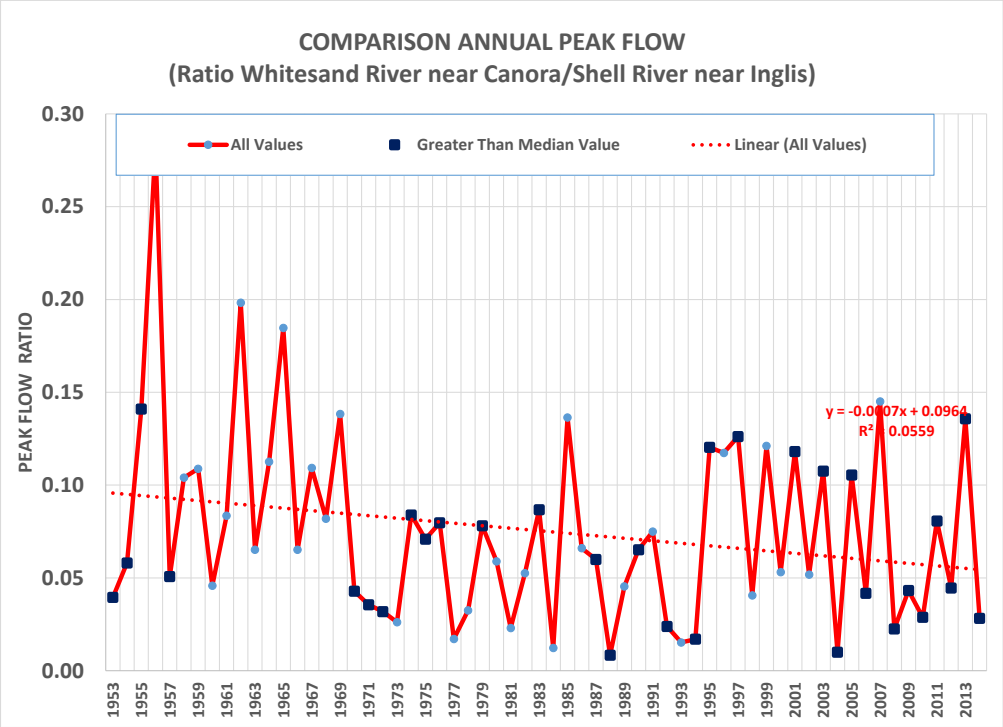


FIGURE 16
WHITESAND RIVER TO SHELL RIVER PEAK FLOW RATIO



8.0 CONCLUSIONS

Hydrologic analysis of streamflow records in the Upper Assiniboine River Basin indicates that both peak flows and runoff volumes appear to be increasing. The difficulty is isolating and quantifying the impacts of drainage practices from other land management practices. Changes in drainage practices is likely the predominate impact on the increasing runoff response of the watershed. Drainage practices generally results in a gradual change and as such is not easily quantifiable and isolatable through the analysis of streamflow data. This makes the detection of drainage nearly impossible from the direct analysis of streamflow data. Even through a paired watershed approach, the impacts of drainage practices could not be quantified. Ideally, it may take paired watersheds with the same land use practices but with different drainage practices: one in a drained state and the other in undrained state.

9.0 REFERENCES

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APPENDIX O

DETAILS AND RESULTS OF ECONOMIC ANALYSIS

BASE EVALUATION OF INDIVIDUAL OPTIONS

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Summary List of Flood Mitigation Alternatives - Individual Options

Study Area	Concepts	Options Considered	Capacity of Options	Capital Cost (Million $)	Average Annual Benefits (Million $)				B/C Ratio	Comments
					Lake Manitoba & Lake St. Martin	Lower Assiniboine	City of Winnipeg	Local Benefits and/or Other Regions		
Upper Assiniboine	Diking along Upper Assiniboine River	Shellmouth to Brandon	1:5 year	297.6	-(1)	-(1)	-(1)	1.7	0.13(2)	Raising of water levels. Oxbows and small streams cut-off from river.
		Shellmouth to Brandon	1:50 year	470.8	-(1)	-(1)	-(1)	7.3	0.32(2)	Raising of water levels. Oxbows and small streams cut-off from river.
		Shellmouth to St. Lazare	1:5 year	72.5	-(1)	-(1)	-(1)	0.2	0.06(2)	Raising of water levels. Oxbows and small streams cut-off from river.
		Shellmouth to St. Lazare	1:50 year	281.9	-(1)	-(1)	-(1)	1.0	0.07(2)	Raising of water levels. Oxbows and small streams cut-off from river.
	"Buyout" Scheme along Upper Assiniboine River	Shellmouth to St. Lazare	3,000 cfs	24.0	+(3)	+(3)	+(3)	1.1	1.00	Benefits calculation assumes no seeding takes place regardless of flood duration or timing.
	Developing Storage on the Assiniboine River and Tributaries	Alexander Dam	1,600,000 dam^3	590.6	2.46	10.08	1.42	+(5)	0.50(2)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
		Holland Dam	880,000 dam^3	306.0	1.20	5.13	0.78	+(5)	0.49(2)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
		Victor Dam	170,000 dam^3	162.3	< 1.20 (4)	1.68	0.23	+(5)	0.4(2)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
		Zelena Dam	250,000 dam^3	103.9	< 1.20 (4)	0.97	0.12	+(5)	0.46(2)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
Souris River	Developing Storage on the Souris River	High Souris Dam	40,000 dam^3	130.5	< 1.20 (4)	2.38	0.28	+(5)	0.62(2)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
		Nesbitt Dam	410,000 dam^3	237.8	< 1.20 (4)	3.31	0.39	+(5)	0.43(2)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
	Assessment of Diking Options	Town of Souris Community Dikes	1:200 year	1.5	N/A	N/A	N/A	0.026	0.35	
		Town of Souris Community Dikes	1:500 year	3.9	N/A	N/A	N/A	0.086	0.46	
		Town of Wawanesa Community Dikes	1:500 year	1.5	N/A	N/A	N/A	0.036	0.50	
		Town of Melita Community Dikes	1:200 year	0.2	N/A	N/A	N/A	0.001	0.16	
		Town of Melita Community Dikes	1:500 year	1.3	N/A	N/A	N/A	0.011	0.18	
Lower Assiniboine	Improvement Schemes for the Lower Assiniboine Dikes	Upgrade Existing Dikes	18,000 cfs	152.6	N/A	0.50	-0.02	N/A	0.07	
		Upgrade Existing Dikes	20,000 cfs	194.6	0.09	0.86	-0.21	N/A	0.08	
		Upgrade Existing Dikes	25,000 cfs	320.5	0.34	2.51	-0.59	N/A	0.15	
		Upgrade Existing Dikes	35,000 cfs	557.1	0.80	6.12	-1.01	N/A	0.22	
		Major Setbacks to Dikes	18,000 cfs	745.7	N/A	0.50	-0.02	N/A	0.01	
		Major Setbacks to Dikes	20,000 cfs	788.6	0.09	0.86	-0.21	N/A	0.02	
		Major Setbacks to Dikes	25,000 cfs	925.9	0.34	2.51	-0.59	N/A	0.05	
		Major Setbacks to Dikes	35,000 cfs	1234.4	0.80	6.12	-1.01	N/A	0.10	
		Combination of Upgrades and Setbacks to Dikes	18,000 cfs	459.4	N/A	0.50	-0.02	N/A	0.02	
		Combination of Upgrades and Setbacks to Dikes	20,000 cfs	496.1	0.09	0.86	-0.21	N/A	0.03	
		Combination of Upgrades and Setbacks to Dikes	25,000 cfs	640.7	0.34	2.51	-0.59	N/A	0.07	
		Combination of Upgrades and Setbacks to Dikes	35,000 cfs	906.3	0.80	6.12	-1.01	N/A	0.14	
	Improvement Schemes Downstream of the Assiniboine Dikes	Expansion to Headingley	18,000 cfs	114.1	N/A	0.15	N/A	N/A	0.03	
		Expansion to Headingley	23,100 cfs	155.0	N/A	0.51	N/A	N/A	0.07	
		Expansion to Headingley	28,000 cfs	197.1	N/A	0.97	N/A	N/A	0.10	
		Combination of Individual Flood Proofing, Neighbourhood Diking and Purchase of Vulnerable Properties	18,000 cfs	21.4	N/A	0.15	N/A	N/A	0.15	
			23,100 cfs	62.6	N/A	0.51	N/A	N/A	0.17	
			28,000 cfs	122.9	N/A	0.97	N/A	N/A	0.17	
	Control Structure at Hoop and Holler Bend on Assiniboine River	"Wasteway" only	3,900 cfs	109.6	N/A	<2.71(6)	N/A	N/A	0.52(2)	Deliberate flooding of land. Interbasin transfer. Increased flows downstream on La Salle River.
		Control Structure with "Wasteway"	3,900 cfs	118.9	N/A	<2.71(6)	N/A	N/A	0.48(2)	Deliberate flooding of land. Interbasin transfer. Increased flows downstream on La Salle River.
		Control Structure with Diversion Channel	3,900 cfs	386.9	N/A	2.71	N/A	N/A	0.15(2)	Interbasin transfer. Increased flows downstream on La Salle River.
Portage Diversion	Increasing Capacity of Portage Diversion	Channel Widening	34,000 cfs	479.3 (7)	-0.73	6.42	1.02	N/A	0.32	
		Parallel Channel	34,000 cfs	264.9	-0.73	6.42	1.02	N/A	0.53	
		Parallel Channel	46,000 cfs	405 (7)	-1.21	9.49	1.34	N/A	0.58	
		Retrofit	34,000 cfs	237.6 (7)	-0.73	6.42	1.02	N/A	0.68	
Dauphin Lake	Diversion Channel	Short Channel (30 m wide weir)	4,000 cfs	143.5	-(8)	N/A	N/A	1.0	0.15	
		Short Channel (60 m wide weir)	7,500 cfs	253.5	-(8)	N/A	N/A	1.3	0.11	
		East Channel	4,000 cfs	211.4	-(8)	N/A	N/A	1.0	0.10	
		West Channel	4,000 cfs	322.8	-(8)	N/A	N/A	1.0	0.07	
	Mossy River Excavation	Excavate Channel by 0.3 m	Varies	31.6	-(8)	N/A	N/A	0.6	0.38(2)	Physical and biological impacts due to dredging of river.
		Excavate Channel by 0.5 m	Varies	52.6	-(8)	N/A	N/A	0.8	0.33(2)	Physical and biological impacts due to dredging of river.
		Excavate Channel by 1.0 m	Varies	105.3	-(8)	N/A	N/A	1.2	0.24(2)	Physical and biological impacts due to dredging of river.
Lake Winnipegosis	Lake Winnipegosis Diversion	Diversion to Cedar Lake	2,500 cfs	160.2	2.06	N/A	N/A	0.3(9)	0.31(2)	Potential for First Nations impacts at Cedar Lake, Changes to natural Lake Winnipegosis regime.
		Diversion to Cedar Lake	5,000 cfs	215.2	3.47	N/A	N/A	0.6(9)	0.39(2)	Potential for First Nations impacts at Cedar Lake, Changes to natural Lake Winnipegosis regime.
		Diversion to Cedar Lake	10,000 cfs	325.2	4.62	N/A	N/A	1.1(9)	0.37(2)	Potential for First Nations impacts at Cedar Lake, Changes to natural Lake Winnipegosis regime.
		Diversion to Lake Winnipeg	10,000 cfs	1334.0	4.62	N/A	N/A	N/A	0.08(2)	Changes to natural Lake Winnipegosis regime.
	Lake Winnipegosis Outlet	Control Structure in Waterhen River	Varies	57.7	2.58	N/A	N/A	- (10)	0.94(2)	Increased water level. Changes to Waterhen River regime. Potential for First Nations impacts.
Lake Manitoba and Lake St. Martin	LMB & LSM Outlet Channels (11)	Option C1 - Channel less South of Pinaymootang FN	5,000 cfs	228.8 (12)	5.04 (13)	N/A	N/A	N/A	0.46	
		Option C2 - Channel less South of Pinaymootang FN	10,000 cfs	426.1 (12)	8.31 (13)	N/A	N/A	N/A	0.41	
		Option C3 - Channel less South of Pinaymootang FN	15,000 cfs	615.3 (12)	9.15 (13)	N/A	N/A	N/A	0.31	
		Option D1 - Channel following Birch Creek	5,000 cfs	212.3 (12)	5.04 (13)	N/A	N/A	N/A	0.50	
		Option D2 - Channel following Birch Creek	10,000 cfs	386.6 (12)	8.31 (13)	N/A	N/A	N/A	0.45	
		Option D3 - Channel following Birch Creek	15,000 cfs	553.6 (12)	9.15 (13)	N/A	N/A	N/A	0.35	
Varies	Multitude of Small Dams	Total of 21 Dams	200,000 dam^3	482.2	< 1.20 (4)	3.75	0.60	+ (14)	0.24	

Notes:

1. Loss in flood plain storage could increase flows downstream.
2. Potential significant environmental impacts. Not Identified as a preferred alternative.
3. Alternate Shellmouth Dam operating strategy could have benefits downstream.
4. Based on results of the Holland Dam option. Actual benefits would be less.
5. Local benefits likely relatively small and assumed negligible.
6. Based on Control Structure with Diversion Channel. Actual benefits would be less due to overland flooding downstream of structure.
7. The cost of "Do Nothing" was subtracted from the Capital Cost of the option.
8. Dis-benefit due to increase in LMB water level is likely negligible.
9. Potential benefits of power generation on Cedar Lake.
10. Potential damages due to increase of water levels on Lake Winnipegosis.
11. Only Lake Manitoba options and capacity listed. Corresponding Lake St. Martin options and capacity varies. B/C Ratio only computed for preferred alternatives.
12. The cost of making the existing channel (base condition) permanent was subtracted from the Capital Cost of LMB & LSM alternatives.
13. Average Annual Benefit calculated assuming the Portage Diversion has a capacity of 34,000 cfs.
14. Water supply benefits and other local benefits not computed.

\+ indicates benefits
\- indicates dis-benefits

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Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

Sheet:	1	**of**	1
Date:	16-Jun-15	**By:**	PAL
Checked:	14-Jul-15	**By:**	GW

Upper Assiniboine River
Diking to Protect Agricultural Land from Shellmouth Dam to Brandon
SUMMARY OF COSTS AND BENEFITS
FPL 1:5 Year Return Period

Summary of Costs

Construction Item		Cost
1	Dike (fill + land acquisition + revegetation)	$87,870,130
2	Pump Stations	$70,000,000
	Subtotal of Costs	***$157,870,130***
3	Miscelaneous Quantity Allowance (20% of direct costs)	$31,574,026
4	Environmental Mitigation Costs (5% of direct costs)	$7,893,506
5	Indirect Costs (20% of direct costs)	$31,574,026
	Subtotal of Costs	***$228,911,688***
6	Contingencies (30% of costs)	$68,673,506
Total Dike Construction Cost		***$297,585,194***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,487,926 /yr
(1% of Capital Cost)

Present Value of Costs $329,247,374
(In 2014$)

Benefits

Total Average Annual Benefit $1,717,875 /yr
(Section 8.6.5)

Present Value of Benefits $43,683,749
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.133	**-$285,563,625**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
File No:	12-0300-011		
Project:	AR & LMB Basins Flood Mitigation Study		
Sheet:	1	**of**	1
Date:	16-Jun-15	**By:**	PAL
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Upper Assiniboine River
Diking to Protect Agricultural Land from Shellmouth Dam to Brandon
SUMMARY OF COSTS AND BENEFITS
FPL 1:50 Year Return Period

Summary of Costs

Construction Item		Cost
1	Dike (fill + land acquisition + revegetation)	$179,780,774
2	Pump Stations	$70,000,000
	Subtotal of Costs	***$249,780,774***
3	Miscelaneous Quantity Allowance (20% of direct costs)	$49,956,155
4	Environmental Mitigation Costs (5% of direct costs)	$12,489,039
5	Indirect Costs (20% of direct costs)	$49,956,155
	Subtotal of Costs	***$362,182,122***
6	Contingencies (30% of costs)	$108,654,637
Total Dike Construction Cost		***$470,836,759***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $4,708,368 /yr
(1% of Capital Cost)

Present Value of Costs $571,028,021
(In 2014$)

Benefits

Total Average Annual Benefit $7,260,084 /yr
(Section 8.6.5)

Present Value of Benefits $184,616,276
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.323	**-$386,411,745**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

Sheet:	1	**of**	1
Date:	16-Jun-15	**By:**	PAL
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Upper Assiniboine River
Diking to Protect Agricultural Land from Shellmouth Dam to St. Lazare
SUMMARY OF COSTS AND BENEFITS
FPL 1:5 Year Return Period

Summary of Costs

Construction Item		Cost
1	Dike (fill + land acquisition + revegetation)	$18,448,483
2	Pump Stations	$20,000,000
	Subtotal of Costs	***$38,448,483***
3	Miscelaneous Quantity Allowance (20% of direct costs)	$7,689,697
4	Environmental Mitigation Costs (5% of direct costs)	$1,922,424
5	Indirect Costs (20% of direct costs)	$7,689,697
	Subtotal of Costs	***$55,750,300***
6	Contingencies (30% of costs)	$16,725,090
Total Dike Construction Cost		***$72,475,391***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $724,754 /yr
(1% of Capital Cost)

Present Value of Costs $87,897,723
(In 2014$)

Benefits

Total Average Annual Benefit $218,854 /yr
(Section 8.6.5)

Present Value of Benefits $5,565,226
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.063	**-$82,332,496**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Upper Assiniboine River
Diking to Protect Agricultural Land from Shellmouth Dam to St. Lazare
SUMMARY OF COSTS AND BENEFITS
FPL 1:50 Year Return Period

Summary of Costs

Construction Item		Cost
1	Dike (fill + land acquisition + revegetation)	$129,530,591
2	Pump Stations	$20,000,000
	Subtotal of Costs	***$149,530,591***
3	Miscelaneous Quantity Allowance (20% of direct costs)	$29,906,118
4	Environmental Mitigation Costs (5% of direct costs)	$7,476,530
5	Indirect Costs (20% of direct costs)	$29,906,118
	Subtotal of Costs	***$216,819,357***
6	Contingencies (30% of costs)	$65,045,807
Total Dike Construction Cost		***$281,865,164***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,818,652 /yr
(1% of Capital Cost)

Present Value of Costs $341,844,395
(In 2014$)

Benefits

Total Average Annual Benefit $983,643 /yr
(Section 8.6.5)

Present Value of Benefits $25,013,004
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.073	**-$316,831,391**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Upper Assiniboine River
Purchase of Flood Prone Lands - Shellmouth Dam to St. Lazare
SUMMARY OF COSTS AND BENEFITS
FPL 1:10 Year Return Period (3,000 cfs)

Summary of Costs

Total Dike Construction Cost ***$23,956,344***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	0	percent

Costs

Annual O & M Cost $0 /yr
(1% of Capital Cost)

Present Value of Costs $23,956,344
(In 2014$)

Benefits

Total Average Annual Benefit $1,130,010 /yr
(Section 8.6.5)

Present Value of Benefits $24,045,941
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
1.004	**$89,597**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Alexander Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 1,600,000 dam^3

Summary of Costs

Total Construction Cost ***$590,643,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$2,456,343** /yr
Lower Assiniboine (Section 8.6.1)	**$10,079,626** /yr
City of Winnipeg (Section 8.6.3)	**$1,416,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $5,906,430 /yr
(1% of Capital Cost)

Present Value of Costs $716,328,318
(In 2014$)

Benefits

Total Average Annual Benefit $13,951,969 /yr

Present Value of Benefits $354,783,851
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.495	**-$361,544,467**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $371,140,449
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $180,865,990
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.487	**-$190,274,459**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastructure and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Victor Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 170,000 dam^3

Summary of Costs

Total Construction Cost ***$162,276,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$1,683,116** /yr
City of Winnipeg (Section 8.6.3)	**$234,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $1,622,760 /yr
(1% of Capital Cost)

Present Value of Costs $196,807,368
(In 2014$)

Benefits

Total Average Annual Benefits $3,118,567 /yr

Present Value of Benefits $79,301,868
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.403	**-$117,505,499**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Zelena Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 250,000 dam^3

Summary of Costs

Total Construction Cost ***$103,877,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$972,555** /yr
City of Winnipeg (Section 8.6.3)	**$121,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $1,038,770 /yr
(1% of Capital Cost)

Present Value of Costs $125,981,408
(In 2014$)

Benefits

Total Average Annual Benefits $2,295,006 /yr

Present Value of Benefits $58,359,581
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.463	**-$67,621,827**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Souris River
High Souris Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 40,000 dam^3

Summary of Costs

Total Construction Cost ***$130,537,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$2,383,325** /yr
City of Winnipeg (Section 8.6.3)	**$281,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $1,305,370 /yr
(1% of Capital Cost)

Present Value of Costs $158,314,497
(In 2014$)

Benefits

Total Average Annual Benefit $3,865,776 /yr

Present Value of Benefits $98,302,605
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.621	**-$60,011,892**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastructure and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Souris River
Nesbitt Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 410,000 dam^3

Summary of Costs

Total Construction Cost ***$237,783,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$3,311,020** /yr
City of Winnipeg (Section 8.6.3)	**$390,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $2,377,830 /yr
(1% of Capital Cost)

Present Value of Costs $288,381,808
(In 2014$)

Benefits

Total Average Annual Benefit $4,902,471 /yr

Present Value of Benefits $124,664,665
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.432	**-$163,717,143**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Town of Souris
Souris River Community Dike
SUMMARY OF COSTS AND BENEFITS
1:200 year Flood

Summary of Costs

Total Dike Construction Cost ***$1,545,562***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $15,500 /yr
(1% of Capital Cost)

Present Value of Costs $1,875,393
(In 2014$)

Benefits

Total Average Annual Benefit $25,600 /yr
(Section 8.6.6)

Present Value of Benefits $650,981
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.347	**-$1,224,412**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
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Project: AR & LMB Basins Flood Mitigation Study

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Town of Souris
Souris River Community Dike
SUMMARY OF COSTS AND BENEFITS
1:500 year Flood

Summary of Costs

Total Dike Construction Cost ***$3,935,591***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $39,400 /yr
(1% of Capital Cost)

Present Value of Costs $4,774,000
(In 2014$)

Benefits

Total Average Annual Benefit $85,500 /yr
(Section 8.6.6)

Present Value of Benefits $2,174,175
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.455	**-$2,599,825**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Town of Melita
Souris River Community Dike
SUMMARY OF COSTS AND BENEFITS
1:200 year Flood

Summary of Costs

1.	South Dike (R.M. of Arthur)	$	-
2.	East Dike (Town of Melita)	$	178,133

Total Dike Construction Cost ***$178,133***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,800 /yr
(1% of Capital Cost)

Present Value of Costs $216,436
(In 2014$)

Benefits

Total Average Annual Benefit $1,400 /yr
(Section 8.6.6)

Present Value of Benefits $35,601
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.164	**-$180,835**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Town of Melita
Souris River Community Dike
SUMMARY OF COSTS AND BENEFITS
1:500 year Flood

Summary of Costs

1.	South Dike (R.M. of Arthur)	$	623,570
2.	East Dike (Town of Melita)	$	674,561

Total Dike Construction Cost ***$1,298,131***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $13,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,574,763
(In 2014$)

Benefits

Total Average Annual Benefit $11,200 /yr
(Section 8.6.6)

Present Value of Benefits $284,804
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.181	**-$1,289,959**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Town of Wawanesa
Souris River Community Dike
SUMMARY OF COSTS AND BENEFITS
1:500 year Flood

Summary of Costs

Total Dike Construction Cost ***$1,490,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $14,900 /yr
(1% of Capital Cost)

Present Value of Costs $1,807,063
(In 2014$)

Benefits

Total Average Annual Benefit $35,700 /yr
(Section 8.6.6)

Present Value of Benefits $907,813
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.502	**-$899,250**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Upgrade of Existing Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 18,000 cfs

Summary of Costs

Total Dike Construction Cost ***$152,600,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$501,492**	/yr
City of Winnipeg (Section 8.6.3)	**-$19,000**	/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,526,000 /yr
(1% of Capital Cost)

Present Value of Costs $185,072,373
(In 2014$)

Benefits

Total Average Annual Benefit $482,492 /yr

Present Value of Benefits $12,269,262
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.066	**-$172,803,110**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Upgrade of Existing Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 20,000 cfs

Summary of Costs

Total Dike Construction Cost ***$194,600,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$88,880** /yr
Lower Assiniboine (Section 8.6.1)	**$862,635** /yr
City of Winnipeg (Section 8.6.3)	**-$213,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $1,946,000 /yr
(1% of Capital Cost)

Present Value of Costs $236,009,723
(In 2014$)

Benefits

Total Average Annual Benefit $738,515 /yr

Present Value of Benefits $18,779,657
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.080	**-$217,230,066**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Upgrade of Existing Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 25,000 cfs

Summary of Costs

Total Dike Construction Cost ***$320,500,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$340,110** /yr
Lower Assiniboine (Section 8.6.1)	**$2,506,894** /yr
City of Winnipeg (Section 8.6.3)	**-$590,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $3,205,000 /yr
(1% of Capital Cost)

Present Value of Costs $388,700,494
(In 2014$)

Benefits

Total Average Annual Benefit $2,257,004 /yr

Present Value of Benefits $57,393,230
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.148	**-$331,307,264**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastructure and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Upgrade of Existing Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 35,000 cfs

Summary of Costs

Total Dike Construction Cost ***$557,100,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$799,417** /yr
Lower Assiniboine (Section 8.6.1)	**$6,108,817** /yr
City of Winnipeg (Section 8.6.3)	**-$1,012,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $5,571,000 /yr
(1% of Capital Cost)

Present Value of Costs $675,647,567
(In 2014$)

Benefits

Total Average Annual Benefit $5,896,234 /yr

Present Value of Benefits $149,935,010
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.222	**-$525,712,557**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Major Setbacks to Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 18,000 cfs

Summary of Costs

Total Dike Construction Cost ***$745,700,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$501,492**	/yr
City of Winnipeg (Section 8.6.3)	**-$19,000**	/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $7,457,000 /yr
(1% of Capital Cost)

Present Value of Costs $904,380,526
(In 2014$)

Benefits

Total Average Annual Benefit $482,492 /yr

Present Value of Benefits $12,269,262
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.014	**-$892,111,263**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Major Setbacks to Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 20,000 cfs

Summary of Costs

Total Dike Construction Cost ***$788,600,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$88,880** /yr
Lower Assiniboine (Section 8.6.1)	**$862,635** /yr
City of Winnipeg (Section 8.6.3)	**-$213,000** /yr
	/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $7,886,000 /yr
(1% of Capital Cost)

Present Value of Costs $956,409,390
(In 2014$)

Benefits

Total Average Annual Benefit $738,515 /yr

Present Value of Benefits $18,779,657
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.020	**-$937,629,733**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Major Setbacks to Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 25,000 cfs

Summary of Costs

Total Dike Construction Cost ***$925,900,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$340,110** /yr
Lower Assiniboine (Section 8.6.1)	**$2,506,894** /yr
City of Winnipeg (Section 8.6.3)	**-$590,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $9,259,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,122,926,014
(In 2014$)

Benefits

Total Average Annual Benefit $2,257,004 /yr

Present Value of Benefits $57,393,230
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.051	**-$1,065,532,784**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Major Setbacks to Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 35,000 cfs

Summary of Costs

Total Dike Construction Cost ***$1,234,400,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$799,417** /yr
Lower Assiniboine (Section 8.6.1)	**$6,108,817** /yr
City of Winnipeg (Section 8.6.3)	**-$1,012,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $12,344,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,497,072,979
(In 2014$)

Benefits

Total Average Annual Benefit $5,896,234 /yr

Present Value of Benefits $149,935,010
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.100	**-$1,347,137,970**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Combination of Upgrades and Setbacks to Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 18,000 cfs

Summary of Costs

Total Dike Construction Cost ***$459,400,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$501,492**	/yr
City of Winnipeg (Section 8.6.3)	**-$19,000**	/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $4,594,000 /yr
(1% of Capital Cost)

Present Value of Costs $557,157,588
(In 2014$)

Benefits

Total Average Annual Benefit $482,492 /yr

Present Value of Benefits $12,269,262
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.022	**-$544,888,326**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Date:	10-Jun-15	**By:**	GW
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Improvement Schemes of the Lower Assiniboine Dikes
Combination of Upgrades and Setbacks to Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 20,000 cfs

Summary of Costs

Total Dike Construction Cost ***$496,100,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$88,880** /yr
Lower Assiniboine (Section 8.6.1)	**$862,635** /yr
City of Winnipeg (Section 8.6.3)	**-$213,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $4,961,000 /yr
(1% of Capital Cost)

Present Value of Costs $601,667,130
(In 2014$)

Benefits

Total Average Annual Benefit $738,515 /yr

Present Value of Benefits $18,779,657
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.031	**-$582,887,473**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Combination of Upgrades and Setbacks to Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 25,000 cfs

Summary of Costs

Total Dike Construction Cost ***$640,700,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$340,110** /yr
Lower Assiniboine (Section 8.6.1)	**$2,506,894** /yr
City of Winnipeg (Section 8.6.3)	**-$590,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $6,407,000 /yr
(1% of Capital Cost)

Present Value of Costs $777,037,150
(In 2014$)

Benefits

Total Average Annual Benefit $2,257,004 /yr

Present Value of Benefits $57,393,230
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.074	**-$719,643,920**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes of the Lower Assiniboine Dikes
Combination of Upgrades and Setbacks to Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 35,000 cfs

Summary of Costs

Total Dike Construction Cost ***$906,300,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$799,417** /yr
Lower Assiniboine (Section 8.6.1)	**$6,108,817** /yr
City of Winnipeg (Section 8.6.3)	**-$1,012,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $9,063,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,099,155,250
(In 2014$)

Benefits

Total Average Annual Benefit $5,896,234 /yr

Present Value of Benefits $149,935,010
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.136	**-$949,220,241**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes Downstream of the Lower Assiniboine Dikes
Extension of Dikes to Headingley
SUMMARY OF COSTS AND BENEFITS
Capacity: 18,000 cfs

Summary of Costs

Total Dike Construction Cost ***$114,100,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$148,837**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,141,000 /yr
(1% of Capital Cost)

Present Value of Costs $138,379,801
(In 2014$)

Benefits

Total Average Annual Benefit $148,837 /yr

Present Value of Benefits $3,784,768
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.027	**-$134,595,034**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes Downstream of the Lower Assiniboine Dikes
Extension of Dikes to Headingley
SUMMARY OF COSTS AND BENEFITS
Capacity: 23,100 cfs

Summary of Costs

Total Dike Construction Cost ***$155,000,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$506,610**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,550,000 /yr
(1% of Capital Cost)

Present Value of Costs $187,983,078
(In 2014$)

Benefits

Total Average Annual Benefit $506,610 /yr

Present Value of Benefits $12,882,558
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.069	**-$175,100,520**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes Downstream of the Lower Assiniboine Dikes
Extension of Dikes to Headingley
SUMMARY OF COSTS AND BENEFITS
Capacity: 28,000 cfs

Summary of Costs

Total Dike Construction Cost ***$197,100,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$967,463**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,971,000 /yr
(1% of Capital Cost)

Present Value of Costs $239,041,708
(In 2014$)

Benefits

Total Average Annual Benefit $967,463 /yr

Present Value of Benefits $24,601,563
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.103	**-$214,440,145**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes Downstream of the Lower Assiniboine Dikes

Combination of Individual Flood Proofing, Neighbourhood Dikes and Purchase of Vulnerable Proterties

SUMMARY OF COSTS AND BENEFITS

Capacity: 18,000 cfs

Summary of Costs

Total Dike Construction Cost ***$21,400,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$148,837**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $214,000 /yr
(1% of Capital Cost)

Present Value of Costs $25,953,793
(In 2014$)

Benefits

Total Average Annual Benefit $148,837 /yr

Present Value of Benefits $3,784,768
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.146	**-$22,169,025**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes Downstream of the Lower Assiniboine Dikes

Combination of Individual Flood Proofing, Neighbourhood Dikes and Purchase of Vulnerable Proterties

SUMMARY OF COSTS AND BENEFITS

Capacity: 23,100 cfs

Summary of Costs

Total Dike Construction Cost ***$62,600,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$506,610**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $626,000 /yr
(1% of Capital Cost)

Present Value of Costs $75,920,908
(In 2014$)

Benefits

Total Average Annual Benefit $506,610 /yr

Present Value of Benefits $12,882,558
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.170	**-$63,038,350**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Improvement Schemes Downstream of the Lower Assiniboine Dikes

Combination of Individual Flood Proofing, Neighbourhood Dikes and Purchase of Vulnerable Proterties

SUMMARY OF COSTS AND BENEFITS

Capacity: 28,000 cfs

Summary of Costs

Total Dike Construction Cost ***$122,900,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$967,463**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,229,000 /yr
(1% of Capital Cost)

Present Value of Costs $149,052,389
(In 2014$)

Benefits

Total Average Annual Benefit $967,463 /yr

Present Value of Benefits $24,601,563
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.165	**-$124,450,826**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Wasteway Only
Control Structure at Hoop and Holler Bend on the Assiniboine River
SUMMARY OF COSTS AND BENEFITS
Capacity: 3,900 cfs

Summary of Costs

Total Construction Cost ***$109,600,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$2,705,634**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,096,000 /yr
(1% of Capital Cost)

Present Value of Costs $132,922,228
(In 2014$)

Benefits

Total Average Annual Benefit $2,705,634 /yr

Present Value of Benefits $68,801,418
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.518	**-$64,120,810**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Control Structure with Wasteway

Control Structure at Hoop and Holler Bend on the Assiniboine River

SUMMARY OF COSTS AND BENEFITS

Capacity: 3,900 cfs

Summary of Costs

Total Construction Cost ***$118,900,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$2,705,634**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,189,000 /yr
(1% of Capital Cost)

Present Value of Costs $144,201,213
(In 2014$)

Benefits

Total Average Annual Benefit $2,705,634 /yr

Present Value of Benefits $68,801,418
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.477	**-$75,399,795**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Control Structure with Diversion Channel

Control Structure at Hoop and Holler Bend on the Assiniboine River

SUMMARY OF COSTS AND BENEFITS

Capacity: 3,900 cfs

Summary of Costs

Total Construction Cost ***$386,890,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)		/yr
Lower Assiniboine (Section 8.6.1)	**$2,705,634**	/yr
City of Winnipeg (Section 8.6.3)		/yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $3,868,900 /yr
(1% of Capital Cost)

Present Value of Costs $469,217,891
(In 2014$)

Benefits

Total Average Annual Benefit $2,705,634 /yr

Present Value of Benefits $68,801,418
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.147	**-$400,416,473**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Widening
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft3/s

Summary of Costs

Total Construction Cost	***$614,800,000***
Do Nothing Cost	***$135,500,000***
Total Incremental Cost	***$479,300,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $2,600,000 /yr

Present Value of Costs $534,626,454
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $170,627,661
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.319	**-$363,998,793**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Parallel Channel
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft3/s

Summary of Costs

Total Construction Cost ***$264,900,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $2,600,000 /yr

Present Value of Costs $320,226,454
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $170,627,661
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.533	**-$149,598,793**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
File No:	12-0300-011		
Project:	AR & LMB Basins Flood Mitigation Study		
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Date:	10-Jun-15	**By:**	GW
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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost	*$373,100,000*
Do Nothing Cost	*$135,500,000*
Total Incremental Cost	***$237,600,000***

Summary of Benefits		
	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $252,495,584
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $170,627,661
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.676	**-$81,867,923**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Date:	7-Jul-15	**By:**	GW
Checked:	10-Jul-15	**By:**	PAL

Parallel Channel
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 46,000 ft^3/s

Summary of Costs

Total Construction Cost	*$530,000,000*
Do Nothing Cost	*$135,500,000*
Total Incremental Cost	***$394,500,000***

Summary of Benefits		
	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$1,209,293** /yr
	Lower Assiniboine (Section 8.6.1)	**$9,490,150** /yr
	City of Winnipeg (Section 8.6.3)	**$1,339,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $409,395,584
(In 2014$)

Benefits

Total Average Annual Benefit $9,619,857 /yr

Present Value of Benefits $244,622,814
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.598	**-$164,772,770**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Conceptual Design and Assessment of Mitigation Options for Dauphin Lake
Short Channel, 30 m Weir
SUMMARY OF COSTS AND BENEFITS
Capacity: 4,000 cfs

Summary of Costs

Total Construction Cost ***$143,521,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,435,210 /yr
(1% of Capital Cost)

Present Value of Costs $174,061,415
(In 2014$)

Benefits

Total Average Annual Benefit $1,045,706 /yr
(Section 8.6.4)

Present Value of Benefits $26,591,200
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.153	**-$147,470,215**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Conceptual Design and Assessment of Mitigation Options for Dauphin Lake
Short Channel, 60 m Weir
SUMMARY OF COSTS AND BENEFITS
Capacity: 7,500 cfs

Summary of Costs

Total Dike Construction Cost ***$253,523,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,535,230 /yr
(1% of Capital Cost)

Present Value of Costs $307,471,187
(In 2014$)

Benefits

Total Average Annual Benefit $1,275,887 /yr
(Section 8.6.4)

Present Value of Benefits $32,444,460
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.106	**-$275,026,727**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Date:	12-Jun-15	**By:**	GW
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Conceptual Design and Assessment of Mitigation Options for Dauphin Lake
East Channel, 30 m Weir
SUMMARY OF COSTS AND BENEFITS
Capacity: 4,000 cfs

Summary of Costs

Total Construction Cost ***$211,391,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,113,910 /yr
(1% of Capital Cost)

Present Value of Costs $256,373,748
(In 2014$)

Benefits

Total Average Annual Benefit $1,045,706 /yr
(Section 8.6.4)

Present Value of Benefits $26,591,200
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.104	**-$229,782,547**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Conceptual Design and Assessment of Mitigation Options for Dauphin Lake
West Channel, 30 m Weir
SUMMARY OF COSTS AND BENEFITS
Capacity: 4,000 cfs

Summary of Costs

Total Construction Cost ***$322,763,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $3,227,630 /yr
(1% of Capital Cost)

Present Value of Costs $391,445,047
(In 2014$)

Benefits

Total Average Annual Benefit $1,045,706 /yr
(Section 8.6.4)

Present Value of Benefits $26,591,200
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.068	**-$364,853,847**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
File No:	12-0300-011		
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Conceptual Design and Assessment of Mitigation Options for Dauphin Lake

Excavate Mossy River 0.3

SUMMARY OF COSTS AND BENEFITS

Capacity: 4,250 cfs

Summary of Costs

Total Construction Cost ***$31,590,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $315,900 /yr
(1% of Capital Cost)

Present Value of Costs $38,312,164
(In 2014$)

Benefits

Total Average Annual Benefit $571,471 /yr
(Section 8.6.4)

Present Value of Benefits $14,531,905
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.379	**-$23,780,260**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Conceptual Design and Assessment of Mitigation Options for Dauphin Lake
Excavate Mossy River 0.5
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Total Construction Cost ***$52,650,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $526,500 /yr
(1% of Capital Cost)

Present Value of Costs $63,853,607
(In 2014$)

Benefits

Total Average Annual Benefit $837,361 /yr
(Section 8.6.4)

Present Value of Benefits $21,293,207
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.333	**-$42,560,400**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Conceptual Design and Assessment of Mitigation Options for Dauphin Lake
Excavate Mossy River 1.0 m
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Total Construction Cost ***$105,300,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,053,000 /yr
(1% of Capital Cost)

Present Value of Costs $127,707,214
(In 2014$)

Benefits

Total Average Annual Benefit $1,213,075 /yr
(Section 8.6.4)

Present Value of Benefits $30,847,217
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.242	**-$96,859,996**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastructure and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Winnipegosis Diversion
Lake Winnipegosis Diversion to Cedar Lake
SUMMARY OF COSTS AND BENEFITS
Capacity: 2,500 cfs

Summary of Costs

Total Construction Cost ***$160,188,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$2,059,342** /yr
Other Areas (Section 14.2.4)	**$275,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $1,601,880 /yr
(1% of Capital Cost)

Present Value of Costs $194,275,054
(In 2014$)

Benefits

Total Average Annual Benefit $2,334,342 /yr

Present Value of Benefits $59,359,854
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.306	**-$134,915,200**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Winnipegosis Diversion
Lake Winnipegosis Diversion to Cedar Lake
SUMMARY OF COSTS AND BENEFITS
Capacity: 5,000 cfs

Summary of Costs

Total Construction Cost ***$215,178,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$3,470,250** /yr
Other Areas (Section 14.2.4)	**$550,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $2,151,780 /yr
(1% of Capital Cost)

Present Value of Costs $260,966,599
(In 2014$)

Benefits

Average Annual Benefit $4,020,250 /yr

Present Value of Benefits $102,230,716
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.392	**-$158,735,883**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Winnipegosis Diversion
Lake Winnipegosis Diversion to Cedar Lake
SUMMARY OF COSTS AND BENEFITS
Capacity: 10,000 cfs

Summary of Costs

Total Construction Cost ***$325,158,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$4,618,696** /yr
Other Areas (Section 14.2.4)	**$1,100,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,251,580 /yr
(1% of Capital Cost)

Present Value of Costs $394,349,689
(In 2014$)

Benefits

Average Annual Benefit $5,718,696 /yr

Present Value of Benefits $145,420,406
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.369	**-$248,929,283**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
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Project: AR & LMB Basins Flood Mitigation Study

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Lake Winnipegosis Diversion
Lake Winnipegosis Diversion to Lake Winnipeg
SUMMARY OF COSTS AND BENEFITS
Capacity: 10,000 cfs

Summary of Costs

Total Construction Cost ***$1,333,960,000***

Summary of Benefits Lake Manitoba & Lake St. Martin (Section 8.6.2) **$4,618,696** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $13,339,600 /yr
(1% of Capital Cost)

Present Value of Costs $1,617,818,755
(In 2014$)

Benefits

Average Annual Benefit $4,618,696 /yr

Present Value of Benefits $98,283,104
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.061	**-$1,519,535,651**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastructure and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Winnipegosis Outlet
Control Structure on the Waterhen River
SUMMARY OF COSTS AND BENEFITS
Capacity: 6,000 cfs

Summary of Costs

Total Construction Cost ***$57,722,000***

Summary of Benefits Lake Manitoba & Lake St. Martin (Section 8.6.2) **$2,577,244** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $577,220 /yr
(1% of Capital Cost)

Present Value of Costs $70,004,898
(In 2014$)

Benefits

Average Annual Benefit $2,577,244 /yr

Present Value of Benefits $65,536,596
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.936	**-$4,468,303**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option C1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option C - Channel Slightly Less South of Pinaymootang FN - 5,000 cfs		$167,700,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$335,300,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$106,500,000

Total Incremental Cost ***$228,800,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,288,000 /yr
(1% of Capital Cost)

Present Value of Costs $277,487,279
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $128,212,969
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.462	**-$149,274,310**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option C2
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option C - Channel Slightly Less South of Pinaymootang FN - 10,000 cfs		$291,700,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 14,000 cfs		$240,900,000
	Sub-total	$532,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$106,500,000

Total Incremental Cost ***$426,100,000***

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $4,261,000 /yr
(1% of Capital Cost)

Present Value of Costs $516,771,546
(In 2014$)

Benefits

Average Annual Benefit $8,309,283 /yr
(Section 8.6.2)

Present Value of Benefits $211,296,300
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.409	**-$305,475,246**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option C3
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option C - Channel Slightly Less South of Pinaymootang FN - 15,000 cfs		$415,200,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 19,000 cfs		$306,600,000
	Sub-total	$721,800,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$106,500,000

Total Incremental Cost ***$615,300,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	75	yrs
Growth	1	percent

Costs

Annual O & M Cost $6,153,000 /yr
(1% of Capital Cost)

Present Value of Costs $746,232,181
(In 2014$)

Benefits

Average Annual Benefit $9,149,523 /yr
(Section 8.6.2)

Present Value of Benefits $232,662,716
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.312	**-$513,569,465**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $257,476,178
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $128,212,969
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.498	**-$129,263,208**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D2
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 10,000 cfs		$259,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 14,000 cfs		$240,900,000
	Sub-total	$499,900,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$386,600,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $3,866,000 /yr
(1% of Capital Cost)

Present Value of Costs $468,866,181
(In 2014$)

Benefits

Average Annual Benefit $8,309,283 /yr
(Section 8.6.2)

Present Value of Benefits $211,296,300
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.451	**-$257,569,881**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D3
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 15,000 cfs		$360,300,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 19,000 cfs		$306,600,000
	Sub-total	$666,900,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$553,600,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $5,536,000 /yr
(1% of Capital Cost)

Present Value of Costs $671,402,788
(In 2014$)

Benefits

Average Annual Benefit $9,149,523 /yr
(Section 8.6.2)

Present Value of Benefits $232,662,716
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.347	**-$438,740,071**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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21 Small Dams
Multitude of Small Dams
SUMMARY OF COSTS AND BENEFITS
Capacity: 200,000 dam^3

Summary of Costs

Total Dike Construction Cost ***$482,200,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$3,751,762** /yr
City of Winnipeg (Section 8.6.3)	**$601,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $4,822,000 /yr
(1% of Capital Cost)

Present Value of Costs $584,809,292
(In 2014$)

Benefits

Total Average Annual Benefit $5,554,213 /yr
(Section 8.6.2)

Present Value of Benefits $141,237,776
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.242	**-$443,571,516**

EVALUATION OF COMBINATION OF OPTIONS

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Assiniboine River and Lake Manitoba Basins Flood Mitigation Study
Project No. 12-0300-011

Summary List of Flood Mitigation Alternatives - Combination of Options

Concepts	Options Considered	Level of Protection on Lower Assiniboine River (years)	1:200 Year Wind Eliminated Water Level on Lake Manitoba (ft)	Combined Capital Cost (Million $)	Combined Average Annual Benefits (Million $)			Combined B/C Ratio	Comments
					Lake Manitoba & Lake St. Martin	Lower Assiniboine	City of Winnipeg		
34,000 cfs Portage Diversion Retrofit and 5,000 cfs LMB Outlet Channel Option D Combined with:	20,000 cfs Lower Assiniboine Dikes	160	815.8	645 (1)(2)	5.13	6.85	0.82	0.42	
	25,000 cfs Lower Assiniboine Dikes	230	815.8	758 (1)(2)	5.38	7.68	0.49	0.40	
34,000 cfs Portage Diversion Retrofit and 10,000 cfs LMB Outlet Channel Option D Combined with:	20,000 cfs Lower Assiniboine Dikes	160	815.2	819 (1)(2)	8.40	6.85	0.82	0.45	
	25,000 cfs Lower Assiniboine Dikes	230	815.2	945 (1)(2)	8.65	7.68	0.49	0.41	
34,000 cfs Portage Diversion Retrofit Combined with:	20,000 cfs Lower Assiniboine Dikes	160	816.5	435 (2)	-0.64	6.85	0.82	0.37	
	25,000 cfs Lower Assiniboine Dikes	230	816.5	558 (2)	-0.36	7.68	0.49	0.31	
	35,000 cfs Lower Assiniboine Dikes	450	816.4	795 (2)	0.14	9.45	0.11	0.27	
	Alexander Dam	450	816.0	828 (2)	1.82	11.73	2.37	0.42 (3)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
	Holland Dam	170	816.4	544 (2)	0.55	9.16	1.75	0.47 (3)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
Alexander Dam Combined with:	20,000 cfs Lower Assiniboine Dikes	600	815.8	785	2.45	10.34	1.18	0.38 (3)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
	25,000 cfs Lower Assiniboine Dikes	800	815.8	911	2.76	10.79	0.81	0.33 (3)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
	35,000 cfs Lower Assiniboine Dikes	>1000	815.8	1148	3.22	11.52	0.48	0.28 (3)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
Holland Dam Combined with:	20,000 cfs Lower Assiniboine Dikes	250	816.2	501	1.29	5.57	0.56	0.31 (3)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
	25,000 cfs Lower Assiniboine Dikes	370	816.2	627	1.54	6.60	0.19	0.28 (3)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.
	35,000 cfs Lower Assiniboine Dikes	700	816.1	863	1.99	9.08	-0.14	0.27 (3)	Physical, biological and socio-economic impacts due to formation of reservoir and construction of new dam.

Notes:
1. The cost of making the existing LSM Outlet channels (base condition) permanent was subtracted from the Capital Cost of LMB & LSM alternatives.
2. The Portage Diversion cost of "Do Nothing" was subtracted from the Capital Cost of the option.
3. Potential significant environmental impacts. Not identified as a preferred alternative.

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Combination of Options

Increasing Capacity of the Portage Diversion and Development of Upstream Storage

SUMMARY OF COSTS AND BENEFITS

Capacity: 34,000 cfs Portage Diversion Retrofit and Alexander Dam 1,600,000 dam^3

Summary of Costs

Alexander Dam Construction Cost		*$590,600,000*
Portage Diversion Retrofit Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Total Construction Cost		***$828,200,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,820,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$11,730,000** /yr
	City of Winnipeg (Section 8.6.3)	**$2,370,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost	
Alexander Dam (1%)	$5,906,000 /yr
Portage Diversion	$700,000 /yr
Present Value of Costs (In 2014$)	$968,771,752

Benefits

Total Average Annual Benefit	$15,920,000 /yr
Present Value of Benefits (In 2014$)	$404,828,803

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.418	**-$563,942,948**

KGS GROUP CONSULTING ENGINEERS

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Combination of Options
Increasing Capacity of the Portage Diversion and Development of Upstream Storage
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 cfs Portage Diversion Retrofit and Holland Dam 800,000 dam^3

Summary of Costs

Holland Dam Construction Cost		*$306,000,000*
Portage Diversion Retrofit Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Total Construction Cost		***$543,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$550,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$9,160,000** /yr
	City of Winnipeg (Section 8.6.3)	**$1,750,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost

Holland Dam (1%)	$3,060,000 /yr
Portage Diversion	$700,000 /yr

Present Value of Costs (In 2014$) $623,610,564

Benefits

Total Average Annual Benefit $11,460,000 /yr

Present Value of Benefits (In 2014$) $291,415,709

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.467	**-$332,194,855**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options
Increasing Capacity of the Portage Diversion and Improvement to the Lower Assiniboine Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 cfs Portage Diversion Retrofit and 20,000 cfs Lower Assiniboine Dikes

Summary of Costs

Dike Construction Cost		*$194,600,000*
Portage Diversion Retrofit Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Total Construction Cost		***$432,200,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$640,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,850,000** /yr
	City of Winnipeg (Section 8.6.3)	**$820,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost

Assiniboine Dikes	$1,946,000 /yr
Portage Diversion (1%)	$700,000 /yr

Present Value of Costs $488,505,307
(In 2014$)

Benefits

Total Average Annual Benefit $7,030,000 /yr

Present Value of Benefits $178,765,483
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.366	**-$309,739,824**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options
Increasing Capacity of the Portage Diversion and Improvement to the Lower Assiniboine Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 cfs Portage Diversion Retrofit and 25,000 cfs Lower Assiniboine Dikes

Summary of Costs

Total Dike Construction Cost		*$320,500,000*
Portage Diversion Retrofit Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Total Construction Cost		***$558,100,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$360,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$7,680,000** /yr
	City of Winnipeg (Section 8.6.3)	**$490,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost	
Assiniboine Dike (1%)	$3,205,000 /yr
Portage Diversion	$700,000 /yr
Present Value of Costs (In 2014$)	$641,196,078

Benefits

Total Average Annual Benefit	$7,810,000 /yr
Present Value of Benefits (In 2014$)	$198,600,060

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.310	**-$442,596,018**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Capacity of the Portage Diversion and Improvement to the Lower Assiniboine Dikes

SUMMARY OF COSTS AND BENEFITS

Capacity: 34,000 cfsPortage Diversion Retrofit and 35,000 cfs Lower Assiniboine Dikes

Summary of Costs

Dike Construction Cost		*$557,100,000*
Portage Diversion Retrofit Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Total Construction Cost		***$794,700,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$140,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$9,450,000** /yr
	City of Winnipeg (Section 8.6.3)	**$110,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost	
Assiniboine Dikes (1%)	$5,571,000 /yr
Portage Diversion	$700,000 /yr
Present Value of Costs (In 2014$)	$928,143,151

Benefits

Total Average Annual Benefit	$9,700,000 /yr
Present Value of Benefits (In 2014$)	$246,660,766

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.266	**-$681,482,385**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Storage on the Assiniboine River and Improvement to the Lower Assiniboine Dikes

SUMMARY OF COSTS AND BENEFITS

Capacity: 1,600,000 dam^3 Alexander Dam and 20,000 cfs Lower Assiniboine Dikes

Summary of Costs

Lower Assiniboine Dike Construction Cost	*$194,600,000*
Alexander Dam Construction Cost	*$590,600,000*
Total Construction Cost	***$785,200,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$2,540,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$10,340,000** /yr
	City of Winnipeg (Section 8.6.3)	**$1,180,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $7,852,000 /yr
(1% of Capital Cost)

Present Value of Costs $952,285,891
(In 2014$)

Benefits

Total Average Annual Benefit $14,060,000 /yr

Present Value of Benefits $357,530,966
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.375	**-$594,754,925**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Storage on the Assiniboine River and Improvement to the Lower Assiniboine Dikes

SUMMARY OF COSTS AND BENEFITS

Capacity: 1,600,000 dam^3 Alexander Dam and 25,000 cfs Lower Assiniboine Dikes

Summary of Costs

Lower Assiniboine Dike Construction Cost	*$320,500,000*
Alexander Dam Construction Cost	*$590,600,000*
Total Construction Cost	***$911,100,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$2,760,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$10,790,000** /yr
	City of Winnipeg (Section 8.6.3)	**$810,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $9,111,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,104,976,662
(In 2014$)

Benefits

Total Average Annual Benefit $14,360,000 /yr

Present Value of Benefits $365,159,649
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.330	**-$739,817,013**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options
Increasing Storage on the Assiniboine River and Improvement to the Lower Assiniboine Dikes
SUMMARY OF COSTS AND BENEFITS
Capacity: 1,600,000 dam^3 Alexander Dam and 35,000 cfs Lower Assiniboine Dikes

Summary of Costs

Lower Assiniboine Dike Construction Cost	*$557,100,000*
Alexander Dam Construction Cost	*$590,600,000*
Total Construction Cost	***$1,147,700,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$3,220,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$11,520,000** /yr
	City of Winnipeg (Section 8.6.3)	**$480,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $11,477,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,391,923,735
(In 2014$)

Benefits

Total Average Annual Benefit $15,220,000 /yr

Present Value of Benefits $387,028,542
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.278	**-$1,004,895,193**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Storage on the Assiniboine River and Improvement to the Lower Assiniboine Dikes

SUMMARY OF COSTS AND BENEFITS

Capacity: 800,000 dam^3 Holland Dam and 20,000 cfs Lower Assiniboine Dikes

Summary of Costs

Lower Assiniboine Dike Construction Cost	*$194,600,000*
Holland Dam Construction Cost	*$306,000,000*
Total Construction Cost	***$500,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,290,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$5,570,000** /yr
	City of Winnipeg (Section 8.6.3)	**$560,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $5,006,000 /yr
(1% Capital Cost)

Present Value of Costs $607,124,703
(In 2014$)

Benefits

Total Average Annual Benefit $7,420,000 /yr

Present Value of Benefits $188,682,771
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.311	**-$418,441,932**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Storage on the Assiniboine River and Improvement to the Lower Assiniboine Dikes

SUMMARY OF COSTS AND BENEFITS

Capacity: 800,000 dam^3 Holland Dam and 25,000 cfs Lower Assiniboine Dikes

Summary of Costs

Lower Assiniboine Dike Construction Cost	*$320,500,000*
Holland Dam Construction Cost	*$306,000,000*
Total Construction Cost	***$626,500,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,540,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,600,000** /yr
	City of Winnipeg (Section 8.6.3)	**$190,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $6,265,000 /yr
(1% of Capital Cost)

Present Value of Costs $759,815,474
(In 2014$)

Benefits

Total Average Annual Benefit $8,330,000 /yr

Present Value of Benefits $211,823,111
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.279	**-$547,992,363**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Storage on the Assiniboine River and Improvement to the Lower Assiniboine Dikes

SUMMARY OF COSTS AND BENEFITS

Capacity: 800,000 dam^3 Holland Dam and 35,000 cfs Lower Assiniboine Dikes

Summary of Costs

Lower Assiniboine Dike Construction Cost	*$557,100,000*
Holland Dam Construction Cost	*$306,000,000*
Total Construction Cost	***$863,100,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,990,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$9,080,000** /yr
	City of Winnipeg (Section 8.6.3)	**-$140,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $8,631,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,046,762,547
(In 2014$)

Benefits

Total Average Annual Benefit $10,930,000 /yr

Present Value of Benefits $277,938,368
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.266	**-$768,824,179**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Capacity of the Portage Diversion and the Assiniboine Dikes with LMB & LSM Outlet Channels

SUMMARY OF COSTS AND BENEFITS

Capacity: 34,000 cfs Portage Diversion Retrofit Diversion Channel, 20,000 cfs Assiniboine Dikes, 5,000 cfs D1 Outlet Channel

Summary of Costs

Lower Assiniboine Dike Upgrades		*$194,600,000*
Portage Diversion Retrofit Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Lake Manitoba Outlet Channel D1		*$325,600,000*
Lake Manitoba Outlet Base Case	-	*$113,300,000*
Total Construction Cost		***$644,500,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$5,130,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,850,000** /yr
	City of Winnipeg (Section 8.6.3)	**$820,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost

Assiniboine Dikes (1%)	$1,946,000 /yr
Portage Diversion	$700,000 /yr
Outlet Channel (1%)	$3,256,000 /yr

Present Value of Costs (In 2014$) $770,091,050

Benefits

Total Average Annual Benefit $12,800,000 /yr

Present Value of Benefits (In 2014$) $325,490,495

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.423	**-$444,600,555**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Capacity of the Portage Diversion and the Assiniboine Dikes with LMB & LSM Outlet Channels

SUMMARY OF COSTS AND BENEFITS

Capacity: 34,000 cfs Portage Diversion Retrofit Diversion Channel, 25,000 cfs Assiniboine Dikes, 5,000 cfs D1 Outlet Channel

Summary of Costs

Lower Assiniboine Dike Upgrades		*$320,500,000*
Portage Diversion Retrofit Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Lake Manitoba Outlet Channel D1		*$325,600,000*
Lake Manitoba Outlet Base Case	-	*$125,300,000*
Total Construction Cost		***$758,400,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$5,380,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$7,680,000** /yr
	City of Winnipeg (Section 8.6.3)	**$490,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost

Assiniboine Dikes (1%)	$3,205,000 /yr
Portage Diversion	$700,000 /yr
Outlet Channel (1%)	$1,355,000 /yr

Present Value of Costs (In 2014$) $870,329,672

Benefits

Total Average Annual Benefit $13,550,000 /yr

Present Value of Benefits (In 2014$) $344,562,204

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.396	**-$525,767,468**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Capacity of the Portage Diversion and the Assiniboine Dikes with LMB & LSM Outlet Channels

SUMMARY OF COSTS AND BENEFITS

Capacity: 34,000 cfs Portage Diversion Retrofit Diversion Channel, 20,000 cfs Assiniboine Dikes, 10,000 cfs D2 Outlet Channel

Summary of Costs

Lower Assiniboine Dike Upgrades		*$194,600,000*
Portage Diversion Retrofit Diversion Channel Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Lake Manitoba Outlet Channel D2		*$499,900,000*
Lake Manitoba Outlet Base Case	-	*$113,300,000*
Total Construction Cost		***$818,800,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$8,400,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,850,000** /yr
	City of Winnipeg (Section 8.6.3)	**$820,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost

Assiniboine Dikes (1%)	$1,946,000 /yr
Portage Diversion	$700,000 /yr
Outlet Channel (1%)	$1,355,000 /yr

Present Value of Costs (In 2014$) $903,938,901

Benefits

Total Average Annual Benefit $16,070,000 /yr

Present Value of Benefits (In 2014$) $408,643,145

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.452	**-$495,295,756**

KGS GROUP CONSULTING ENGINEERS

Client:	Manitoba Infrastracture and Transportation		
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Combination of Options

Increasing Capacity of the Portage Diversion and the Assiniboine Dikes with LMB & LSM Outlet Channels

SUMMARY OF COSTS AND BENEFITS

Capacity: 34,000 cfs Portage Diversion Retrofit Diversion Channel, 25,000 cfs Assiniboine Dikes, 10,000 cfs D2 Outlet Channel

Summary of Costs

Lower Assiniboine Dike Upgrades		*$320,500,000*
Portage Diversion Retrofit Diversion Channel Construction Cost		*$373,100,000*
Portage Diversion Do Nothing Cost	-	*$135,500,000*
Lake Manitoba Outlet Channel D2		*$499,900,000*
Lake Manitoba Outlet Base Case	-	*$113,300,000*
Total Construction Cost		***$944,700,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$8,650,000** /yr
	Lower Assiniboine (Section 8.6.1)	**$7,680,000** /yr
	City of Winnipeg (Section 8.6.3)	**$490,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost

Assiniboine Dikes (1%)	$3,205,000 /yr
Portage Diversion	$700,000 /yr
Outlet Channel (1%)	$1,355,000 /yr

Present Value of Costs (In 2014$) $1,056,629,672

Benefits

Total Average Annual Benefit $16,820,000 /yr

Present Value of Benefits (In 2014$) $427,714,854

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.405	**-$628,914,818**

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SENSITIVITY TO ECONOMIC PARAMETERS

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Portage Diversion Retrofit – 34,000 cfs

SENSITIVITY OF BENEFIT/COST RATIO TO INTEREST RATE

Interest	**5.0%**	**7.0%**	**9.0%**
Inflation	3.0%	3.0%	3.0%
Growth	1.0%	1.0%	1.0%
Economic Life	50 yr	50 yr	50 yr
B/C Ratio	0.99	0.68	0.49

SENSITIVITY OF BENEFIT/COST RATIO TO INFLATION RATE

Interest	7.0%	7.0%	7.0%
Inflation	**2.0%**	**3.0%**	**4.0%**
Growth	1.0%	1.0%	1.0%
Economic Life	50 yr	50 yr	50 yr
B/C Ratio	0.57	0.68	0.81

SENSITIVITY OF BENEFIT/COST RATIO TO GROWTH RATE

Interest	7.0%	7.0%
Inflation	3.0%	3.0%
Growth	**0.0%**	**1.0%**
Economic Life	50 yr	50 yr
B/C Ratio	0.57	0.68

SENSITIVITY OF BENEFIT/COST RATIO TO ECONOMIC LIFE

Interest	7.0%	7.0%	7.0%
Inflation	3.0%	3.0%	3.0%
Growth	1.0%	1.0%	1.0%
Economic Life	**40 yr**	**50 yr**	**75 yr**
B/C Ratio	0.61	0.68	0.79

Holland Dam – 880,000 dam^3

SENSITIVITY OF BENEFIT/COST RATIO TO INTEREST RATE

Interest	**5.0%**	**7.0%**	**9.0%**
Inflation	3.0%	3.0%	3.0%
Growth	1.0%	1.0%	1.0%
Economic Life	50 yr	50 yr	50 yr
B/C Ratio	0.68	0.49	0.37

SENSITIVITY OF BENEFIT/COST RATIO TO INFLATION RATE

Interest	7.0%	7.0%	7.0%
Inflation	**2.0%**	**3.0%**	**4.0%**
Growth	1.0%	1.0%	1.0%
Economic Life	50 yr	50 yr	50 yr
B/C Ratio	0.42	0.49	0.57

SENSITIVITY OF BENEFIT/COST RATIO TO GROWTH RATE

Interest	7.0%	7.0%
Inflation	3.0%	3.0%
Growth	**0.0%**	**1.0%**
Economic Life	50 yr	50 yr
B/C Ratio	0.41	0.49

SENSITIVITY OF BENEFIT/COST RATIO TO ECONOMIC LIFE

Interest	7.0%	7.0%	7.0%
Inflation	3.0%	3.0%	3.0%
Growth	1.0%	1.0%	1.0%
Economic Life	**40 yr**	**50 yr**	**75 yr**
B/C Ratio	0.44	0.49	0.57

Lake Manitoba and Lake St. Martin Outlet Channel – Option D1

SENSITIVITY OF BENEFIT/COST RATIO TO INTEREST RATE

Interest	**5.0%**	**7.0%**	**9.0%**
Inflation	3.0%	3.0%	3.0%
Growth	1.0%	1.0%	1.0%
Economic Life	50 yr	50 yr	50 yr
B/C Ratio	0.70	0.50	0.37

SENSITIVITY OF BENEFIT/COST RATIO TO INFLATION RATE

Interest	7.0%	7.0%	7.0%
Inflation	**2.0%**	**3.0%**	**4.0%**
Growth	1.0%	1.0%	1.0%
Economic Life	50 yr	50 yr	50 yr
B/C Ratio	0.43	0.50	0.58

SENSITIVITY OF BENEFIT/COST RATIO TO GROWTH RATE

Interest	7.0%	7.0%
Inflation	3.0%	3.0%
Growth	**0.0%**	**1.0%**
Economic Life	50 yr	50 yr
B/C Ratio	0.42	0.50

SENSITIVITY OF BENEFIT/COST RATIO TO ECONOMIC LIFE

Interest	7.0%	7.0%	7.0%
Inflation	3.0%	3.0%	3.0%
Growth	1.0%	1.0%	1.0%
Economic Life	**40 yr**	**50 yr**	**75 yr**
B/C Ratio	0.45	0.50	0.58

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KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	5	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $259,219,747
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $256,825,349
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.991	**-$2,394,398**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	9	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $248,578,833
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $121,918,711
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.490	**-$126,660,123**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	2	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $250,320,773
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $143,257,091
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.572	**-$107,063,682**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	4	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $255,304,027
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $206,510,783
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.809	**-$48,793,245**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	0	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $252,495,584
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $142,784,342
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.565	**-$109,711,242**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	40	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $252,495,584
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $152,649,906
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.605	**-$99,845,677**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	75	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $252,495,584
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $198,567,698
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.786	**-$53,927,886**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost	*$343,200,000*
Do Nothing Cost	*$135,500,000*
Total Incremental Cost	***$207,700,000***

Summary of Benefits		
	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $222,595,584
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $170,627,661
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.767	**-$51,967,923**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	0 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $371,140,449
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $151,351,962
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.408	**-$219,788,487**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	2 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $361,632,765
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $151,853,078
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.420	**-$209,779,687**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	4 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $383,418,202
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $218,902,240
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.571	**-$164,515,962**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	5 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $400,536,666
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $272,235,877
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.680	**-$128,300,789**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	9 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $354,017,480
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $129,234,312
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.365	**-$224,783,168**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastructure and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	40 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $371,140,449
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $161,809,500
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.436	**-$209,330,949**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	75 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $371,140,449
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $210,482,539
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.567	**-$160,657,910**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastructure and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $371,140,449
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $180,865,990
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.487	**-$190,274,459**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	0	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $257,476,178
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $107,290,953
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.417	**-$150,185,224**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

Sheet:	1	**of**	1
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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	7	percent
Inflation	2	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $250,880,286
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $107,646,186
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.429	**-$143,234,100**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

Sheet:	1	**of**	1
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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	7	percent
Inflation	4	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $265,993,785
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $155,176,250
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.583	**-$110,817,535**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	5	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $277,869,604
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $192,983,601
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.695	**-$84,886,003**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	9	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $245,597,234
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $91,612,109
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.373	**-$153,985,124**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	40	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $257,476,178
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $114,704,132
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.445	**-$142,772,046**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	75	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $257,476,178
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $149,207,661
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.580	**-$108,268,517**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Lake Manitoba and Lake St. Martin Outlet Channel Construction
Channel Option D1
SUMMARY OF COSTS AND BENEFITS

Summary of Costs

Proposed Channel Costs

Option D - Channel Following Birch Creek - 5,000 cfs		$158,000,000
Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 9,000 cfs		$167,600,000
	Sub-total	$325,600,000

Cost of Base Condition

Option R123-JB - Reach 1,2 and 3 to Johnson Beach - 4,000 cfs	$113,300,000

Total Incremental Cost ***$212,300,000***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $2,123,000 /yr
(1% of Capital Cost)

Present Value of Costs $257,476,178
(In 2014$)

Benefits

Average Annual Benefit $5,042,009 /yr
(Section 8.6.2)

Present Value of Benefits $128,212,969
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.498	**-$129,263,208**

SENSITIVITY TO PERIOD OF RECORD

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Increase Capacity of Portage Diversion:
Sensitivity Analyses of Changes to Period of Record to the Lower Assiniboine River Benefits

Capacity of Portage Diversion (cfs)	Estimated Cost of Upgrades (Millions)	Long Period of Record (1882-2011)				Short Period of Record (1971-2011)			
		FPL on Lower Assiniboine River	Average Annual Damages (Millions)	Average Annual Benefits (Millions)	B/C Ratio	FPL on Lower Assiniboine River	Average Annual Damages (Millions)	Average Annual Benefits (Millions)	B/C
25,000	n/a	55 year	$15.2	n/a	n/a	25 year	$52.7	n/a	n/a
34,000	$373.1	110 year	$8.8	$6.4	0.68	40 year	$37.1	$15.6	1.60
46,000	$540.0	260 year	$5.7	$9.5	0.58	70 year	$26.6	$26.1	1.59

Developing Storage on the Assiniboine River and Tributaries – Holland Dam:
Sensitivity Analyses of Changes to Period of Record to the Lower Assiniboine River Benefits

Capacity of Holland Dam (dam^3)	Estimated Cost (Millions)	Long Period of Record (1882-2011)				Short Period of Record (1971-2011)			
		FPL on Lower Assiniboine River	Average Annual Damages (Millions)	Average Annual Benefits (Millions)	B/C Ratio	FPL on Lower Assiniboine River	Average Annual Damages (Millions)	Average Annual Benefits (Millions)	B/C
n/a	n/a	55 year	$15.2	n/a	n/a	25 year	$52.7	n/a	n/a
888,000	$306.0	90 year	$10.0	$5.1	0.49	35 year	$40.1	$12.6	1.00

Increase Capacity of Melita Dikes on the Souris River:
Sensitivity Analyses of Changes to Period of Record to the Souris River Benefits

Capacity of Melita Dikes (cfs)	Estimated Cost of Upgrades (Millions)	Long Period of Record (1882-2011)			Short Period of Record (1971-2011)		
		FPL on Souris River	Average Annual Benefits	B/C Ratio	FPL on Souris River	Average Annual Benefits	B/C Ratio
27,200	n/a	180 year	n/a	n/a	90 year	n/a	n/a
27,900	$0.2	200 year	$1,400	0.16	100 year	$2,100	0.25
39,400	$1.3	500 year	$11,200	0.18	350 year	$28,100	0.45

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KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam - 40 Year Period of Record
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$12,551,390** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $371,140,449
(In 2014$)

Benefits

Total Average Annual Benefit $14,536,841 /yr

Present Value of Benefits $369,656,529
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.996	**-$1,483,920**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastructure and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Developing Storage on the Assiniboine River and Tributaries
Holland Dam
SUMMARY OF COSTS AND BENEFITS
Capacity: 880,000 dam^3

Summary of Costs

Total Construction Cost ***$306,021,000***

Summary of Benefits

Lake Manitoba & Lake St. Martin (Section 8.6.2)	**$1,201,451** /yr
Lower Assiniboine (Section 8.6.1)	**$5,127,152** /yr
City of Winnipeg (Section 8.6.3)	**$784,000** /yr

Present Valuation

Interest	7 percent
Inflation	3 percent
Economic Life	50 yrs
Growth	1 percent

Costs

Annual O & M Cost $3,060,210 /yr
(1% of Capital Cost)

Present Value of Costs $371,140,449
(In 2014$)

Benefits

Total Average Annual Benefit $7,112,603 /yr

Present Value of Benefits $180,865,990
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.487	**-$190,274,459**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Increasing Capacity of Portage Diversion
Channel Retrofit - 40 year Period of Record
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 cfs

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$15,608,712** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $252,495,584
(In 2014$)

Benefits

Total Average Annual Benefit $15,897,452 /yr

Present Value of Benefits $404,255,431
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
1.601	**$151,759,848**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Channel Retrofit
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 34,000 ft^3/s

Summary of Costs

Total Construction Cost		*$373,100,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$237,600,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$732,260** /yr
	Lower Assiniboine (Section 8.6.1)	**$6,421,238** /yr
	City of Winnipeg (Section 8.6.3)	**$1,021,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $252,495,584
(In 2014$)

Benefits

Total Average Annual Benefit $6,709,978 /yr

Present Value of Benefits $170,627,661
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.676	**-$81,867,923**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Increasing Capacity of Portage Diversion
Channel Retrofit - 40 year Period of Record
SUMMARY OF COSTS AND BENEFITS
Capacity: 46,000 cfs

Summary of Costs

Total Construction Cost		*$540,000,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$404,500,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$1,209,293** /yr
	Lower Assiniboine (Section 8.6.1)	**$26,128,402** /yr
	City of Winnipeg (Section 8.6.3)	**$1,339,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $419,395,584
(In 2014$)

Benefits

Total Average Annual Benefit $26,258,109 /yr

Present Value of Benefits $667,716,008
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
1.592	**$248,320,424**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Parallel Channel
Increasing Capacity of Portage Diversion
SUMMARY OF COSTS AND BENEFITS
Capacity: 46,000 ft^3/s

Summary of Costs

Total Construction Cost		*$540,000,000*
Do Nothing Cost	-	*$135,500,000*
Total Incremental Cost		***$404,500,000***

Summary of Benefits	Lake Manitoba & Lake St. Martin (Section 8.6.2)	**-$1,209,293** /yr
	Lower Assiniboine (Section 8.6.1)	**$9,490,150** /yr
	City of Winnipeg (Section 8.6.3)	**$1,339,000** /yr

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Incremental Annual O & M $700,000 /yr

Present Value of Costs $419,395,584
(In 2014$)

Benefits

Total Average Annual Benefit $9,619,857 /yr

Present Value of Benefits $244,622,814
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.583	**-$174,772,770**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Souris River Community Dike - 40 year Period of Record
Town of Melita
SUMMARY OF COSTS AND BENEFITS
1:200 year Flood

Summary of Costs

1.	South Dike (R.M. of Arthur)	$	-
2.	East Dike (Town of Melita)	$	178,133

Total Dike Construction Cost ***$178,133***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,800 /yr
(1% of Capital Cost)

Present Value of Costs $216,436
(In 2014$)

Benefits

Total Average Annual Benefit $2,100 /yr
(Section 8.6.6)

Present Value of Benefits $53,401
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.247	**-$163,035**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Town of Melita
Souris River Community Dike
SUMMARY OF COSTS AND BENEFITS
1:200 year Flood

Summary of Costs

1.	South Dike (R.M. of Arthur)	$	-
2.	East Dike (Town of Melita)	$	178,133

Total Dike Construction Cost ***$178,133***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $1,800 /yr
(1% of Capital Cost)

Present Value of Costs $216,436
(In 2014$)

Benefits

Total Average Annual Benefit $1,400 /yr
(Section 8.6.6)

Present Value of Benefits $35,601
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.164	**-$180,835**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
File No: 12-0300-011
Project: AR & LMB Basins Flood Mitigation Study

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Souris River Community Dike - 40 year Period of Record
Town of Melita
SUMMARY OF COSTS AND BENEFITS
1:500 year Flood

Summary of Costs

1.	South Dike (R.M. of Arthur)	$	623,570
2.	East Dike (Town of Melita)	$	674,561

Total Dike Construction Cost ***$1,298,131***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $13,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,574,763
(In 2014$)

Benefits

Total Average Annual Benefit $28,100 /yr
(Section 8.6.6)

Present Value of Benefits $714,553
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.454	**-$860,210**

KGS GROUP CONSULTING ENGINEERS

Client: Manitoba Infrastracture and Transportation
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Project: AR & LMB Basins Flood Mitigation Study

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Town of Melita
Souris River Community Dike
SUMMARY OF COSTS AND BENEFITS
1:500 year Flood

Summary of Costs

1.	South Dike (R.M. of Arthur)	$	623,570
2.	East Dike (Town of Melita)	$	674,561

Total Dike Construction Cost ***$1,298,131***

Present Valuation

Interest	7	percent
Inflation	3	percent
Economic Life	50	yrs
Growth	1	percent

Costs

Annual O & M Cost $13,000 /yr
(1% of Capital Cost)

Present Value of Costs $1,574,763
(In 2014$)

Benefits

Total Average Annual Benefit $11,200 /yr
(Section 8.6.6)

Present Value of Benefits $284,804
(In 2014$)

Comparison of B/C Ratios Using Present Value Cost and Benefits

B/C	Net Benefits (B-C)
0.181	**-$1,289,959**

FREQUENCY DAMAGE CURVES FOR INDIVIDUAL AND COMBINATION OF OPTIONS

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Frequency-Damage Curves for the Lower Assiniboine River

Base Case: 25,000 cfs Portage Diversion with 15,000 cfs LA Dikes		
Return Period	Flow (cfs)	Average Annual Damages
0.0251	15,000	$ 10,233,227
0.0238	15,245	$ 15,957,949
0.0217	15,682	$ 26,152,253
0.0208	15,889	$ 30,981,635
0.0192	16,289	$ 40,319,537
0.0161	17,242	$ 62,553,101
0.0149	17,696	$ 73,153,559
0.0145	17,874	$ 77,300,536
0.0142	18,000	$ 80,244,722
0.0139	18,274	$ 104,903,488
0.0121	20,000	$ 259,984,800
0.0119	20,244	$ 278,032,482
0.0106	21,692	$ 385,031,017
0.0090	23,897	$ 547,966,202
0.0088	24,142	$ 566,110,603
0.0083	25,000	$ 629,495,545
0.0078	25,895	$ 672,795,524
0.0072	26,959	$ 724,243,077
0.0063	29,008	$ 823,368,309
0.0061	29,341	$ 839,453,753
0.0061	29,424	$ 843,475,114
0.0060	29,507	$ 847,496,475
0.0054	30,943	$ 916,914,706
0.0052	31,453	$ 941,618,573
0.0044	34,000	$ 1,064,791,265
0.0041	34,800	$ 1,103,500,445
0.0041	34,977	$ 1,112,055,580
0.0036	36,560	$ 1,188,612,975
0.0032	38,649	$ 1,289,628,294
0.0031	39,028	$ 1,307,967,480
0.0025	42,377	$ 1,469,931,049
0.0024	43,014	$ 1,500,750,490
0.0022	44,009	$ 1,548,907,292
0.0021	45,183	$ 1,605,647,188
0.0020	46,000	$ 1,645,185,559
0.0015	51,558	$ 1,913,983,038
0.0013	53,761	$ 2,020,572,087
0.0012	55,446	$ 2,102,037,596
0.0012	55,994	$ 2,128,560,922
0.0011	58,800	$ 2,264,272,806
0.0002	115,302	$ 4,997,076,507
0.0001	173,900	$ 7,831,218,412

34,000 cfs Portage Diversion with 15,000 cfs LA Dikes		
Return Period	Flow (cfs)	Average Annual Damages
0.0140	15,000	$ 10,233,227
0.0122	15,537	$ 22,758,537
0.0119	15,634	$ 25,033,429
0.0107	16,084	$ 35,539,633
0.0090	16,878	$ 54,049,978
0.0088	16,949	$ 55,716,112
0.0083	17,251	$ 62,770,324
0.0078	17,585	$ 70,562,232
0.0072	18,000	$ 80,244,722
0.0063	20,000	$ 259,984,800
0.0061	20,341	$ 285,186,306
0.0060	20,507	$ 297,475,354
0.0054	21,943	$ 403,544,415
0.0052	22,453	$ 441,291,214
0.0044	25,000	$ 629,495,545
0.0041	25,800	$ 668,204,725
0.0041	25,977	$ 676,759,859
0.0036	27,560	$ 753,317,254
0.0032	29,649	$ 854,332,574
0.0031	30,028	$ 872,671,760
0.0025	33,377	$ 1,034,635,329
0.0024	34,000	$ 1,064,791,265
0.0022	35,009	$ 1,113,611,572
0.0021	36,183	$ 1,170,351,468
0.0020	37,000	$ 1,209,889,839
0.0015	42,558	$ 1,478,687,318
0.0013	44,761	$ 1,585,276,366
0.0012	46,446	$ 1,666,741,876
0.0012	46,994	$ 1,693,265,201
0.0011	49,800	$ 1,828,977,085
0.0002	106,302	$ 4,561,780,786
0.0001	164,900	$ 7,395,922,691

46,000 cfs Portage Diversion with 15,000 cfs LA Dikes		
Return Period	Flow (cfs)	Average Annual Damages
0.0119	15,000	$ 8,527,689
0.0106	15,428	$ 20,222,838
0.0090	16,099	$ 35,889,660
0.0088	16,177	$ 37,694,945
0.0083	16,445	$ 43,965,250
0.0078	16,742	$ 50,891,275
0.0072	17,097	$ 59,177,350
0.0063	17,807	$ 75,749,521
0.0061	17,927	$ 78,534,108
0.0060	18,000	$ 67,074,279
0.0054	18,509	$ 125,979,156
0.0052	18,700	$ 143,146,416
0.0044	19,668	$ 230,184,159
0.0041	20,000	$ 259,984,800
0.0041	20,069	$ 265,072,524
0.0036	20,718	$ 313,019,891
0.0032	21,603	$ 378,475,859
0.0031	21,767	$ 390,579,211
0.0025	23,272	$ 501,780,003
0.0024	23,567	$ 523,618,963
0.0022	24,037	$ 558,329,415
0.0021	24,601	$ 600,027,528
0.0020	25,000	$ 629,495,545
0.0015	30,558	$ 898,293,024
0.0013	32,761	$ 1,004,882,072
0.0012	34,446	$ 1,086,347,582
0.0012	34,994	$ 1,112,870,907
0.0011	37,800	$ 1,248,582,792
0.0002	94,302	$ 3,981,386,492
0.0001	152,900	$ 6,815,528,398

18,000 cfs LA Dikes with 25,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0142	18,000	$ 80,244,722
0.0139	18,274	$ 104,903,488
0.0121	20,000	$ 259,984,800
0.0119	20,244	$ 278,032,482
0.0106	21,692	$ 385,031,017
0.0090	23,897	$ 547,966,202
0.0088	24,142	$ 566,110,603
0.0083	25,000	$ 629,495,545
0.0078	25,895	$ 672,795,524
0.0072	26,959	$ 724,243,077
0.0063	29,008	$ 823,368,309
0.0061	29,341	$ 839,453,753
0.0060	29,507	$ 847,496,475
0.0054	30,943	$ 916,914,706
0.0052	31,453	$ 941,618,573
0.0044	34,000	$ 1,064,791,265
0.0041	34,800	$ 1,103,500,445
0.0041	34,977	$ 1,112,055,580
0.0036	36,560	$ 1,188,612,975
0.0032	38,649	$ 1,289,628,294
0.0031	39,028	$ 1,307,967,480
0.0025	42,377	$ 1,469,931,049
0.0024	43,014	$ 1,500,750,490
0.0022	44,009	$ 1,548,907,292
0.0021	45,183	$ 1,605,647,188
0.0020	46,000	$ 1,645,185,559
0.0015	51,558	$ 1,913,983,038
0.0013	53,761	$ 2,020,572,087
0.0012	55,446	$ 2,102,037,596
0.0012	58,799	$ 2,264,224,440
0.0011	58,800	$ 2,264,272,806
0.0002	115,302	$ 4,997,076,507
0.0001	173,900	$ 7,831,218,412

20,000 cfs LA Dikes with 25,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0121	20,000	$ 259,984,800
0.0119	20,244	$ 278,032,482
0.0106	21,692	$ 385,031,017
0.0090	23,897	$ 547,966,202
0.0088	24,142	$ 566,110,603
0.0083	25,000	$ 629,495,545
0.0078	25,895	$ 672,795,524
0.0072	26,959	$ 724,243,077
0.0063	29,008	$ 823,368,309
0.0061	29,341	$ 839,453,753
0.0060	29,507	$ 847,496,475
0.0054	30,943	$ 916,914,706
0.0052	31,453	$ 941,618,573
0.0044	34,000	$ 1,064,791,265
0.0041	34,800	$ 1,103,500,445
0.0041	34,977	$ 1,112,055,580
0.0036	36,560	$ 1,188,612,975
0.0032	38,649	$ 1,289,628,294
0.0031	39,028	$ 1,307,967,480
0.0025	42,377	$ 1,469,931,049
0.0024	43,014	$ 1,500,750,490
0.0022	44,009	$ 1,548,907,292
0.0021	45,183	$ 1,605,647,188
0.0020	46,000	$ 1,645,185,559
0.0016	50,671	$ 1,871,122,833
0.0013	53,761	$ 2,020,572,087
0.0012	55,446	$ 2,102,037,596
0.0012	55,994	$ 2,128,560,922
0.0011	58,800	$ 2,264,272,806
0.0002	115,302	$ 4,997,076,507
0.0001	173,900	$ 7,831,218,412

25,000 cfs LA Dikes with 25,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0083	25,000	$ 629,495,545
0.0078	25,895	$ 672,795,524
0.0072	26,959	$ 724,243,077
0.0063	29,008	$ 823,368,309
0.0061	29,341	$ 839,453,753
0.0060	29,507	$ 847,496,475
0.0054	30,943	$ 916,914,706
0.0052	31,453	$ 941,618,573
0.0044	34,000	$ 1,064,791,265
0.0041	34,800	$ 1,103,500,445
0.0041	34,977	$ 1,112,055,580
0.0036	36,560	$ 1,188,612,975
0.0032	38,649	$ 1,289,628,294
0.0031	39,028	$ 1,307,967,480
0.0025	42,377	$ 1,469,931,049
0.0024	43,014	$ 1,500,750,490
0.0022	44,009	$ 1,548,907,292
0.0021	45,183	$ 1,605,647,188
0.0020	46,000	$ 1,645,185,559
0.0015	51,558	$ 1,913,983,038
0.0013	53,761	$ 2,020,572,087
0.0012	55,446	$ 2,102,037,596
0.0012	55,994	$ 2,128,560,922
0.0011	58,800	$ 2,264,272,806
0.0002	115,302	$ 4,997,076,507
0.0001	173,900	$ 7,831,218,412

Frequency-Damage Curves for the Lower Assiniboine River

35,000 cfs LA Dikes		
Return Period	Flow (cfs)	Average Annual Damages
0.0041	35,000	$ 1,113,157,456
0.0036	36,560	$ 1,188,612,975
0.0032	38,649	$ 1,289,628,294
0.0031	39,028	$ 1,307,967,480
0.0025	42,377	$ 1,469,931,049
0.0024	43,014	$ 1,500,750,490
0.0022	44,009	$ 1,548,907,292
0.0021	45,183	$ 1,605,647,188
0.0020	46,000	$ 1,645,185,559
0.0015	51,545	$ 1,913,361,929
0.0013	53,761	$ 2,020,572,087
0.0012	55,446	$ 2,102,037,596
0.0012	55,994	$ 2,128,560,922
0.0011	58,800	$ 2,264,272,806
0.0002	115,302	$ 4,997,076,507
0.0001	173,900	$ 7,831,218,412

Multitude of Small Dams		
Return Period	Flow (cfs)	Average Annual Damages
0.0192	15,000	$ 10,233,227
0.0161	15,794	$ 28,769,565
0.0149	16,171	$ 37,569,254
0.0145	16,318	$ 41,003,023
0.0143	16,392	$ 42,719,908
0.0139	16,534	$ 46,034,140
0.0122	17,218	$ 61,992,161
0.0119	17,346	$ 64,989,597
0.0107	18,000	$ 80,244,722
0.0090	20,000	$ 259,984,800
0.0088	20,188	$ 273,873,834
0.0083	20,977	$ 332,194,218
0.0078	21,824	$ 394,784,242
0.0072	22,810	$ 467,637,520
0.0063	24,688	$ 606,432,395
0.0061	25,000	$ 629,495,545
0.0060	25,140	$ 636,269,057
0.0054	26,435	$ 698,905,232
0.0052	26,891	$ 720,971,633
0.0044	29,245	$ 834,814,265
0.0041	30,026	$ 872,572,318
0.0041	30,193	$ 880,646,743
0.0036	31,679	$ 952,557,286
0.0032	33,629	$ 1,046,842,682
0.0031	33,982	$ 1,063,899,978
0.0025	37,072	$ 1,213,365,117
0.0024	37,656	$ 1,241,620,778
0.0022	38,566	$ 1,285,611,437
0.0021	39,633	$ 1,337,223,772
0.0020	40,374	$ 1,373,076,709
0.0015	46,000	$ 1,645,185,559
0.0013	48,135	$ 1,748,463,236
0.0012	49,820	$ 1,829,928,746
0.0012	50,368	$ 1,856,452,071
0.0011	53,174	$ 1,992,163,955
0.0002	109,676	$ 4,724,967,656
0.0001	168,274	$ 7,559,109,561

Alexander Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0072	15,004	$ 10,330,358
0.0063	15,561	$ 23,330,483
0.0061	15,653	$ 25,464,861
0.0060	15,694	$ 26,422,676
0.0054	16,073	$ 35,270,698
0.0052	16,204	$ 38,340,854
0.0044	17,081	$ 58,807,350
0.0041	17,404	$ 66,326,722
0.0041	17,473	$ 67,935,848
0.0036	18,264	$ 103,972,586
0.0032	19,903	$ 251,290,964
0.0031	20,208	$ 275,359,371
0.0025	22,831	$ 469,229,142
0.0024	23,320	$ 505,324,279
0.0022	24,074	$ 561,037,853
0.0021	24,949	$ 625,742,617
0.0020	25,553	$ 656,232,761
0.0015	31,098	$ 924,409,130
0.0013	33,314	$ 1,031,619,288
0.0012	34,998	$ 1,113,084,798
0.0012	35,547	$ 1,139,608,123
0.0011	38,353	$ 1,275,320,007
0.0002	94,855	$ 4,008,123,708
0.0001	153,453	$ 6,842,265,613

Victor Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0217	15,000	$ 10,233,227
0.0208	15,171	$ 14,215,014
0.0192	15,516	$ 22,266,238
0.0161	16,428	$ 43,548,667
0.0149	16,866	$ 53,769,001
0.0145	17,037	$ 57,777,979
0.0143	17,123	$ 59,782,468
0.0139	17,290	$ 63,680,343
0.0122	18,228	$ 100,694,462
0.0119	18,517	$ 126,666,130
0.0107	19,937	$ 254,366,254
0.0090	22,306	$ 430,390,877
0.0088	22,510	$ 445,484,272
0.0083	23,369	$ 508,930,756
0.0078	24,295	$ 577,379,182
0.0072	25,380	$ 647,859,710
0.0063	27,476	$ 749,239,550
0.0061	27,829	$ 766,318,378
0.0060	27,987	$ 773,982,609
0.0054	29,461	$ 845,245,518
0.0052	29,986	$ 870,666,761
0.0044	32,450	$ 989,837,190
0.0041	33,241	$ 1,028,084,966
0.0041	33,410	$ 1,036,264,333
0.0036	34,918	$ 1,109,189,304
0.0032	36,898	$ 1,204,943,873
0.0031	37,256	$ 1,222,281,012
0.0025	40,403	$ 1,374,474,615
0.0024	40,999	$ 1,403,289,960
0.0022	41,927	$ 1,448,190,107
0.0021	43,017	$ 1,500,921,812
0.0020	43,775	$ 1,537,579,296
0.0015	49,320	$ 1,805,755,666
0.0013	51,537	$ 1,912,965,824
0.0012	53,221	$ 1,994,431,333
0.0012	53,769	$ 2,020,954,659
0.0011	56,575	$ 2,156,666,543
0.0002	113,078	$ 4,889,470,244
0.0001	171,675	$ 7,723,612,149

Zelena Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0237	15,000	$ 10,233,227
0.0217	15,382	$ 19,152,013
0.0208	15,572	$ 23,583,855
0.0192	15,947	$ 32,331,806
0.0161	16,873	$ 53,949,237
0.0149	17,316	$ 64,280,509
0.0145	17,489	$ 68,324,803
0.0143	17,576	$ 70,346,950
0.0139	17,770	$ 74,879,501
0.0122	19,128	$ 181,576,314
0.0119	19,432	$ 208,936,208
0.0107	20,813	$ 320,030,748
0.0090	23,081	$ 487,692,385
0.0088	23,275	$ 502,013,184
0.0083	24,089	$ 562,171,284
0.0078	24,964	$ 626,861,555
0.0072	25,986	$ 677,169,607
0.0063	27,942	$ 771,785,624
0.0061	28,269	$ 787,591,168
0.0060	28,415	$ 794,684,004
0.0054	29,774	$ 860,412,877
0.0052	30,255	$ 883,681,995
0.0044	32,741	$ 1,003,882,938
0.0041	33,569	$ 1,043,964,007
0.0041	33,747	$ 1,052,536,221
0.0036	35,333	$ 1,129,258,726
0.0032	37,426	$ 1,230,513,171
0.0031	37,807	$ 1,248,897,905
0.0025	41,164	$ 1,411,305,563
0.0024	41,803	$ 1,442,216,099
0.0022	42,802	$ 1,490,520,931
0.0021	43,979	$ 1,547,443,017
0.0020	44,799	$ 1,587,112,338
0.0015	50,344	$ 1,855,288,707
0.0013	52,561	$ 1,962,498,865
0.0012	54,245	$ 2,043,964,375
0.0012	54,793	$ 2,070,487,700
0.0011	57,599	$ 2,206,199,585
0.0002	114,102	$ 4,939,003,285
0.0001	172,699	$ 7,773,145,191

Holland Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0149	15,000	$ 10,233,227
0.0145	15,135	$ 13,387,782
0.0143	15,211	$ 15,165,567
0.0139	15,361	$ 18,649,404
0.0122	16,091	$ 35,687,897
0.0119	16,231	$ 38,971,292
0.0107	16,887	$ 54,277,145
0.0090	18,033	$ 83,218,499
0.0088	18,205	$ 98,645,179
0.0083	19,046	$ 174,255,238
0.0078	19,953	$ 255,716,054
0.0072	21,013	$ 334,823,913
0.0063	23,054	$ 485,710,254
0.0061	23,397	$ 511,049,613
0.0060	23,551	$ 522,420,808
0.0054	24,980	$ 628,020,099
0.0052	25,488	$ 653,114,226
0.0044	27,804	$ 765,099,517
0.0041	28,534	$ 800,398,084
0.0041	28,690	$ 807,945,478
0.0036	30,071	$ 874,761,374
0.0032	31,868	$ 961,668,317
0.0031	32,191	$ 977,320,193
0.0025	34,998	$ 1,113,082,491
0.0024	35,525	$ 1,138,527,312
0.0022	36,340	$ 1,177,950,357
0.0021	37,291	$ 1,223,941,014
0.0020	37,948	$ 1,255,753,369
0.0015	43,493	$ 1,523,929,739
0.0013	45,710	$ 1,631,139,897
0.0012	47,394	$ 1,712,605,406
0.0012	47,942	$ 1,739,128,732
0.0011	50,748	$ 1,874,840,616
0.0002	107,251	$ 4,607,644,317
0.0001	165,848	$ 7,441,786,222

Frequency-Damage Curves for the Lower Assiniboine River

High Souris Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0220	15,000	$ 10,233,227
0.0208	15,208	$ 15,086,693
0.0192	15,539	$ 22,822,014
0.0161	16,351	$ 41,765,288
0.0149	16,735	$ 50,717,933
0.0145	16,884	$ 54,205,274
0.0143	16,959	$ 55,948,945
0.0139	17,103	$ 59,302,641
0.0122	17,792	$ 75,387,574
0.0119	17,981	$ 79,808,215
0.0107	19,029	$ 172,731,273
0.0090	21,222	$ 350,317,849
0.0088	21,423	$ 365,153,712
0.0083	22,267	$ 427,504,068
0.0078	23,176	$ 494,699,775
0.0072	24,240	$ 573,310,157
0.0063	26,289	$ 691,849,543
0.0061	26,634	$ 708,503,943
0.0060	26,788	$ 715,977,709
0.0054	28,223	$ 785,395,940
0.0052	28,734	$ 810,099,807
0.0044	31,271	$ 932,805,870
0.0041	32,106	$ 973,205,772
0.0041	32,285	$ 981,846,308
0.0036	33,885	$ 1,059,229,254
0.0032	35,998	$ 1,161,440,198
0.0031	36,382	$ 1,180,007,123
0.0025	39,777	$ 1,344,191,138
0.0024	40,423	$ 1,375,466,055
0.0022	41,434	$ 1,424,363,006
0.0021	42,626	$ 1,482,013,855
0.0020	43,457	$ 1,522,206,973
0.0015	49,002	$ 1,790,383,342
0.0013	51,219	$ 1,897,593,500
0.0012	52,903	$ 1,979,059,010
0.0012	53,451	$ 2,005,582,335
0.0011	56,257	$ 2,141,294,220
0.0002	112,760	$ 4,874,097,920
0.0001	171,357	$ 7,708,239,826

Nesbitt Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0208	15,000	$ 10,233,227
0.0192	15,305	$ 17,359,070
0.0161	16,053	$ 34,810,374
0.0149	16,402	$ 42,960,208
0.0145	16,538	$ 46,117,368
0.0143	16,605	$ 47,695,948
0.0139	16,735	$ 50,724,484
0.0122	17,355	$ 65,184,762
0.0119	17,469	$ 67,854,298
0.0107	18,073	$ 86,847,878
0.0090	20,060	$ 264,432,449
0.0088	20,256	$ 278,924,936
0.0083	21,080	$ 339,813,788
0.0078	21,967	$ 405,339,204
0.0072	23,002	$ 481,861,722
0.0063	24,990	$ 628,728,925
0.0061	25,322	$ 645,082,317
0.0060	25,472	$ 652,302,130
0.0054	26,856	$ 719,260,789
0.0052	27,347	$ 743,008,156
0.0044	29,865	$ 864,814,654
0.0041	30,704	$ 905,373,972
0.0041	30,883	$ 914,048,669
0.0036	32,490	$ 991,761,835
0.0032	34,613	$ 1,094,451,030
0.0031	34,999	$ 1,113,109,049
0.0025	38,412	$ 1,278,181,243
0.0024	39,062	$ 1,309,638,351
0.0022	40,079	$ 1,358,831,362
0.0021	41,279	$ 1,416,846,591
0.0020	42,115	$ 1,457,301,608
0.0015	47,660	$ 1,725,477,977
0.0013	49,877	$ 1,832,688,135
0.0012	51,561	$ 1,914,153,645
0.0012	52,109	$ 1,940,676,970
0.0011	54,915	$ 2,076,388,855
0.0002	111,418	$ 4,809,192,555
0.0001	170,015	$ 7,643,334,461

Hoop and Holler		
Return Period	Flow (cfs)	Average Annual Damages
0.0251	15,000	$ 10,233,227
0.0238	15,245	$ 15,957,949
0.0217	15,682	$ 26,152,253
0.0208	15,889	$ 30,981,635
0.0192	16,290	$ 40,340,968
0.0161	17,242	$ 62,553,101
0.0149	17,696	$ 73,153,559
0.0145	17,874	$ 77,300,536
0.0143	17,963	$ 79,374,025
0.0139	18,000	$ 80,242,352
0.0122	18,000	$ 80,244,722
0.0119	18,000	$ 80,244,722
0.0107	18,312	$ 108,317,582
0.0090	20,442	$ 292,617,219
0.0088	20,642	$ 307,453,082
0.0083	21,486	$ 369,803,438
0.0078	22,395	$ 436,999,145
0.0072	23,459	$ 515,609,527
0.0063	25,508	$ 654,086,640
0.0061	25,853	$ 670,741,040
0.0060	26,007	$ 678,214,806
0.0054	27,443	$ 747,633,036
0.0052	27,953	$ 772,336,904
0.0044	30,473	$ 894,217,416
0.0041	31,300	$ 934,218,776
0.0041	31,477	$ 942,773,911
0.0036	33,060	$ 1,019,331,306
0.0032	35,149	$ 1,120,346,625
0.0031	35,528	$ 1,138,685,811
0.0025	38,877	$ 1,300,649,380
0.0024	39,514	$ 1,331,468,821
0.0022	40,509	$ 1,379,625,623
0.0021	41,683	$ 1,436,365,519
0.0020	42,500	$ 1,475,903,890
0.0015	48,045	$ 1,744,080,260
0.0013	50,261	$ 1,851,290,418
0.0012	51,946	$ 1,932,755,927
0.0012	52,494	$ 1,959,279,253
0.0011	55,300	$ 2,094,991,137
0.0002	111,802	$ 4,827,794,838
0.0001	170,400	$ 7,661,936,743

20,000 cfs LA Dikes + 34,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0063	20,000	$ 259,984,800
0.0061	20,341	$ 285,186,306
0.0060	20,507	$ 297,475,354
0.0054	21,943	$ 403,544,415
0.0052	22,453	$ 441,291,214
0.0044	24,973	$ 627,521,131
0.0041	25,800	$ 668,204,725
0.0041	25,977	$ 676,759,859
0.0036	27,560	$ 753,317,254
0.0032	29,649	$ 854,332,574
0.0031	30,028	$ 872,671,760
0.0025	33,377	$ 1,034,635,329
0.0024	34,014	$ 1,065,454,769
0.0022	35,009	$ 1,113,611,572
0.0021	36,183	$ 1,170,351,468
0.0020	37,000	$ 1,209,889,839
0.0015	42,545	$ 1,478,066,208
0.0013	44,761	$ 1,585,276,366
0.0012	46,446	$ 1,666,741,876
0.0012	46,994	$ 1,693,265,201
0.0011	49,800	$ 1,828,977,085
0.0002	106,302	$ 4,561,780,786
0.0001	164,900	$ 7,395,922,691

25,000 cfs LA Dikes + 34,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0044	25,000	$ 629,495,545
0.0041	25,800	$ 668,204,725
0.0041	25,977	$ 676,759,859
0.0036	27,560	$ 753,317,254
0.0032	29,649	$ 854,332,574
0.0031	30,028	$ 872,671,760
0.0025	33,377	$ 1,034,635,329
0.0024	34,014	$ 1,065,454,769
0.0022	35,009	$ 1,113,611,572
0.0021	36,183	$ 1,170,351,468
0.0020	37,000	$ 1,209,889,839
0.0015	42,545	$ 1,478,066,208
0.0013	44,761	$ 1,585,276,366
0.0012	46,446	$ 1,666,741,876
0.0012	46,994	$ 1,693,265,201
0.0011	49,800	$ 1,828,977,085
0.0002	106,302	$ 4,561,780,786
0.0001	164,900	$ 7,395,922,691

35,000 cfs LA Dikes + 34,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0022	35,000	$ 1,113,157,456
0.0021	36,183	$ 1,170,351,468
0.0020	37,000	$ 1,209,889,839
0.0015	42,545	$ 1,478,066,208
0.0013	44,761	$ 1,585,276,366
0.0012	46,446	$ 1,666,741,876
0.0012	46,994	$ 1,693,265,201
0.0011	49,800	$ 1,828,977,085
0.0002	106,302	$ 4,561,780,786
0.0001	164,900	$ 7,395,922,691

Frequency-Damage Curves for the Lower Assiniboine River

20,000 cfs LA Dikes + 46,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0027	20,000	$ 259,984,800
0.0025	33,377	$ 1,034,635,329
0.0024	34,014	$ 1,065,454,769
0.0022	35,009	$ 1,113,611,572
0.0021	36,183	$ 1,170,351,468
0.0020	37,000	$ 1,209,889,839
0.0015	42,545	$ 1,478,066,208
0.0013	44,761	$ 1,585,276,366
0.0012	46,446	$ 1,666,741,876
0.0012	46,994	$ 1,693,265,201
0.0011	49,800	$ 1,828,977,085
0.0002	94,302	$ 3,981,386,492
0.0001	152,900	$ 6,815,528,398

25,000 cfs LA Dikes + 46,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0020	25,000	$ 629,495,545
0.0015	42,545	$ 1,478,066,208
0.0013	44,761	$ 1,585,276,366
0.0012	46,446	$ 1,666,741,876
0.0012	46,994	$ 1,693,265,201
0.0011	49,800	$ 1,828,977,085
0.0002	94,302	$ 3,981,386,492
0.0001	152,900	$ 6,815,528,398

35,000 cfs LA Dikes + 46,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.0012	35,000	$ 1,113,157,456
0.0011	49,800	$ 1,828,977,085
0.0002	94,302	$ 3,981,386,492
0.0001	152,900	$ 6,815,528,398

20,000 cfs LA Dikes + Alexander Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0032	20,000	$ 259,984,800
0.0031	20,208	$ 275,359,371
0.0025	22,831	$ 469,229,142
0.0024	23,320	$ 505,324,279
0.0022	24,074	$ 561,037,853
0.0021	24,949	$ 625,742,617
0.0020	25,553	$ 656,232,761
0.0015	31,098	$ 924,409,130
0.0013	33,314	$ 1,031,619,288
0.0012	34,998	$ 1,113,084,798
0.0012	35,547	$ 1,139,608,123
0.0011	38,353	$ 1,275,320,007
0.0002	94,855	$ 4,008,123,708
0.0001	153,453	$ 6,842,265,613

25,000 cfs LA Dikes + Alexander Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0021	25,000	$ 629,495,545
0.0020	25,553	$ 656,232,761
0.0015	31,098	$ 924,409,130
0.0013	33,314	$ 1,031,619,288
0.0012	34,998	$ 1,113,084,798
0.0012	35,547	$ 1,139,608,123
0.0011	38,353	$ 1,275,320,007
0.0002	94,855	$ 4,008,123,708
0.0001	153,453	$ 6,842,265,613

35,000 cfs LA Dikes + Alexander Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0012	35,000	$ 1,113,157,456
0.0012	35,547	$ 1,139,609,405
0.0011	38,353	$ 1,275,320,007
0.0002	94,855	$ 4,008,123,708
0.0001	153,453	$ 6,842,265,613

Frequency-Damage Curves for the Lower Assiniboine River

20,000 cfs LA Dikes + Holland Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0078	20,000	$ 259,984,800
0.0072	21,013	$ 334,823,913
0.0063	23,054	$ 485,710,254
0.0061	23,397	$ 511,049,613
0.0061	23,468	$ 516,302,382
0.0060	23,551	$ 522,420,808
0.0054	24,980	$ 628,020,099
0.0052	25,488	$ 653,114,226
0.0044	27,804	$ 765,099,517
0.0041	28,534	$ 800,398,084
0.0041	28,690	$ 807,945,478
0.0036	30,071	$ 874,761,374
0.0032	31,868	$ 961,668,317
0.0031	32,191	$ 977,320,193
0.0025	34,998	$ 1,113,082,491
0.0024	35,525	$ 1,138,527,312
0.0022	36,340	$ 1,177,950,357
0.0021	37,291	$ 1,223,941,014
0.0020	37,948	$ 1,255,753,369
0.0015	43,493	$ 1,523,929,739
0.0013	45,710	$ 1,631,139,897
0.0012	47,394	$ 1,712,605,406
0.0012	47,942	$ 1,739,128,732
0.0011	50,748	$ 1,874,840,616
0.0002	107,251	$ 4,607,644,317
0.0001	165,848	$ 7,441,786,222

25,000 cfs LA Dikes + Holland Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0054	25,000	$ 629,495,545
0.0052	25,488	$ 653,114,226
0.0044	27,804	$ 765,099,517
0.0041	28,534	$ 800,398,084
0.0041	28,690	$ 807,945,478
0.0036	30,071	$ 874,761,374
0.0032	31,868	$ 961,668,317
0.0031	32,191	$ 977,320,193
0.0025	34,998	$ 1,113,082,491
0.0024	35,525	$ 1,138,527,312
0.0022	36,340	$ 1,177,950,357
0.0021	37,291	$ 1,223,941,014
0.0020	37,948	$ 1,255,753,369
0.0015	43,493	$ 1,523,929,739
0.0013	45,710	$ 1,631,139,897
0.0012	47,394	$ 1,712,605,406
0.0012	47,942	$ 1,739,128,732
0.0011	50,748	$ 1,874,840,616
0.0002	107,251	$ 4,607,644,317
0.0001	165,848	$ 7,441,786,222

35,000 cfs LA Dikes + Holland Dam		
Return Period	Flow (cfs)	Average Annual Damages
0.0025	35,000	$ 1,113,157,456
0.0024	35,525	$ 1,138,527,312
0.0022	36,340	$ 1,177,950,357
0.0021	37,291	$ 1,223,941,014
0.0020	37,948	$ 1,255,753,369
0.0015	43,493	$ 1,523,929,739
0.0013	45,710	$ 1,631,139,897
0.0012	47,394	$ 1,712,605,406
0.0012	47,942	$ 1,739,128,732
0.0011	50,748	$ 1,874,840,616
0.0002	107,251	$ 4,607,644,317
0.0001	165,848	$ 7,441,786,222

Holland Dam and 34,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.008852	15,000	$ 10,233,227
0.008333	15,257	$ 16,234,775
0.007813	15,541	$ 22,862,230
0.007226	15,895	$ 31,126,740
0.006254	16,565	$ 46,751,465
0.006098	16,684	$ 49,521,745
0.006024	16,741	$ 50,872,621
0.005435	17,249	$ 62,727,870
0.005236	17,436	$ 67,076,854
0.004359	18,827	$ 154,607,817
0.004115	19,534	$ 218,061,547
0.004065	19,690	$ 232,085,486
0.003636	21,071	$ 339,135,337
0.003155	22,868	$ 471,926,646
0.003077	23,191	$ 495,842,263
0.002481	25,998	$ 677,786,770
0.002389	26,513	$ 702,688,377
0.002247	27,340	$ 742,654,637
0.002096	28,291	$ 788,645,293
0.002	28,948	$ 820,457,649
0.001481	34,506	$ 1,089,255,128
0.001328	36,710	$ 1,195,844,176
0.001225	38,394	$ 1,277,309,686
0.001195	38,942	$ 1,303,833,011
0.001053	41,748	$ 1,439,544,895
0.0002	98,251	$ 4,172,348,596
0.00008	156,848	$ 7,006,490,501

Alexander Dam and 34,000 cfs Portage Diversion		
Return Period	Flow (cfs)	Average Annual Damages
0.003639	15,000	$ 10,233,227
0.003155	15,526	$ 22,517,391
0.003077	15,623	$ 24,761,624
0.002481	16,492	$ 45,062,834
0.002389	16,657	$ 48,898,178
0.002247	16,924	$ 55,142,190
0.002096	17,239	$ 62,484,133
0.002	17,459	$ 67,626,507
0.001481	22,110	$ 415,944,718
0.001328	24,314	$ 578,809,718
0.001225	25,998	$ 677,789,078
0.001195	26,547	$ 704,312,403
0.001053	29,353	$ 840,024,287
0.0002	85,855	$ 3,572,827,988
0.00008	144,453	$ 6,406,969,893

34,000 cfs Portage Diversion (short period)		
Return Period	Flow (cfs)	Average Annual Damages
0.034559	15,000	$ 10,233,227
0.025	17,079	$ 58,745,975
0.02381	17,495	$ 68,448,722
0.022901	18,000	$ 80,244,722
0.021739	19,016	$ 171,583,831
0.019231	21,808	$ 393,599,885
0.016129	24,848	$ 618,262,418
0.014925	26,368	$ 695,660,494
0.014493	26,976	$ 725,067,139
0.014286	27,280	$ 739,770,461
0.013889	27,888	$ 769,177,105
0.012195	30,928	$ 916,210,326
0.011905	31,536	$ 945,616,970
0.010693	34,429	$ 1,085,550,970
0.00898	38,387	$ 1,276,982,957
0.00885	38,675	$ 1,290,903,209
0.008333	39,900	$ 1,350,151,793
0.007813	41,300	$ 1,417,864,461
0.007226	43,118	$ 1,505,793,311
0.006254	46,882	$ 1,687,856,382
0.006098	47,600	$ 1,722,571,465
0.006024	47,950	$ 1,739,499,632
0.005435	51,100	$ 1,891,853,134
0.005236	52,325	$ 1,951,101,718
0.004359	56,558	$ 2,155,842,922
0.004115	57,784	$ 2,215,148,878
0.004065	58,055	$ 2,228,256,116
0.003636	60,675	$ 2,354,959,414
0.003155	64,469	$ 2,538,460,744
0.003077	65,192	$ 2,573,413,378
0.002481	72,238	$ 2,914,201,561
0.002389	73,651	$ 2,982,542,277
0.002247	76,032	$ 3,097,702,890
0.002096	78,922	$ 3,237,513,426
0.002	81,000	$ 3,338,002,249
0.001481	89,400	$ 3,744,278,255
0.001328	93,144	$ 3,925,361,275
0.001225	96,168	$ 4,071,620,637
0.001195	97,176	$ 4,120,373,758
0.001053	102,597	$ 4,382,564,617
0.0002	165,000	$ 7,400,762,306
0.00008	213,350	$ 9,739,284,590

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Frequency-Damage Curves for Lake Manitoba

Basline: 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 798,618
0.2	$ 4,159,989
0.1	$ 3,205,296
0.05	$ 3,005,152
0.02	$ 1,429,814
0.01	$ 913,528
0.005	$ 373,381
0.002	$ 589,260
0.001	$ 647,431

34,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 830,415
0.2	$ 4,304,936
0.1	$ 3,358,413
0.05	$ 3,132,203
0.02	$ 1,482,636
0.01	$ 945,650
0.005	$ 391,692
0.002	$ 645,217
0.001	$ 705,724

46,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 850,132
0.2	$ 4,407,807
0.1	$ 3,436,939
0.05	$ 3,218,395
0.02	$ 1,517,702
0.01	$ 972,623
0.005	$ 408,102
0.002	$ 680,817
0.001	$ 746,321

Alexander Dam	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 691,542
0.2	$ 3,588,417
0.1	$ 2,668,473
0.05	$ 2,333,813
0.02	$ 1,116,223
0.01	$ 786,463
0.005	$ 342,189
0.002	$ 560,202
0.001	$ 627,400

Holland Dam	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 730,037
0.2	$ 3,787,175
0.1	$ 2,905,906
0.05	$ 2,735,701
0.02	$ 1,331,382
0.01	$ 876,965
0.005	$ 362,859
0.002	$ 575,083
0.001	$ 635,477

Alexander Dam with 34,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 725,302
0.2	$ 3,753,897
0.1	$ 2,783,965
0.05	$ 2,427,996
0.02	$ 1,159,624
0.01	$ 812,266
0.005	$ 357,139
0.002	$ 608,665
0.001	$ 685,555

Frequency-Damage Curves for Lake Manitoba

Holland Dam with 34,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 759,785
0.2	$ 3,944,670
0.1	$ 3,026,722
0.05	$ 2,841,398
0.02	$ 1,373,512
0.01	$ 902,981
0.005	$ 379,987
0.002	$ 626,248
0.001	$ 694,461

20,000 cfs Assiniboine Dikes with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 790,383
0.2	$ 4,126,303
0.1	$ 3,185,003
0.05	$ 2,990,568
0.02	$ 1,425,101
0.01	$ 911,336
0.005	$ 372,543
0.002	$ 587,899
0.001	$ 645,941

25,000 cfs Assiniboine Dikes with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 769,469
0.2	$ 4,023,599
0.1	$ 3,125,696
0.05	$ 2,954,315
0.02	$ 1,412,809
0.01	$ 905,682
0.005	$ 370,413
0.002	$ 584,105
0.001	$ 642,208

35,000 cfs Assiniboine Dikes with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 729,087
0.2	$ 3,851,494
0.1	$ 3,019,477
0.05	$ 2,880,117
0.02	$ 1,388,471
0.01	$ 894,501
0.005	$ 366,127
0.002	$ 575,634
0.001	$ 633,221

20,000cfs Assiniboine Dikes with 34,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 822,207
0.2	$ 4,271,367
0.1	$ 3,338,278
0.05	$ 3,117,639
0.02	$ 1,477,949
0.01	$ 943,095
0.005	$ 390,632
0.002	$ 643,215
0.001	$ 703,136

25,000cfs Assiniboine Dikes with 34,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 801,684
0.2	$ 4,187,435
0.1	$ 3,251,570
0.05	$ 3,059,589
0.02	$ 1,466,296
0.01	$ 936,699
0.005	$ 387,976
0.002	$ 638,710
0.001	$ 697,672

Frequency-Damage Curves for Lake Manitoba

35,000cfs Assiniboine Dikes with 34,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 760,205
0.2	$ 4,000,639
0.1	$ 3,141,355
0.05	$ 2,986,770
0.02	$ 1,442,843
0.01	$ 924,404
0.005	$ 381,756
0.002	$ 625,525
0.001	$ 690,160

5,000 cfs Lake Winnipegosis Outlet with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 729,418
0.2	$ 3,577,233
0.1	$ 2,590,810
0.05	$ 1,939,308
0.02	$ 935,713
0.01	$ 719,021
0.005	$ 308,130
0.002	$ 467,595
0.001	$ 518,000

2,500 cfs Lake Winnipegosis Outlet with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 755,353
0.2	$ 3,851,832
0.1	$ 2,879,178
0.05	$ 2,288,159
0.02	$ 1,127,127
0.01	$ 817,823
0.005	$ 338,562
0.002	$ 515,219
0.001	$ 572,787

10,000 cfs Lake Winnipegosis Outlet with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 702,237
0.2	$ 3,413,587
0.1	$ 2,341,905
0.05	$ 1,712,729
0.02	$ 786,574
0.01	$ 609,861
0.005	$ 272,270
0.002	$ 409,406
0.001	$ 445,530

6,000 cfs Waterhen Outlet Control with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 788,372
0.2	$ 3,890,978
0.1	$ 2,842,297
0.05	$ 2,130,718
0.02	$ 991,332
0.01	$ 734,535
0.005	$ 310,437
0.002	$ 468,619
0.001	$ 519,162

5,000 cfs LMB Outlet with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ -
0.2	$ 1,567,187
0.1	$ 2,069,288
0.05	$ 2,144,808
0.02	$ 1,106,247
0.01	$ 765,247
0.005	$ 322,723
0.002	$ 489,180
0.001	$ 539,264

Frequency-Damage Curves for Lake Manitoba

10,000 cfs LMB Outlet with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ -
0.2	$ -
0.1	$ 868,813
0.05	$ 1,551,887
0.02	$ 872,391
0.01	$ 648,697
0.005	$ 284,697
0.002	$ 426,138
0.001	$ 459,518

15,000 cfs LMB Outlet with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ -
0.2	$ -
0.1	$ 533,999
0.05	$ 1,094,098
0.02	$ 686,510
0.01	$ 538,779
0.005	$ 244,626
0.002	$ 380,302
0.001	$ 419,366

20,000cfs Assiniboine Dikes with the Alexander Dam with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 683,646
0.2	$ 3,555,748
0.1	$ 2,649,154
0.05	$ 2,319,864
0.02	$ 1,111,190
0.01	$ 784,158
0.005	$ 341,393
0.002	$ 558,902
0.001	$ 626,031

25,000cfs Assiniboine Dikes with the Alexander Dam with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 663,906
0.2	$ 3,474,690
0.1	$ 2,600,312
0.05	$ 2,284,465
0.02	$ 1,098,213
0.01	$ 778,200
0.005	$ 339,403
0.002	$ 555,653
0.001	$ 622,606

35,000cfs Assiniboine Dikes with the Alexander Dam with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 625,286
0.2	$ 3,314,509
0.1	$ 2,505,065
0.05	$ 2,211,829
0.02	$ 1,070,963
0.01	$ 766,176
0.005	$ 333,386
0.002	$ 539,041
0.001	$ 615,724

20,000cfs Assiniboine Dikes with the Holland Dam with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 722,131
0.2	$ 3,750,844
0.1	$ 2,884,874
0.05	$ 2,721,673
0.02	$ 1,326,806
0.01	$ 874,875
0.005	$ 362,036
0.002	$ 573,710
0.001	$ 633,996

Frequency-Damage Curves for Lake Manitoba

25,000cfs Assiniboine Dikes with the Holland Dam with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 702,366
0.2	$ 3,651,960
0.1	$ 2,828,265
0.05	$ 2,686,600
0.02	$ 1,315,361
0.01	$ 869,649
0.005	$ 359,803
0.002	$ 569,416
0.001	$ 630,300

35,000cfs Assiniboine Dikes with the Holland Dam with a 25,000 cfs Portage Diversion	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 657,594
0.2	$ 3,478,351
0.1	$ 2,729,881
0.05	$ 2,614,492
0.02	$ 1,290,263
0.01	$ 858,069
0.005	$ 355,545
0.002	$ 562,337
0.001	$ 623,438

34,000 cfs Portage Diversion in Combination with 5,000 cfs LMB Outlet and 20,000 cfs LA Dikes	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 413,256
0.2	$ 2,461,834
0.1	$ 2,154,848
0.05	$ 2,239,652
0.02	$ 1,156,223
0.01	$ 792,548
0.005	$ 338,019
0.002	$ 541,446
0.001	$ 593,442

34,000 cfs Portage Diversion in Combination with 10,000 cfs LMB Outlet and 20,000 cfs LA Dikes	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 13,295
0.2	$ 75,673
0.1	$ 254,810
0.05	$ 619,608
0.02	$ 459,146
0.01	$ 399,860
0.005	$ 198,221
0.002	$ 342,126
0.001	$ 384,651

34,000 cfs Portage Diversion in Combination with 5,000 cfs LMB Outlet and 25,000 cfs LA Dikes	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 413,030
0.2	$ 2,460,665
0.1	$ 2,155,885
0.05	$ 2,240,348
0.02	$ 1,157,015
0.01	$ 792,113
0.005	$ 337,719
0.002	$ 545,927
0.001	$ 601,886

34,000 cfs Portage Diversion in Combination with 10,000 cfs LMB Outlet and 25,000 cfs LA Dikes	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 14,591
0.2	$ 84,427
0.1	$ 270,623
0.05	$ 637,564
0.02	$ 467,199
0.01	$ 405,132
0.005	$ 200,274
0.002	$ 344,673
0.001	$ 387,043

Frequency-Damage Curves for Lake Manitoba

34,000 cfs Portage Diversion in Combination with 7,500 cfs LMB Outlet and 23,100 cfs LA Dikes	
Frequency	Damages
0.7	$ -
0.4	$ -
0.25	$ 30,468
0.2	$ 432,726
0.1	$ 680,407
0.05	$ 1,030,590
0.02	$ 651,772
0.01	$ 516,596
0.005	$ 241,437
0.002	$ 396,269
0.001	$ 434,403

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Frequency-Damage Curves for The City of Winnipeg

Frequency	Base Case: 25,000 cfs Portage Diversion	34,000 cfs Portage Diversion	46,000 cfs Portage Diversion	18,000 cfs Lower Assiniboine Dikes with 25,000 cfs Portage Diversion	20,000 cfs Lower Assiniboine Dikes with 25,000 cfs Portage Diversion	25,000 cfs Lower Assiniboine Dikes with 25,000 cfs Portage Diversion
	(Million $)	(Million $)	(Million $)	(Million $)	(Million $)	(Million $)
0.06	1	0	0	1	1	1
0.05	2	1	1	2	2	2
0.04	4	3	3	4	4	5
0.02	33	30	28	33	34	35
0.01	46	42	41	46	46	48
0.007	142	126	121	143	145	151
0.005	267	254	250	267	270	274
0.004	362	341	333	362	365	373
0.003	827	723	695	829	850	898
0.002	3,566	3,254	3,159	3,571	3,630	3,741
0.001	9,537	9,213	9,104	9,542	9,598	9,706

Frequency-Damage Curves for The City of Winnipeg

Frequency	35,000 cfs Lower Assiniboine Dikes with 25,000 cfs Portage Diversion	Multitude of Small Dams	Alexander Dam	Victor Dam	Zelena Dam	Holland Dam
	(Million $)	(Million $)	(Million $)	(Million $)	(Million $)	(Million $)
0.06	1	1	0	-	1	1
0.05	2	2	1	-	2	2
0.04	6	4	3	-	4	4
0.02	37	33	31	-	33	32
0.01	53	43	41	-	45	43
0.007	158	132	119	2	141	129
0.005	279	259	248	33	266	257
0.004	384	349	330	44	359	345
0.003	946	756	680	264	813	738
0.002	3,839	3,376	3,118	3,494	3,530	3,317
0.001	9,815	9,337	9,068	9,460	9,497	9,279

Frequency-Damage Curves for The City of Winnipeg

Frequency	High Souris Dam	Nesbitt Dam	34,000 cfs Portage Diversion with 18,000 cfs Lower Assiniboine Dikes	34,000 cfs Portage Diversion with 20,000 cfs Lower Assiniboine Dikes	34,000 cfs Portage Diversion with 25,000 cfs Lower Assiniboine Dikes	34,000 cfs Portage Diversion with 35,000 cfs Lower Assiniboine Dikes
	(Million $)	(Million $)	(Million $)	(Million $)	(Million $)	(Million $)
0.06	1	1	0	0	1	1
0.05	2	2	1	1	2	2
0.04	4	4	3	3	4	4
0.02	33	33	30	30	31	34
0.01	44	44	42	42	43	46
0.007	137	135	126	129	134	142
0.005	264	262	255	257	261	266
0.004	356	354	341	345	350	357
0.003	793	780	725	740	772	811
0.002	3,480	3,446	3,257	3,309	3,414	3,530
0.001	9,444	9,409	9,216	9,269	9,376	9,494

Frequency-Damage Curves for the Upper Assiniboine River

Assiniboine River from Shellmouth to St. Lazare	
Frequency	Damages
0.500	$ 127,778
0.400	$ 377,219
0.333	$ 482,267
0.286	$ 896,129
0.250	$ 1,319,765
0.200	$ 2,382,866
0.167	$ 3,531,349
0.100	$ 4,725,056
0.020	$ 5,050,080
0.002	$ 5,050,080

Assiniboine River from Shellmouth to Brandon	
Frequency	Damages
0.5000	$ 127,778
0.4000	$ 377,219
0.3333	$ 2,983,026
0.2857	$ 7,404,071
0.2500	$ 14,706,991
0.2000	$ 22,831,600
0.1667	$ 30,002,404
0.1000	$ 32,193,190
0.0200	$ 32,518,213
0.0020	$ 32,518,213

Frequency-Damage Curves for the Souris River

Souris River at Wawanesa	
Frequency	Damages
0.0020	$ 26,775,000
0.0033	$ 26,775,000
0.0033	$ 26,775,000
0.0033	$ -

Souris River at Souris	
Frequency	Damages
0.0020	$ 29,662,500
0.0040	$ 15,225,000
0.0050	$ 14,358,750
0.0068	$ 13,492,500
0.0068	$ -

Souris River at Melita	
Frequency	Damages
0.0020	$ 6,174,666
0.0027	$ 5,844,237
0.0027	$ 2,619,600
0.0037	$ 2,425,457
0.0050	$ 2,425,457
0.0056	$ 2,250,978
0.0056	$ -

Souris River at Melita (Short Period of Record)	
Frequency	Damages
0.0028	$ 6,174,666
0.0052	$ 5,844,237
0.0052	$ 2,619,600
0.0074	$ 2,425,457
0.0110	$ 2,250,978
0.0110	$ -

APPENDIX P

LAKE MANITOBA BATHYMETRY

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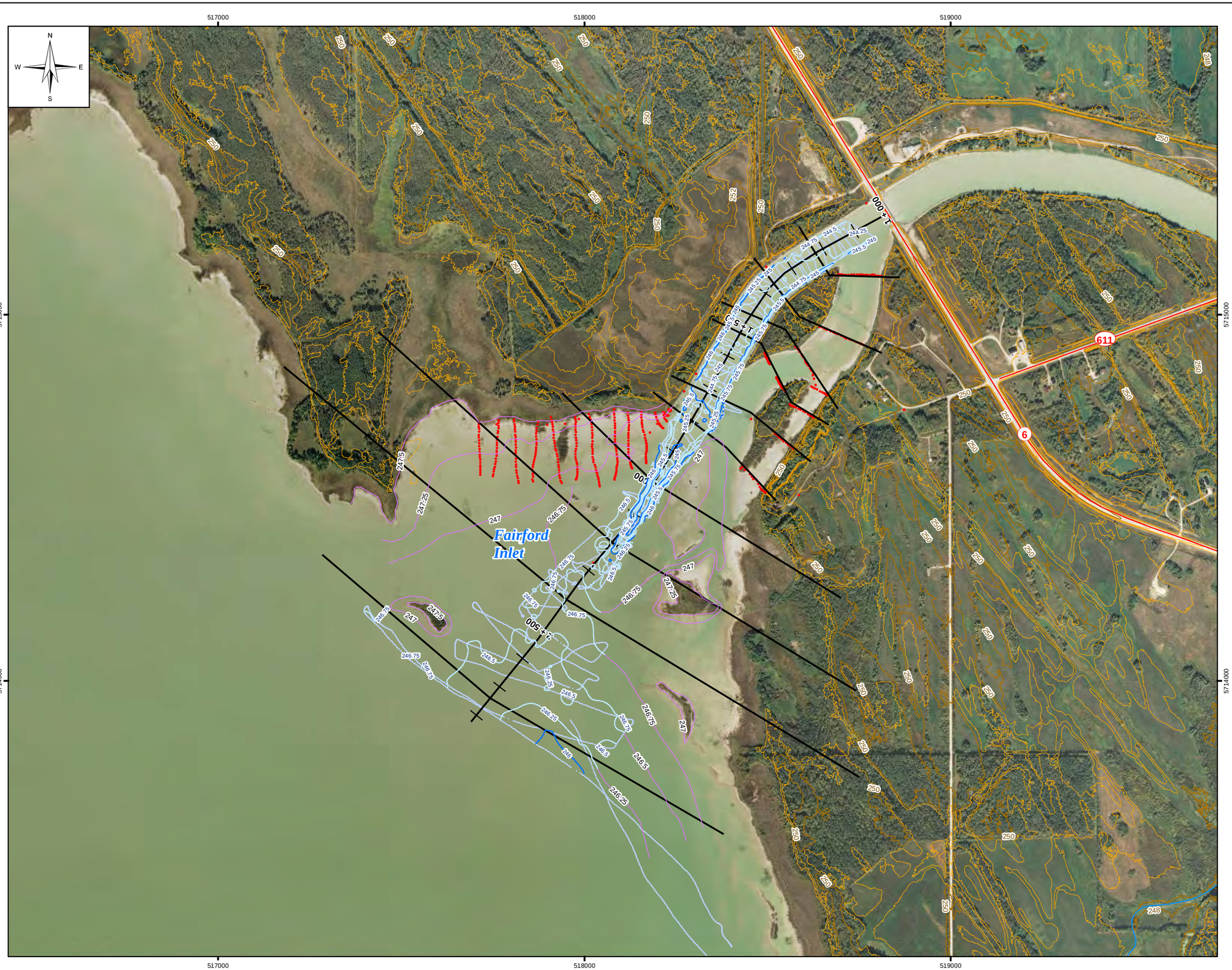
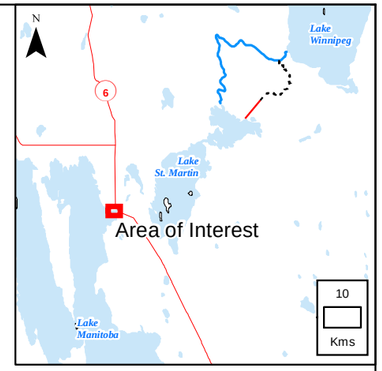




LEGEND:

- Survey Point
- Sonar Point
- Cross Section
- Highway
- Watercourse
- 2m Index Contour
- 0.5m Contour
- 1m Index Sonar Contour
- 0.25m Sonar Contour
- Interpolated Contours

NOTES:

1. Aerial Imagery: July 2011.
2. Original ground surface based on LiDAR provided by Atlis Geomatics (June/July 2011).

100 0 100 200 300 400
Metres

SCALE: 1:10,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	15/07/10	ISSUED WITH DRAFT REPORT	PAL

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

BATHYMETRY OF FAIRFORD RIVER INLET LOCATION

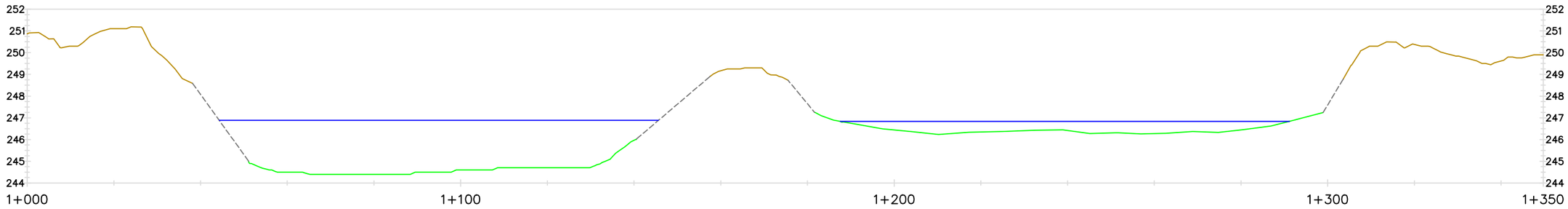
JULY 2015 | FIGURE P-1 | REV: A

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\RevA\Final Report\12-0300-011_FigP1_RevA.mxd
11"x17" PLOT SCALE 1:1

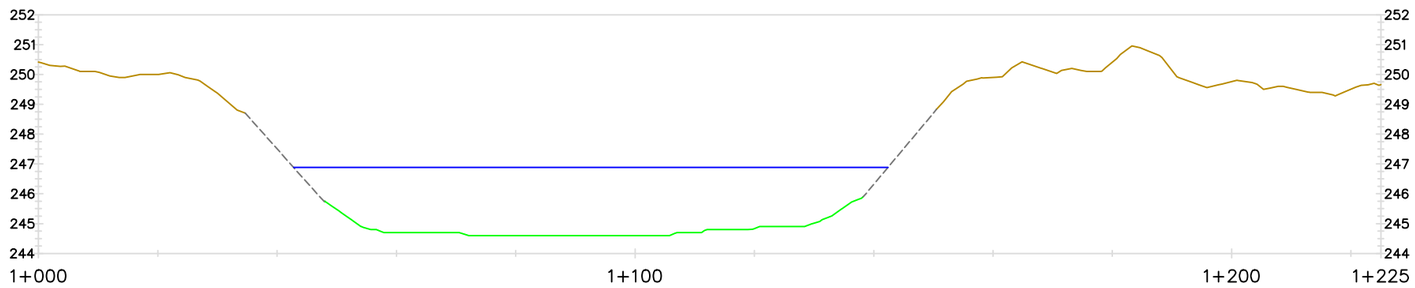
JULY 10/15
1 : 1000 5X VERT.

FAIRFORD INLET PROFILES

Profile View of 1+223

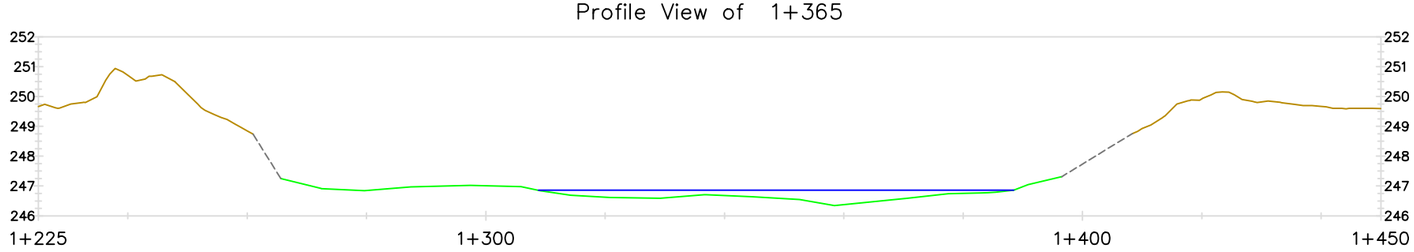


Profile View of 1+365

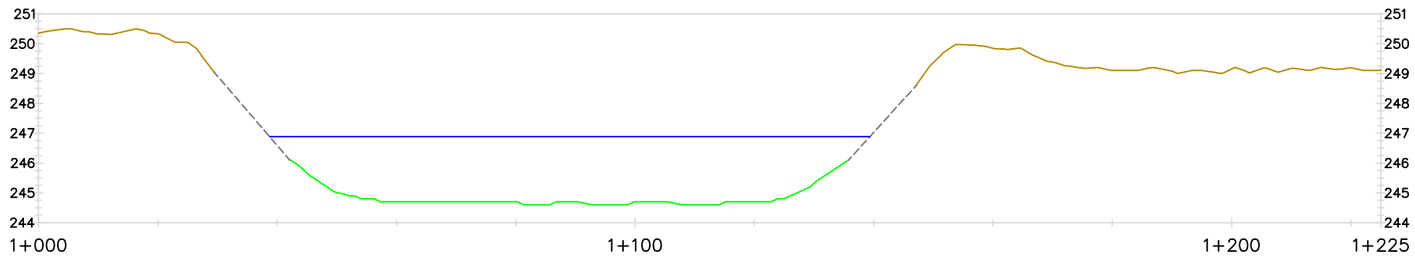


LiDAR
SONAR/TOPO
INTERPOLATED
WATER LEVEL

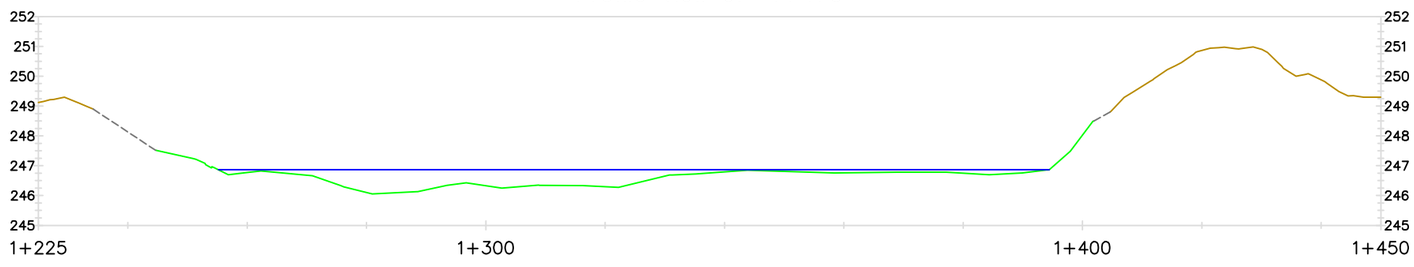
Profile View of 1+365



Profile View of 1+470



Profile View of 1+470



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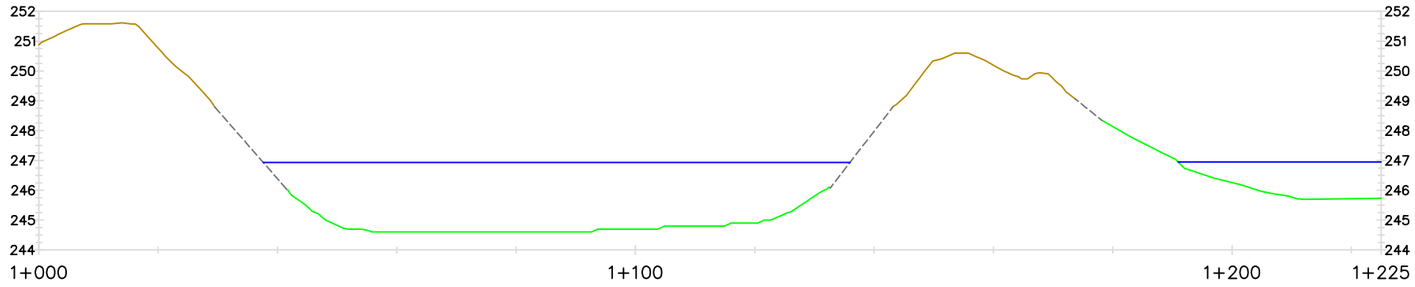
JULY 10/15
1 : 1000 5X VERT.

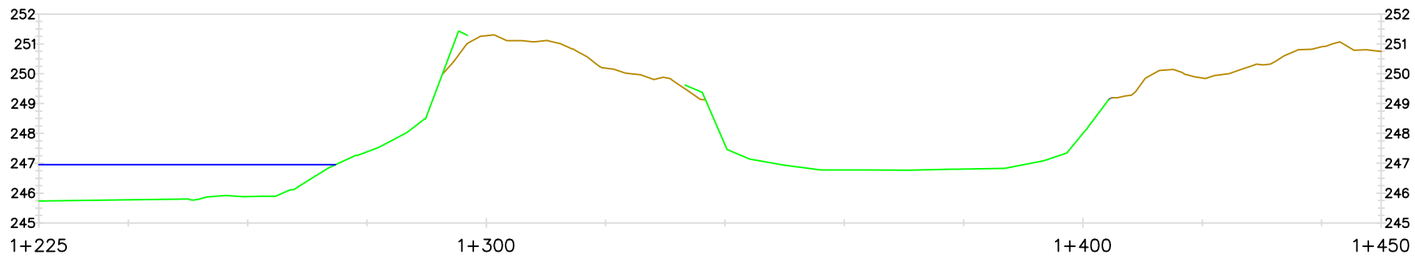
LiDAR
SONAR/TOPO
INTERPOLATED
WATER LEVEL

FAIRFORD INLET PROFILES

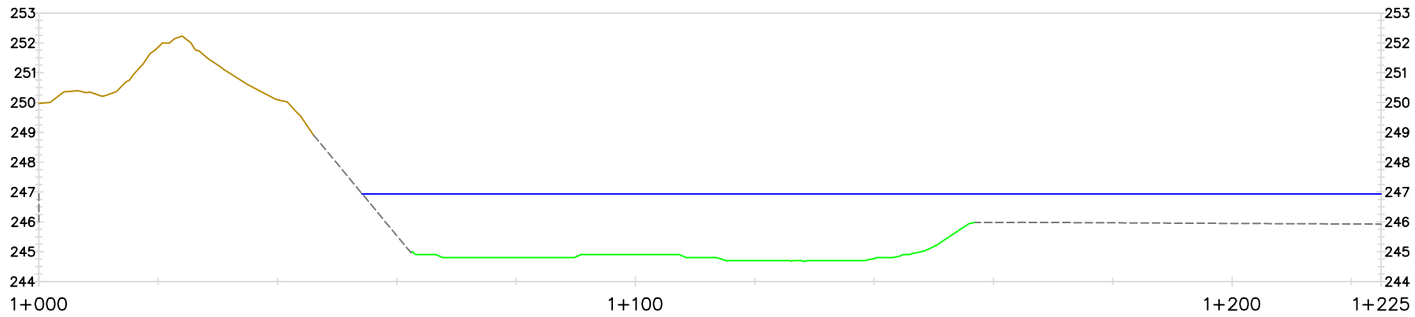
Profile View of 1+528



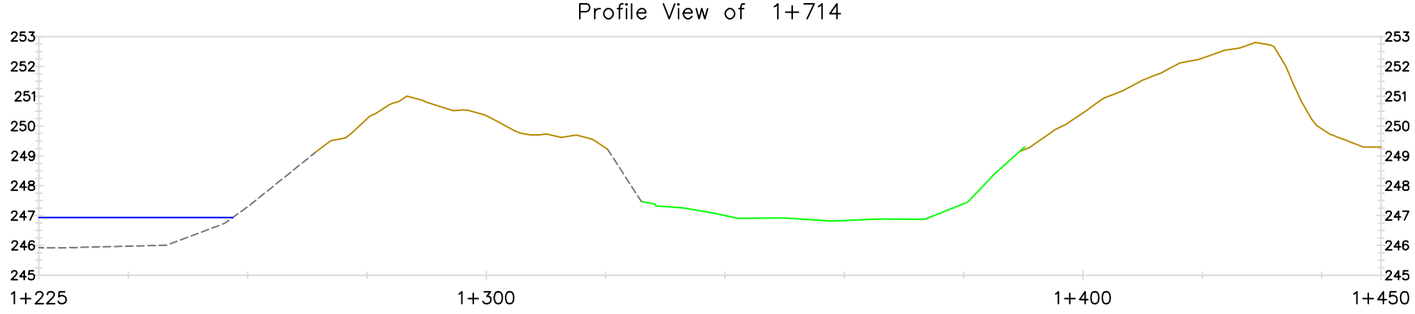
Profile View of 1+528



Profile View of 1+714



Profile View of 1+714



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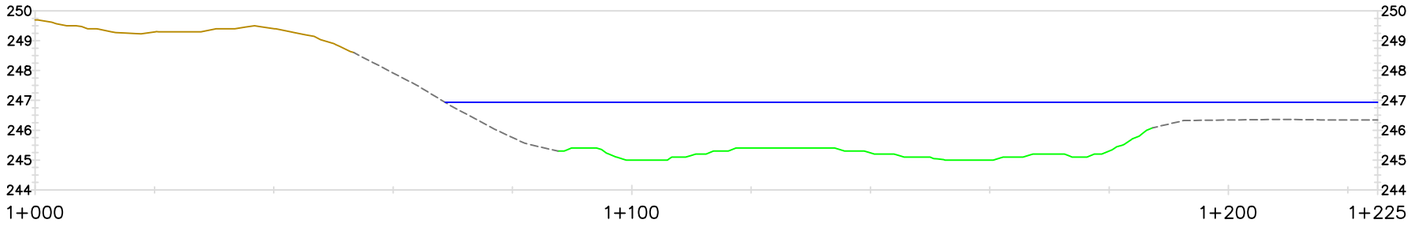
JULY 10/15
1 : 1000 5X VERT.

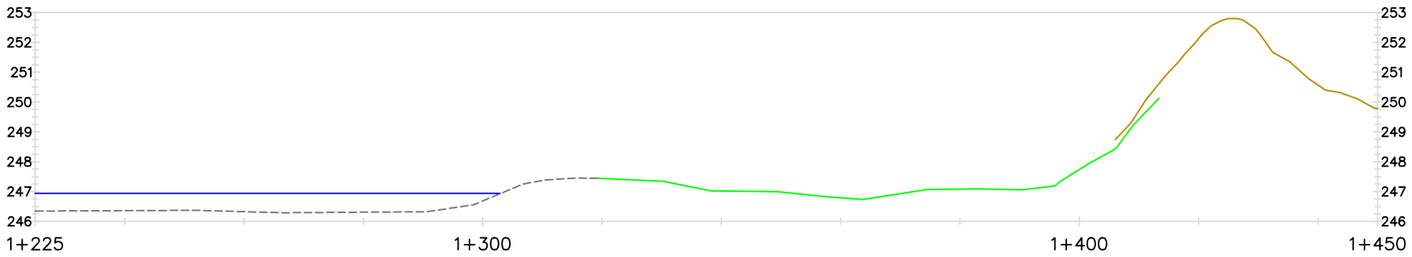
LiDAR
SONAR/TOPO
INTERPOLATED
WATER LEVEL

FAIRFORD INLET PROFILES

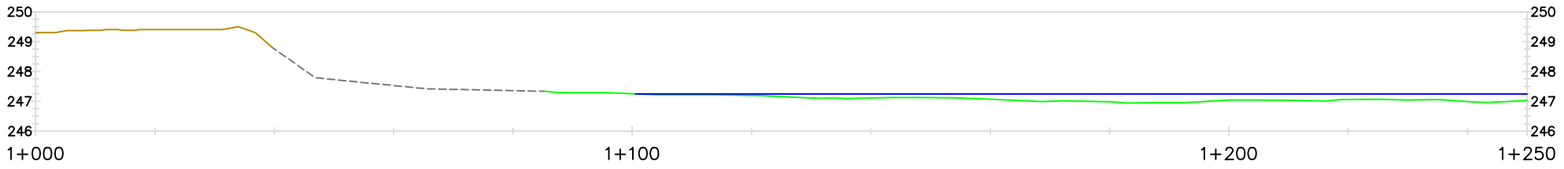
Profile View of 1+803



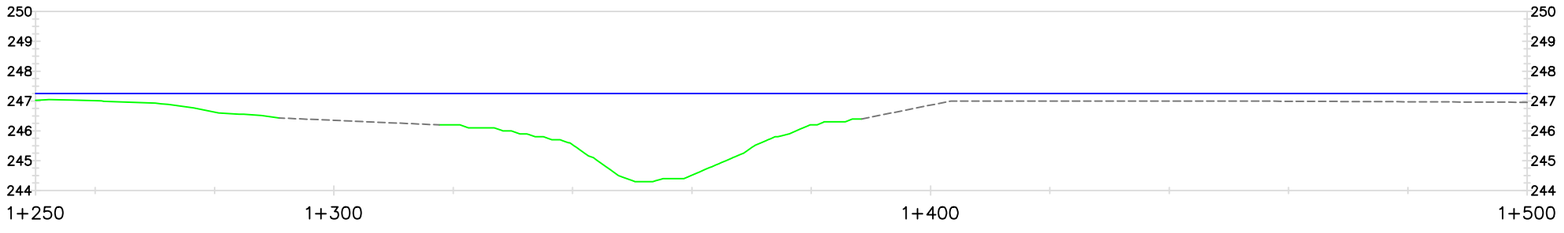
Profile View of 1+803



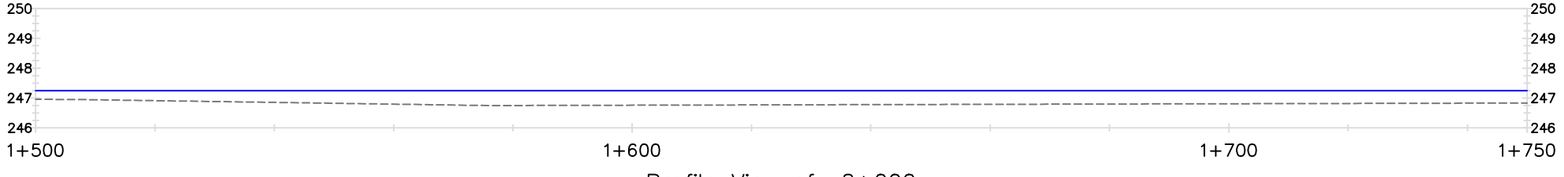
Profile View of 2+000



Profile View of 2+000



Profile View of 2+000



Profile View of 2+000

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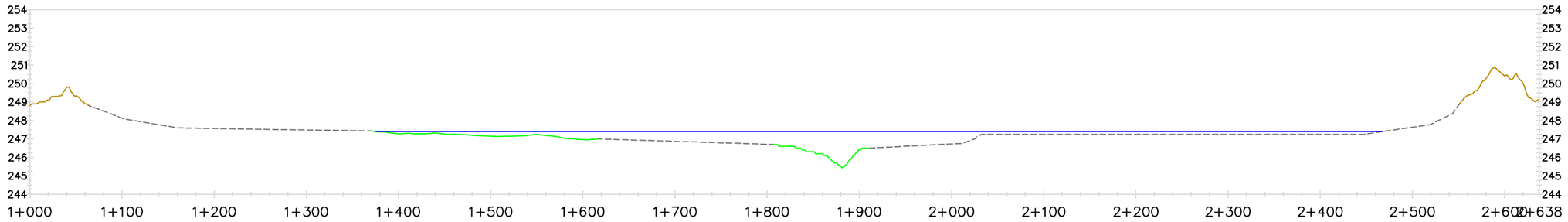
JULY 10/15
1 : 5000 20X VERT.

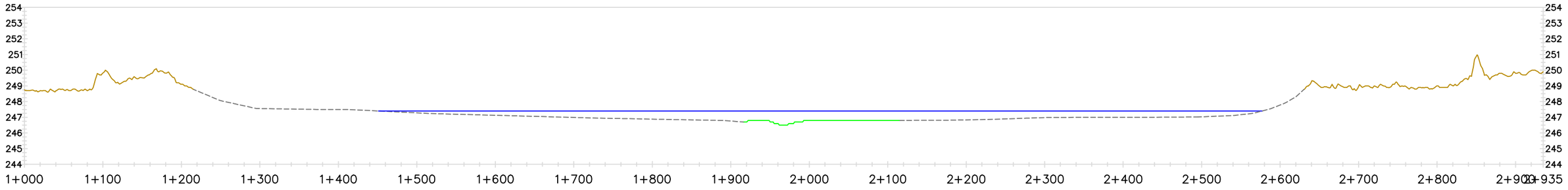
LiDAR
SONAR/TOPO
INTERPOLATED
WATER LEVEL

FAIRFORD INLET PROFILES

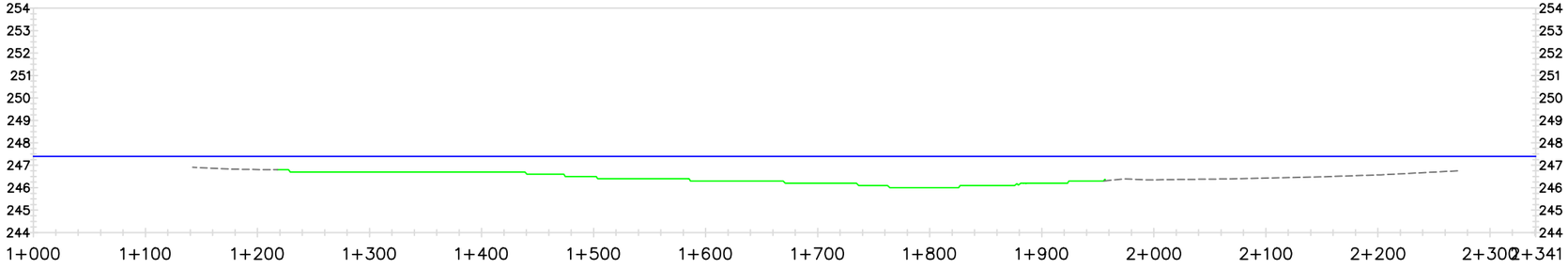
Profile View of 2+200



Profile View of 2+400



Profile View of 2+740



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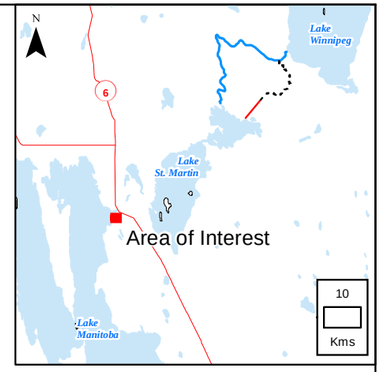


LEGEND:

- Alignment
- 1m Index Sonar Contour
- 0.25m Sonar Contour

NOTES:

1. Aerial Imagery: July 2011.
2. Original ground surface based on LiDAR provided by Atlis Geomatics (June/July 2011).

50 0 50 100 150 200
Metres

SCALE: 1:5,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	15/07/10	ISSUED WITH DRAFT REPORT	PAL

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

BATHYMETRY OF ROUTE B AND C INLET LOCATIONS

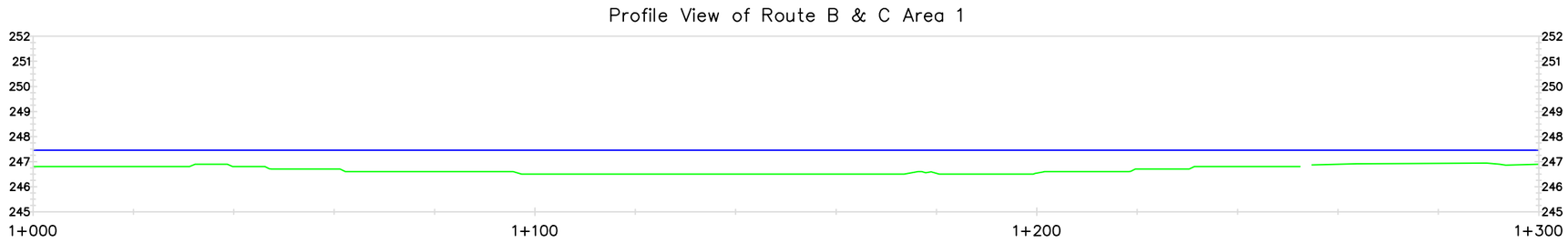
JULY 2015 | FIGURE P-2 | REV: A

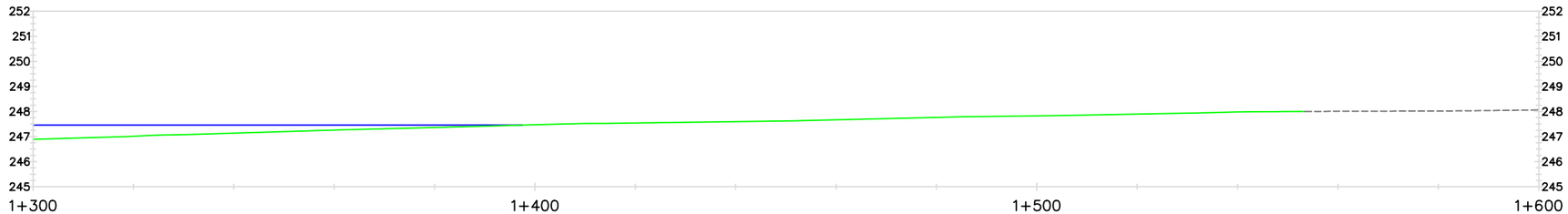
FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\RevA\Final Report\12-0300-011_FigP2_RevA.mxd
11"x17" PLOT SCALE 1:1

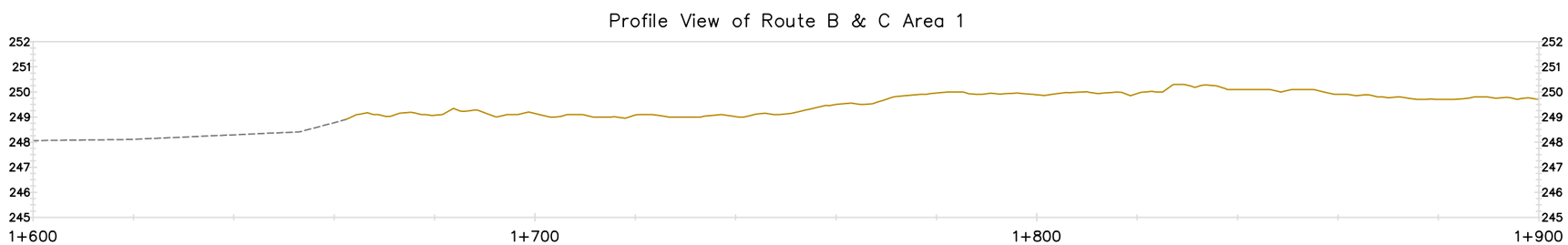
JULY 10/15
1 : 1000 5X VERT.

LiDAR
SONAR/TOPO
INTERPOLATED
WATER LEVEL

ROUTE B & C INLET PROFILES







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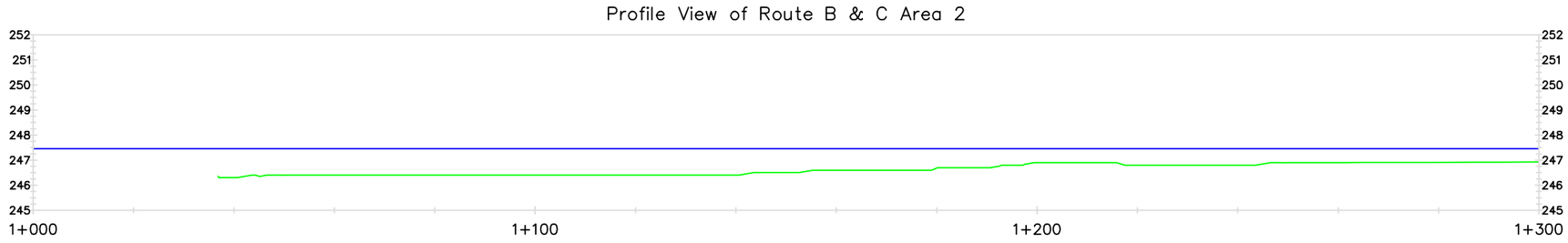
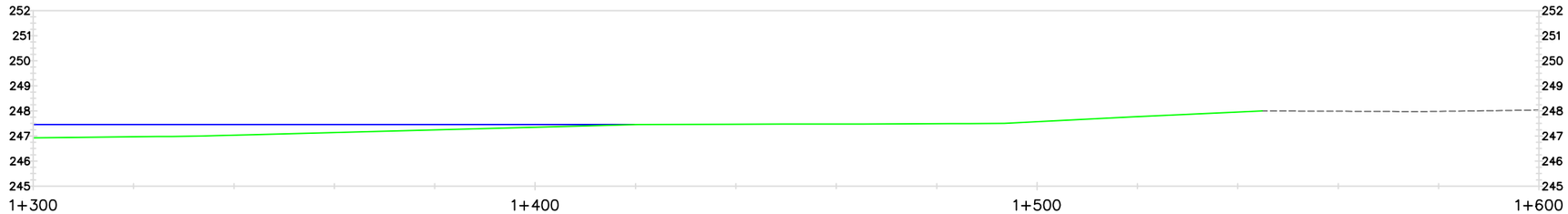
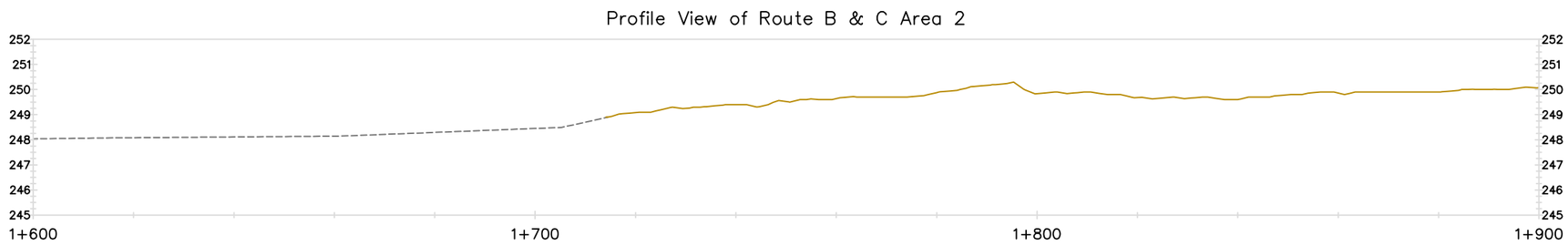
JULY 10/15
1 : 1000 5X VERT.

LiDAR
SONAR/TOPO
INTERPOLATED
WATER LEVEL

ROUTE B & C INLET PROFILES







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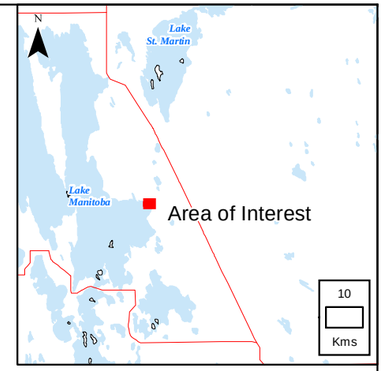

LEGEND:

- Alignment
- 1m Index Sonar Contour
- 0.25m Sonar Contour

NOTES:

1. Aerial Imagery: July 2011.
2. Original ground surface based on LiDAR provided by Atlis Geomatics (June/July 2011).

50 0 50 100 150 200
Metres

SCALE: 1:6,000 METRIC 11"x17"

All units are metric and in metres unless otherwise specified.
Transverse Mercator Projection, NAD 1983, Zone 14
Elevations are in metres above sea level (MSL)

NO.	YY/MM/DD	DESCRIPTION	BY
A	15/07/10	ISSUED WITH DRAFT REPORT	PAL

REVISIONS / ISSUE

KGS GROUP CONSULTING ENGINEERS

Manitoba INFRASTRUCTURE AND TRANSPORTATION

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

BATHYMETRY OF ROUTE D INLET LOCATIONS

JULY 2015 | FIGURE P-3 | REV: A

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\RevA\Final Report\12-0300-011_FigP3_RevA.mxd
11"x17" PLOT SCALE 1:1

JULY 10/15
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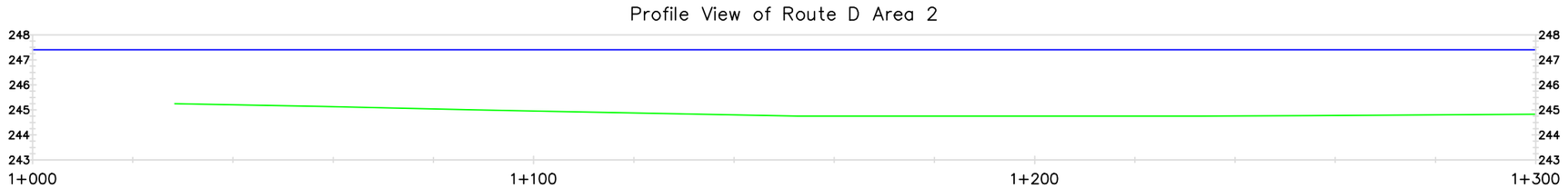
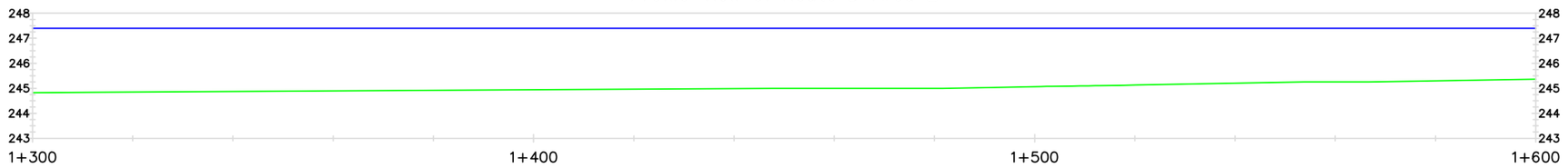
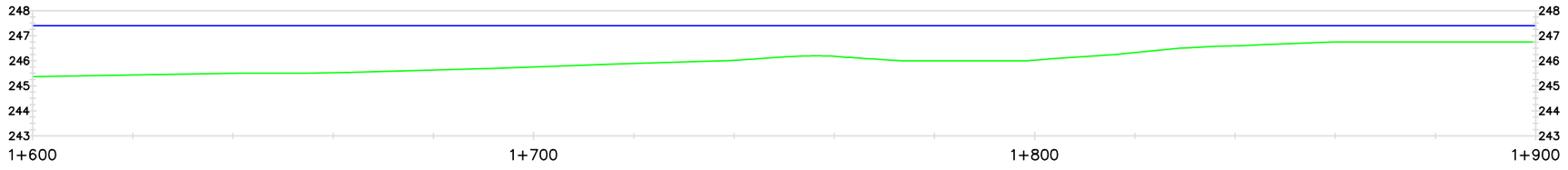
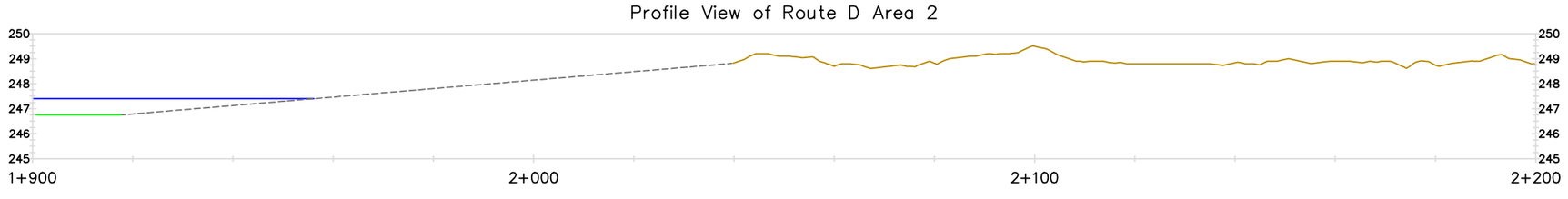
LiDAR
SONAR/TOPO
INTERPOLATED
WATER LEVEL

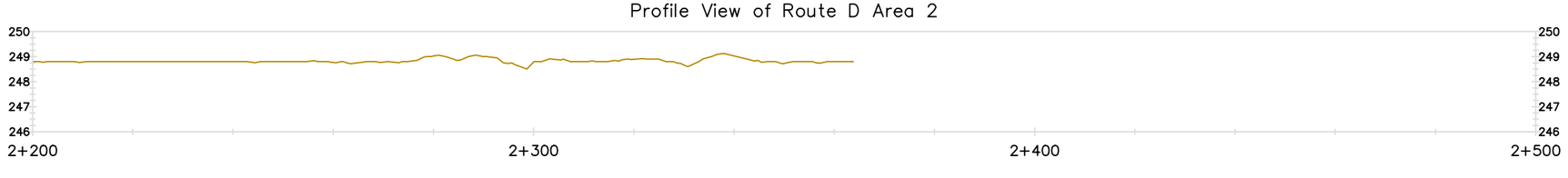
ROUTE D INLET PROFILES











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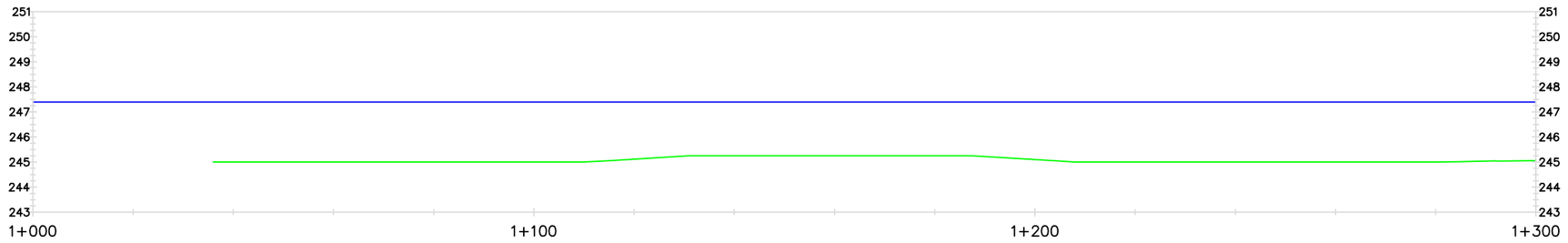
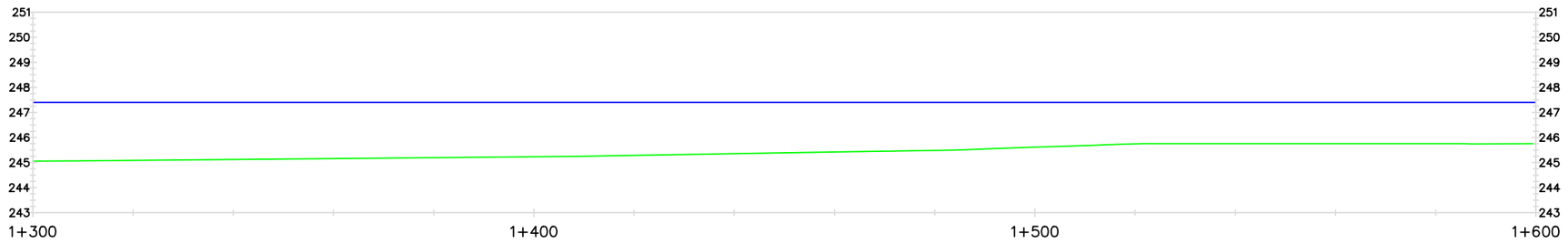
LiDAR
SONAR/TOPO
INTERPOLATED
WATER LEVEL

ROUTE D INLET PROFILES

Profile View of Route D Area 1



Profile View of Route D Area 1



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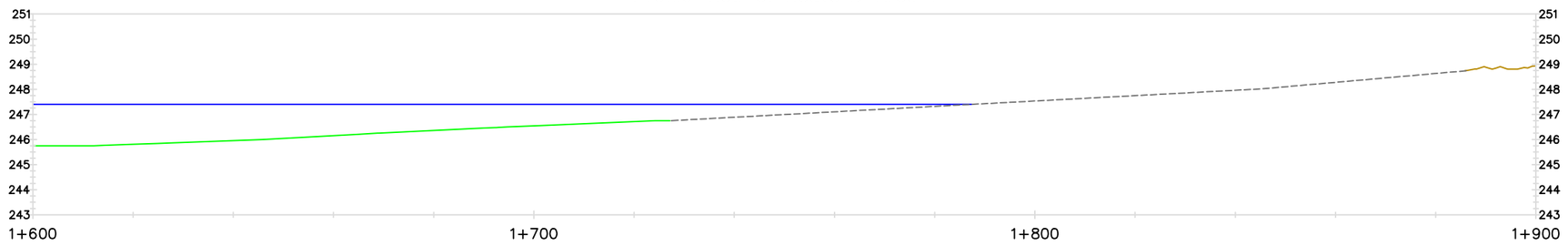
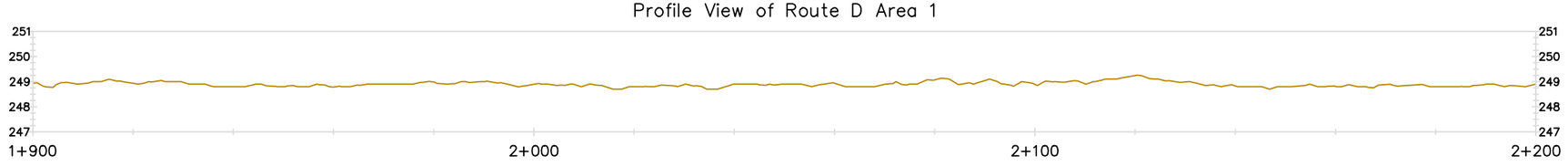


Profile View of Route D Area 1



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APPENDIX Q

REPORTS OF THE PUBLIC OPEN HOUSE EVENTS

APPENDIX Q 1
ASSINBOINE RIVER & LAKE MANITOBA BASINS
FLOOD MITIGATION STUDY
ROUND 1 PUBLIC CONSULTATION REPORT

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ASSINIBOINE RIVER & LAKE MANITOBA BASINS

FLOOD MITIGATION STUDY

ROUND 1 PUBLIC CONSULTATION REPORT

Prepared by:

InterGroup Consultants Ltd.

500-280 Smith Street

Winnipeg, MB R3C 1K2

Submitted to:

KGS Group

865 Waverly Street, 3rd Floor

Winnipeg, MB R3T 5P4

November 2013

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1.0 BACKGROUND

Manitoba Infrastructure and Transportation (MIT) commissioned a conceptual-level study for the Assiniboine River and Lake Manitoba watersheds (the Study Area). The purpose of the study is to provide the Province with recommendations on how future flood events can be mitigated along the main stems of the Assiniboine River and Souris River, Dauphin Lake, Lake Manitoba, Lake St. Martin, and the Shoal Lakes. A map of the Study Area is included in Appendix B1.

The study will include examination of a wide range of possible measures, including non-structural measures, such as land use changes, as well as structural measures, such as large dams. The results of this study will be the foundation for flood mitigation programs for the next few decades.

One major component of the study is the public consultation process, which serves as an opportunity to obtain input from the public.

To prepare for public consultation activities, a draft public consultation strategy (PCS) was developed to help facilitate internal discussions related to identification of potential stakeholders, proposed consultation activities, and overall study messaging. This included creating opportunities for:

1. The Study team to share information with stakeholders;
2. Stakeholders to ask questions and request more information;
3. The Study team and stakeholders to have discussions about potential effects, enhancement of benefits and mitigation options;
4. Addressing interests and concerns as they arise throughout the process; and
5. Facilitating the regulatory review and approvals by ensuring that an open and transparent public process is documented for the regulators.

1.1 OBJECTIVES

The following principles guided the development and implementation of the public consultation activities:

- **Accessible process:** Provide opportunities and sufficient time for potentially affected and interested parties to learn about the study and provide input.
- **Transparent and open process:** Encourage open dialogue and provide information in a form easily understood by non-experts.
- **Variety of communication mechanisms:** Encourage participation through a variety of mechanisms.
- **Responsive approach:** Consider results of the consultation process and provide feedback on the results of the process.

- **Adaptive approach:** Adjust the engagement program in response to issues and concerns.

1.2 STAKEHOLDER IDENTIFICATION

Engagement activities provided information to a broad array of individuals and organizations in the study region. The PCS helped identify an initial list of potential target audiences for engagement activities. This list was later refined by the technical committee and study team. The following categories were targeted for engagement:

- **Community leadership:** Included Reeves and CAOs (Rural Municipalities), Mayor and Councillors (Cities and Towns), and leadership and representatives from First Nation and Metis communities; and

- **Local residents:** Residents within the Study Area including businesses, landowners, interested groups and other interested individuals.

1.3 TWO ROUNDS OF ENGAGEMENT

The PCS identified two rounds of consultation activities.

- **Round 1: Study Introduction and Information Collection –** The first round of consultation, held in June 2013, effectively communicated as much information as possible regarding the scope of the study. Input from participants was solicited in order to understand their thoughts on what mitigation options should be considered to reduce flood effects and how these options should be prioritized, as well as their preferences for future consultation processes. The perspectives on preferred consultation mechanisms will influence the design of Round Two consultation.

- **Round 2: Preliminary Results –** The second and final round of consultation, scheduled for the spring of 2014, will present an opportunity for stakeholders to review preliminary results and provide feedback prior to the development of the final study report. This round will also inform participants about any changes in the Study since Round 1, respond to questions, which were raised, and illustrate how input received in the prior round influenced the study report.

2.0 ROUND 1 PUBLIC CONSULTATION SUMMARY

The first round of consultation communicated information about the scope of the study, including the objective of the study, the overall scope of the study, anticipated timelines, and how people can get involved. Input from participants was solicited in order to understand their experiences and concerns, as well as their preferences for future consultation processes.

2.1 EVENT COMMUNICATION

Community leadership within the study region were identified and mailed an invitation to briefing sessions being held prior to the public open houses.

Advertisements were placed in various newspapers to notify the public of the upcoming open houses.

Copies of the key stakeholder invitation letter and newspaper advertisement are provided in Appendix B1.

2.2 ROUND 1 PUBLIC CONSULTATION MATERIALS

Materials prepared for Round 1 activities included the following:

- **Storyboards:** Storyboards were used to communicate key messages of the study. These included: information on why the study is being conducted; a highlight of potential mitigation options; benefits, costs, priorities; study process and schedule; and the process moving forward (consultation activities and opportunities to participate).

- **Comment forms:** Attendees at both the leadership sessions and open houses were asked to share their concerns/issues about the proposed Study, as well as provide feedback on the public involvement process. Questions and comments received during the stakeholder briefing sessions were recorded and summarized. For some discussions with open house participants, a study team member documented information provided by participants. More than 100 comment forms were distributed during Round 1 public engagement. Some attendees took additional copies for those who were unable to attend the sessions. Participants were given the option to fill out a form at the event, mail the form in, or email comments to the Study email address – feedback@floodstudy.ca.

Copies of the storyboards and comment forms used during Round 1 are provided in Appendix B2.

2.3 ROUND 1 PUBLIC CONSULTATION EVENTS

Information was shared at a total of six events: three meetings with community leadership and three public open houses. Events were held at the following locations:

1. Dauphin, MB: Super 8 Hotel, June 18, 2013;

2. Brandon, MB: Keystone Centre, June 19, 2013; and

3. Portage la Prairie, MB: Canad Inns, June 20, 2013.

2.3.1 Key Stakeholder Briefing Session

June 18-20, 2013, 1:00 pm – 3:00 pm

Key stakeholder briefing sessions were held to provide local leadership (municipal leadership, First Nation Chiefs, and Manitoba Metis Federation) with current information regarding the study in advance of the public open house. It allowed community leadership to be informed of the study process should questions arise from their constituents. Two weeks prior to these events, identified stakeholders were mailed invitation letters that included a study area map and an open house poster. Table 2-1 provides a list of local leadership that participated in the key stakeholder briefing sessions. Copies of the mailing list and the invitation letter are provided in Appendix B1. Meeting summaries are included in Appendix B3.

Table 2-1: Key Stakeholder Briefing Session Attendance

Location	Attendees
Dauphin Super 8 Hotel June 18, 2013	• Brian Sigfusson, Reeve, RM of Coldwell • Hope Sigfusson, RM of Coldwell • Wes T. Bernat, Councillor, City of Dauphin • Clinton Cleave, Reeve, RM of Ochre River • Lyle Finney, Councillor, RM of Alonsa
Brandon Keystone Centre June 19,2013	• Rick Saler, Councillor, Town of Minnedosa • Ken Jenkins, CAO, Town of Minnedosa • Wendy Davidson, Reeve, RM of Archie • Olive McKean, Reeve, RM of Miniota • Alvin Zimmer, Reeve, RM of Shellmouth • Denis Carter, Reeve, RM of Woodworth • Sandy Sanderson, Reeve, RM of Glenwood • Sandra Couling, Councillor, RM of Whitehead • Joe Goodwill, Deputy Reeve, RM of Glenwood • Rick Plaisik, Reeve, RM of Sifton • Stan Cochrane, Councillor, RM of Sifton • Herb Mahood, Director of Regional Operations, Manitoba Infrastructure and Transportation
Portage la Prairie Canad Inns June 20, 2013	• Earl J. Porter, Mayor, City of Portage la Prairie • Kam Blight, Reeve, RM of Portage la Prairie • Wally Melnyk, CAO, Town of Russell • Doug Pickell, Councillor, RM of Rockwood • David Single, Reeve, RM of Westbourne • Daryl Hrehirchuk, CAO, RM of Portage la Prairie • Harold Purkess, Reeve, RM of Victoria • Ivan Bruneau, RM of Victoria • Don Walsh, Reeve, RM of Woodlands • Eilleen Lynxleg, Tootinaowaziibeeng Treaty Reserve • Robert Thiessen, Councillor, RM of St. François Xavier • Roland Rasmussen, Reeve, RM of Cartier • Randi Thomas, Manitoba Metis Federation

2.3.2 Public Open Houses

June 18-20, 2013, 4:30 pm - 8:30 pm

Following meetings with local leadership, public open house events were held to provide opportunities for the general public to learn about the study. These open house events also allowed the public to share their own experience and local knowledge of recent flood events. Study team members were present to walk attendees through the storyboards, provide answers to questions, and receive input to the study. Participants were encouraged to fill out comment forms. Total attendance for the Round 1 public open houses was approximately 49 people.

2.4 SUMMARY OF ROUND 1 PUBLIC CONSULTATION INTERESTS AND CONCERNS

While stakeholders shared different perspectives on the study, many common themes emerged throughout the discussions. Stakeholders seemed genuinely enthusiastic about the study and appreciated the effort made to share and collect information. Table 2-2 provides a summary of common themes heard throughout Round 1 Public Consultation.

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Table 2-2: Summary of Round 1 Public Consultation Interests and Concerns

	Interests/Concerns		
Topic	**Dauphin**	**Brandon**	**Portage la Prairie**
Current and completed provincial activities	• General questions about the Lake St. Martin emergency channel construction. • Construction of the Lake Manitoba Outlet Channel should begin immediately.	• Are any efforts are being made within government departments to coordinate all of the flood-related studies completed to date? • Interest in the operation of the Lake St. Martin Outlet Channel, included design flow of the outlet channel, whether or not it can be closed, and future plans. • How will past recommendations regarding storage reservoirs on the Assiniboine River be incorporated in this study?	• Request for executive summaries of all recent flood-related studies completed by the Province. • Concern that this study will duplicate what was done for other recent studies. • Concerns regarding the operation of the Portage Diversion.
Potential infrastructure options	• Emphasis should be placed on development controls, not just for the buildings but for services (roads, sewer, power, etc) to prevent secondary damage. • The study should not consider diverting water from Shoal Lakes into Lake Manitoba because there are already issues at Lake Manitoba. • The Government of Manitoba Mines Branch may have information on bedrock for diversion options between Lake Manitoba and Lake St. Martin. • Large diversion channels are necessary to accommodate quick runoff.	• Interest in the feasibility of constructing another dam. • The Province should focus on maintaining current infrastructure. • Concern that there is a lack of ongoing attention to infrastructure and general degradation of flood mitigation assets is occurring along the Assiniboine River. • Suggest investigating water retention as a possibility.	• More work should be done to promote upstream water retention to slow down drainage, including providing farmers with incentives to reduce drainage. • Outflows and Inflows must to be taken into account when opening and closing the Shellmouth Dam. There needs to be better operational timing and notifying landowners when this takes place. A similar approach will be needed if major new water control works are built to mitigate flood risks. • Lake outlets need to match inflows. Existing infrastructure and control structures do not allow for the appropriate upstream and downstream water level management. As a result, accumulated water takes a significant amount of time to recede, unnecessarily creating and preserving flood conditions.
Potential policy and planning options	• Concerns about the lack of regulation on drainage in Alberta and Saskatchewan. • The Province should consider a property tax abatement program for landowners who remove land from production to create wetlands. • Any planning and options analysis should clearly describe that Assiniboine River is already a regulated river. • The Province should find a way to regulate lake levels in a way that accommodates the activities of both farmers and cottagers.	• Concerns regarding flooding as a result of drainage from Saskatchewan. • Too many upstream landowners are enhancing drainage and the impacts are being felt on downstream interests. Frustration was expressed that the focus of the study appeared to be on people residing at the bottom of the hill. • Development of a provincial strategy will be critical to this process.	• Will the study consider flooding more areas through water management protection? If so, when will the water management study be publicly available?

	Interests/Concerns		
Topic	**Dauphin**	**Brandon**	**Portage la Prairie**
Other flood reduction options	• Preference for super sandbags over the smaller sandbags because they are easier to handle and the smaller sandbags are labour intensive and costly to install/remove. • Tiger dams are not effective because they move and can float.	• Support for current temporary mitigation measures such as pumps, aqua dikes, and sandbags.	N/A
Benefits, costs, and priorities	• How does the study propose to define benefits and to whom will these benefits go to? • The level of cost analysis to be included in the final report. • Land buyouts as a potential mitigation measure. • Several discussions occurred regarding compensation as a potential form of mitigation. • Suggest that the study consider imposing a fee for those who choose to remain in flood prone zones, similar to what is done in the United States.	• Interest in how the data used to estimate losses will be collected. • Questions on how unregistered losses will be accounted for in the study. • Overall discussion on compensation as a potential mitigation option. Several concerns were expressed related to the delay, and in some cases lack of compensation. • The same value should be given to those who lose cropland due to flooding as it is for those who lose their residence.	• The study should review the tributaries that flow from Duck Mountain because flood events in this area result in human and environmental health issues (i.e., mercury in the community water supply). • Will potential impact on northern communities be considered in the study, particularly First Nations along Lake Winnipeg? • Questions about potential training and employment opportunities would be available if the Province chooses to move forward with recommended mitigation options. • Concerns regarding the absence of adequate compensation for lost crops. • Producers downstream from the Shellmouth Dam have repeatedly faced excessive flooding and associated losses due to the unplanned and unscheduled release of water. Short and long-term solutions are needed, both related to compensation and to best managing flows.
Study process	• Questions about what environmental review process, if any, would be required. • Cottagers on Lake Manitoba are being told to raise their cottages while at the same time everyone knows a new outlet is being studied and will very likely proceed. They wanted to know why they are being told to spend considerable sums of personal funds to raise structures that are unlikely to flood again once the outlet channel is built.	• Suggest that the study focus on potential mitigation measures for all of Manitoba rather than one specific area because everyone is affected. • While the Province is conducting several studies, it is still slow when it comes to starting projects.	• Interest in how data will be collected for the study. • Suggest the study should review the entire watershed, including Saskatchewan and North Dakota. • Study should compare the relatively small area of the Assiniboine River and Lake Manitoba Basins to the larger Red River, Saskatchewan, and Winnipeg River systems.

	Interests/Concerns		
Topic	**Dauphin**	**Brandon**	**Portage la Prairie**
Public consultation	• Particular attention should be paid to RMs along the Assiniboine River. • Print media advertising should be increased for Round 2 Public Consultation.	• To gain support from farmers and landowners, the Province must show them how they will benefit. • Include food and door prizes to increase participation at future open houses. • Contact individuals who have been affected by recent floods, as well as special interest groups (e.g., Assiniboine Valley Producers).	• Questions regarding expectations for Round 2 Public Consultation. • Concern for the absence of a formal consultation process for First Nation and Metis groups. • Concern that the topic of duty to consult would not be fulfilled (some confusion between study consultation and required consultation if the Province chooses to proceed with recommended forms of mitigation). • The Province should meet separately with First Nations because they generally dislike meeting in public forums outside of their own community. • The Province should work to regain public trust (lack of follow-through). • Increase advertising efforts for Round 2 Public Consultation.

Other	• Dauphin Lake
	  ○ It was suggested that the desired water level at Dauphin Lake was 854 feet.
	  ○ Overall interest in the 2011 flood on Dauphin Lake.
	• Questions on how global warming will be considered as part of the study.
	• Discussion on how the Mossy River Dam has been historically operated.
	• Concern that weather events are becoming more dramatic and severe, adding to the adverse impacts historically associated with flooding in the region.
		• How does the study propose to consider other issues such as a potential near-term water shortage in Manitoba?
		• Several discussions on issues with debris along the river (e.g., silt deposits, trees).
		• A review and simplification of the current regulatory process for dam construction was suggested.
			• Concerns regarding silt and debris along the shorelines of Lake Manitoba. It was suggested that debris and silt were entering the lake from the Portage Diversion.
			• Chronic high water levels around the Shoal Lakes have forced producers to exit the industry or to relocate all or parts of their operations.

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APPENDIX A
DRAFT PUBLIC CONSULTATION STRATEGY

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ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

ROUND ONE PUBLIC CONSULTATION STRATEGY

Prepared by:

InterGroup Consultants Ltd.

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Winnipeg, MB R3C 1K2

Submitted to:

Manitoba Infrastructure and Transportation

March 28, 2013

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1.0 INTRODUCTION

The Public Consultation Strategy (PCS) is a key element of the proposed P754 Assiniboine River and Lake Manitoba Basins Flood Mitigation Study (the "Project") and is intended to provide meaningful opportunities for people to receive information and provide their perspectives in relation to the Project.

The PCS will seek to identify and understand stakeholder interests and concerns, and respond in appropriate and meaningful ways. This will include creating opportunities for:

- The Project team to share Project-specific information with stakeholders;
- Stakeholders to ask questions and request more information;
- The Project team and stakeholders to have discussions about potential effects, enhancement of benefits and mitigation options;
- Addressing interests and concerns as they arise throughout the process; and
- Facilitating the regulatory review and approvals by ensuring that an open and transparent public process is documented for the regulators.

Public input will help to identify issues and potential effects of the different alternatives of the Project that can be considered and assessed during the study.

2.0 OBJECTIVES

The following principles will guide the development and implementation of the PCS:

- **Accessible process:** Provide opportunities and sufficient time for potentially affected and interested parties to learn about the project and provide input.
- **Transparent and open process:** Encourage open dialogue and provide information in a form easily understood by non-experts.
- **Variety of communication mechanisms:** Encourage participation through a variety of mechanisms.
- **Responsive approach:** Consider results of the consultation process and provide feedback on the results of the process.
- **Adaptive approach:** Adjust the engagement program in response to issues and concerns.

3.0 STAKEHOLDER IDENTIFICATION

Engagement activities will provide information to a broad array of individuals and organizations in the study region. The identification of relevant stakeholders will be done in cooperation with Manitoba Infrastructure and Transportation (MIT). A preliminary list of Project stakeholders is provided in Appendix A.

Preliminary work has been done to identify relevant stakeholders. Due to the size and scale of the study it will be necessary to work with the Study Team (including MIT) to further clarify and confirm the appropriate list of stakeholders. These stakeholders will represent the following categories:

- **Key Opinion and Political Leaders** – To include appropriate rural municipality leadership, associated Mayor and Council as well as First Nation and Metis leadership.

- **Residents, businesses and landowners** – To include residents and businesses within the Project study region. Residents with previous experience with recent floods are likely to have a high degree of interest in this study.

- **Other interested groups or individuals** – Appropriate government departments (e.g., MB Conservation, Manitoba Health and MIT) and non-government organizations (e.g., local, provincial and national environmental organizations that may have an interest in the study).

4.0 ENGAGEMENT OPPORTUNITIES

Given the scope of the project and the length of time for its completion, two rounds of public engagement are proposed to support the study process.

Round 1: Project introduction and information collection – The first round of the consultation will effectively communicate the scope of the study, including hydrologic and hydraulic analyses, flood control infrastructure options, non-structural land use and development options, benefit-cost analyses and environmental and social considerations. Input from participants will be solicited in order to understand their experiences and knowledge of local flooding events as well as their preferences for future consultation processes. The perspective on preferred consultation mechanisms will influence the design of subsequent rounds of consultation.

Round 2: Preliminary Results – The final round of consultation will present an opportunity for stakeholders to review preliminary results and provide feedback prior to the development of the final study report. Comments and input received in Round Two will be used in finalizing the study report. This round will also inform participants about any changes in the Project since Round One, respond to questions that were raised and illustrate how input received in the prior round influenced the study report.

5.0 ENGAGEMENT MECHANISMS

Several methods will be used to facilitate public engagement for the Project. These methods will be designed to ensure that the Project Team and MIT are providing relevant project information, as well as providing effective opportunities to receive information and perspectives from affected and interested public parties. The following mechanisms are proposed to involve the public during the consultation process:

- Public meetings; and
- Written and electronic communications.

5.1 PUBLIC MEETINGS

Two rounds of public meetings are proposed, consisting of key stakeholder meetings and public open houses. The final locations, format and timing of these meetings will be confirmed through discussions between the Study team and Manitoba Infrastructure and Transportation. A preliminary proposed schedule for Round One is provided in Appendix A. Based on experience with other similar studies, the following initial format for these meetings is recommended.

- **Public Open Houses:** Open house style public meetings will be held in three different locations during each of Round One and Round Two. Study team members will be present to walk attendees through the storyboards, provide answers to questions and receive input to the study. Copies of the storyboards will be available for attendees to take with them and they will be provided with comment forms that can be filled out at that time. Notes describing questions, concerns and input will be assembled.
- **Key Stakeholder Meetings:** Prior to each Open House, one or more workshop-style meeting(s) will be held with key stakeholders. Attendance would be by invitation. Study team members would walk the group (approximately 10-15 people) through the Open House storyboards, providing them with an opportunity for attendees to ask questions and provide comments. The workshop format allows a relatively informal opportunity for key stakeholders (such as municipal and First Nation leadership) to receive information in a smaller group setting. Meeting notes from each of these workshops will be prepared and can be attached to the final report.

InterGroup acknowledges that public consultation processes must be adaptive in nature. In the event that additional meetings or approaches are required, we will work with the Study Team (including MIT) to determine appropriate steps.

5.2 WRITTEN AND ELECTRONIC COMMUNICATIONS

During each round of consultation, materials will be prepared to provide information appropriate for the target audience. These materials will include storyboards, a Frequently Asked Questions (FAQ) document and maps as required. Comment forms will be made available to Open House and Stakeholder Meeting

attendees so that they can provide written comments regarding the study. A project email address will be implemented for the same purpose.

6.0 DOCUMENTATION AND DELIVERABLES

Documentation is an important part of the engagement process and efforts will be taken to ensure stakeholder interests and concerns are documented. Upon completion of each round of consultation, a summary document will be prepared that describes the activities undertaken; summarizes the interests and concerns expressed by stakeholders; and includes copies of all materials used during the consultation. These engagement summaries will be included in the final study report.

InterGroup has developed a database to house all of the insights and information received. This will allow for the following:

- Effective categorization into themes;
- Quick triaging of follow-up items to appropriate Study Team personnel; and
- Organization of consultation documentation.

7.0 ROUND ONE PUBLIC CONSULTATION STRATEGY EXECUTION PLAN

The proposed schedule for Round One engagement activities includes the following assumptions:

- Key Stakeholder meetings and open houses must occur between May 23, 2013 and May 31, 2013.
- Key stakeholder meetings with Community Leadership and Aboriginal groups must occur prior to the public open house.
- It has been determined that three public open houses are sufficient for Round One. The preferred locations are the City of Portage la Prairie, the City of Dauphin and the City of Brandon.

Steps expected to be employed in Round One are set out below.

7.1 KEY STAKEHOLDER MEETINGS

Steps in the Key Stakeholder Meeting engagement process are expected to consist of the following:

- Personalized letters sent to key stakeholders notifying them of the study and inviting them to participate in a meeting with the Study Team to discuss the study prior to the public open house. This will include an invitation to the Open House sessions as well.
- Walk through, with Study Team members, of the Open House storyboards, with an opportunity ask questions and discussion the Project.
- Provision of information/engagement materials, including copies of storyboards, frequently asked questions document and comment forms.

7.2 PUBLIC OPEN HOUSES

Steps in the Open House engagement process are expected to consist of the following:

- Advertisements in local newspapers, including the Portage Daily Graphic, Central Herald Leader, Brandon Sun, Dauphin Herald, Winnipeg Free Press and/or others determined in consultation with the Study Team.
- Advertisements in local radio stations, including KX96 Radio (Brandon), Star 94*7 FM Radio (Brandon), CKDM Radio 730 (Dauphin), CFRY AM Radio (Portage la Prairie) and Mix 96 FM Radio (Portage la Prairie) and/or others determined in consultation with the Study Team.
- Open Houses in three different locations (see Table 1 below for list of potential venues).

Table 1: Potential Open House Venues (Round 1)

City	Location	Capacity
Brandon	Brandon University Victoria Inn (Salon 1)	200
Dauphin	Dauphin Friendship Centre Canway Inn & Suites Super 8 Hotel	30-300 30-50 80
Portage la Prairie	PCU Centre – Portage Daily Graphic Canad Inns Centre	50 10-400

7.3 ADDITIONAL EXECUTION DETAILS

Attachment A provides a detailed schedule for Round One consultation. The following products will be developed and circulated for review during the initial weeks of Round One consultation. These are expected to include:

- Finalized stakeholder lists with contact information;
- Draft storyboards;
- Frequently Asked Questions document;
- Invitation letters;
- Newspaper and radio advertisements;
- Draft comment forms;
- Draft logistics plan describing details around Open House and Stakeholder Meeting schedule, confirmed locations, required Study Team attendance, technical requirements, refreshments and other information as required;
- Detailed plans (including logistics) for the open houses and stakeholder meetings; and
- Advertisements for the open house to be published in local newspapers and radio stations.

Attachment B provides a preliminary list of appropriate stakeholders. InterGroup expects that the final list and determination of which parties are asked to participate in key stakeholder workshops (rather than Open Houses) will be determined through consultation with the Study Team (including MIT).

Attachment C provides a sample entry form for the public engagement contact database that will allow the Project team to track contact information, comments received from the public and any required follow-up. Contact and commitment tracking may be done through this database or through other means, to be determined between MIT and the Project team.

7.4 ROUND ONE DOCUMENTATION AND DELIVERABLES

Following completion of the engagement meetings and open houses, the following products will be developed:

- A list of key issues raised during the engagement process;
- A list of all follow-up items; and
- A draft Round One consultation report.

The schedule in Attachment A has been developed to assume that Round One engagement activities will begin May 23, 2013. It also recognizes that flexibility and adaptability are required at all times during an engagement process; therefore, the schedule allows for update meetings with the Project team.

NOTE: Details regarding the Round Two Public Consultation Strategy will be provided as those engagement activities approach. Although the process is expected to be similar, feedback during Round One may necessitate changes in the approach to be used.

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APPENDIX A

Assiniboine River and Lake Manitoba Basins Flood Mitigation Study:

Round One Public Consultation Strategy Execution Timeline

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Task	Responsibility	Due Date
Week of March 11 to March 15		
Draft Public Consultation Strategy (PCS) proposal	IG	-
Week of March 18 to March 22		
Draft PCS	IG	-
Technical Committee Meeting (March 20)	TC	March 20
Week of March 25 to March 29		
IG to complete PCS and send to KGS	IG	March 26
KGS to complete review of PCS	KGS	March 28
Submit PCS to MIT for approval	IG, MIT	March 28
Begin drafting newspaper/radio advertisements	IG	March 26
Begin drafting storyboards	IG	March 26
Begin drafting FAQ document	IG	March 26
Week of April 1 to April 5		
IG to complete draft of text for newspaper/radio ads	IG	April 1
Draft storyboards	IG	April 5
Draft FAQ document	IG	-
Week of April 8 to April 12		
IG to complete draft of storyboards and FAQ document; send to KGS for review	IG	April 10
KGS to complete review of newspaper/radio ads	KGS	April 10
Fisaha & SLC to complete review of newspaper/radio ads	Fisaha/SLC	April 12
Week of April 15 to April 19		
KGS to complete review of storyboards/FAQ	KGS	April 15
IG address comments/edits to storyboards/FAQ; send to Fisaha	IG	April 15
Fisaha to circulate storyboards/FAQ to TC & SLC	Fisaha	April 16
TC & SLC review storyboards/FAQ; send comments back to IG	TC/SLC	April 18
Week of April 22 to April 26		
KGS to complete review of edits to storyboards/FAQ list	KGS	April 22
IG address comments/edits; send back to Fisaha	IG	April 24
Week of April 29 to May 3		
Finalize text for radio advertisement	IG	April 29
Storyboards/handouts sent for Ministry review	MIT	April 29

Task	Responsibility	Due Date
Week of April 29 to May 3 continued		
Submit final radio spot text to all radio stations	IG	April 29
Submit newspaper ad text to Portage Daily Graphic/Central Herald Leader (3 ads?)	IG	May 3
Submit newspaper ad text to Brandon Sun	IG	2-day lead
Week of May 6 to May 10		
Begin media campaign (newspaper/radio)	IG	May 6
Submit newspaper ad text to Dauphin Herald	IG	May 8
Week of May 13 to May 17		
Ministry comments due back to IG	MIT	May 13
IG to address and finalize storyboards/handouts	IG	May 15
Final storyboards/handouts sent for printing	IG	May 15/16
Final storyboards/handouts back from printer	IG	May 21
Week of May 20 to May 24		
Attend key stakeholder workshop (May 23)	IG, MIT, KGS	May 23
Attend Open House – Portage la Prairie (May 23)	IG, MIT, KGS	May 23
Attend key stakeholder workshop (tentative)	IG, MIT, KGS	(May 21-23)
Attend Open House – Dauphin (tentative)	IG, MIT, KGS	(May 21-23)
Attend key stakeholder workshop (tentative)	IG, MIT, KGS	(May 21-23)
Attend Open House – Brandon (tentative)	IG, MIT, KGS	(May 21-23)
Draft workshop/open house meeting notes	IG	-
Week of June 3 to June 7		
Complete drafting workshop/open house meeting notes	IG	June 7
Week of June 10 to June 14		
Review of draft meeting notes	IG, MIT	
Week of June 17 to June 21		
Finalize meeting notes		
Week of June 24 to June 28		
Draft round one public consultation strategy summary report	IG	

APPENDIX B

Assiniboine River and Lake Manitoba Basins Flood Mitigation Study:

Round One Public Consultation Strategy Potential Stakeholder List

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Affiliation/Organization	Name
Community Leadership	
R.M. of Ochre River	Clinton Coeave, Reeve
R.M. of Sifton	Rick Plaiser, Reeve
R.M. Woodlands	Trever King, Reeve
Town of Souris	Darryl Jackson, Mayor
City of Brandon	Ian Christiansen
Local Citizen Groups	
Association of Lake Manitoba Stakeholders	Cheryl Smith, President Don Clarkson, Vice President Lia Naksina, Secretary Roger Gillis, Director@ large Dr. Scott Forbes Scott Greenlay, Director@ large Jane Hook, Treasurer
Shellmouth Liaison Committee	Alvin Zimmerman Lorne Henry
Whitemud Watershed Conservation District	Chris Reynolds, Manager
Assiniboine Hills Conservation District	Jeff Elder, Chair Jack Bolack, Vice Chair
West Interlake Watershed Conservation District	Doug Oliver, Chair
LaSalle Redboine Conservation District	Justin Reid, Manager
Turtle River Conservation District	Bobby Bennet, Manager
Canadian Water Resources Association	Carly Delavau, President
2011 Flood Regulation Review Task Force	David Farlinger, Chair
Lakes Regulation Review Advisory	Harold Westdal, Chair
Agriculture and Agri-Food Canada	Ron Woodvine
Aboriginal Groups	
Southern Chiefs' Organization	Gerald McIvor, Political Assistant
Manitoba Keewatinowi Okimakanak Inc	Michael Anderson, First Nations Representative
Assembly of Manitoba Chiefs	Melissa Hotain, Policy Analyst

Affiliation/Organization	Name
First Nations/Metis in Assiniboine Watershed	
Long Plain First Nation	Chief David Meeches
Dakota Plains First Nation	Chief Orville Smoke
Sioux Valley Dakota Nation	Chief Vincent Tacon
Dakota Tipi First Nation	Chief David L. Pashe
Manitoba Metis Federation	David Chartrand, President Allan Benoit, Senior Policy Advisor
FN – Neighbouring Watershed	
Sandy Bay Ojibway First Nation	Chief Russell Beaulieu
Ebb & Flow First Nation	Chief Nelson Houle
Lake Manitoba First Nation	Chief Barry Swan
Pinaymootang First Nation (Fairford)	Chief Garnet Woodhouse
Little Saskatchewan First Nation	Chief Hector Shorting
Lake St. Martin First Nation	Chief Adrian Sinclair
Dauphin River First Nation	Chief John Stagg
Skownan First Nation	Chief Cameron Catheway
Government Departments	
Manitoba Local Government	David Neufeld

APPENDIX C

Assiniboine River and Lake Manitoba Basins Flood Mitigation Study:

Round One Public Consultation Contact Database Form

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Contact Information

ID: 1

Stakeholder Category: Tier 1

Affiliation/Organization: Manitoba Metis Federation

Title: Mr. First Name: Darcy Last Name: McGregor

Mailing Address

Address Line 1: 500-280 Smith Street Address Line 2: 5th Floor Address Type: Business

City/Town: Winnipeg Province: MB Postal Code: R3C 1K2

Phone Number(s):

Home: Cell:

Work: (204) 942-0654 Fax: (204) 943-3922

Email: Website:

Notes

Record of Consultation

Date	In/Out	Contact Method	Internal Recipient	Summary of Communication	Follow-up	Person Responsible	Status	Additional Notes
					☐			
*					☐			

Contact Information

ID: 1

Stakeholder Category: Tier 1

Affiliation/Organization: InterGroup Consultants

Title: Mr.

First Name: Darcy

Last Name: McGregor

Mailing Address

Address Line 1: 500-280 Smith Street

Address Line 2: 5th Floor

Address Type: Business

City/Town: Winnipeg

Province: MB

Postal Code: R3C 1K2

Phone Number(s):

Home:

Cell:

Work: (204) 942-0654

Fax: (204) 943-3922

Email:

Website:

Notes

Record of Consultation

Date	In/Out	Contact Method	Internal Recipient	Summary of Communication	Follow-up	Person Responsible	Status	Additional Notes
*								

APPENDIX B
ROUND 1 PUBLIC CONSULTATION

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APPENDIX B1
ROUND 1 PUBLIC CONSULTATION CONTACT WITH STAKEHOLDERS

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List of Key Stakeholders Invited to AR & LMB Rd 1 Information Session

Invitation letters sent May 31, 2013

Stakeholder Name	Affiliation/Organization
Rural Municipalities	
Tom Campbell, Reeve	RM of Albert
Stan Asham, Reeve	RM of Alonsa
Wendy Davidson, Reeve	RM of Archie
Bob Conibear, Reeve	RM of Argyle
Garry Wasylowski, Reeve	RM of Armstrong
James Trewin, Reeve	RM of Arthur
Roger Wilson, Reeve	RM of Birtle, RM
Brent Fortune, Reeve	RM of Blanshard
Duncan Stewart, Reeve	RM of Brenda
Wayne Drummond, Reeve	RM of Cameron
Roland Rasmussen, Reeve	RM of Cartier
Victor Baraniuk, Reeve	RM of Clanwilliam
Brian Sigfusson, Reeve	RM of Coldwell
Reg Atkinson, Reeve	RM of Conwallis
Evan Smith, Reeve	RM of Daly
Dennis Forbes, Reeve	RM of Dauphin
Ralph Wang, Reeve	RM of Edward
Guy Huberdeau, Reeve	RM of Ellice
James Boyd, Reeve	RM of Elton
Arne Lindell, Reeve	RM of Eriksdale
Art Potoroka, Reeve	RM of Ethelbert
Richard Hyde, Reeve	RM of Fisher
Susan Boyachek, CAO	RM of Gilbert Plains
Derek Klassen, Reeve	RM of Glenella
M.E. Sanderson, Reeve	RM of Glenwood
Diane Price, Reeve	RM of Grahamdale
Cliff Kutzan, Reeve	RM of Grandview
Ted Tkachyk, Reeve	RM of Grey
Randy Lints, Reeve	RM of Hamiota
Murray Davies, Reeve	RM of Harrison
Wilfred Taillieu, Reeve	RM of Headingly
Robert Misko, Reeve	RM of Hillsburg
Philip Thordarson, Reeve	RM of Lakeview
Kathy Jasienczyk, Reeve	RM of Langford
Richard Funk, Reeve	RM of Lansdowne
Fred Taylor, Reeve	RM of Lawrence
Aurel Pantel, Reeve	RM of Lorne
Larry McLauchlan, Reeve	RM of McCreary
Olive McKean, Reeve	RM of Miniota
Michael McIntosh, Reeve	RM of Minitonas
Calvin Jacobson, Reeve	RM of Minto
Robert McCallum, Reeve	RM of Morton

Invitation letters sent May 31, 2013

Stakeholder Name	Affiliation/Organization
Larry Zurba, Reeve	RM of Mossey River
Rural Municipalities	
Robert Hanson, Reeve	RM of Mountain
Ralph Oliver, Reeve	RM of North Cypress
Neil Christoffersen, Reeve	RM of North Norfolk
David Inkster, Reeve	RM of Oakland
Clinton Cleave, Reeve	RM of Ochre River
James Andersen, Reeve	RM of Odanah
Craig Atkinson, Reeve	RM of Park
Kim Taylor, Reeve	RM of Pembina
Ross Tycoles, Reeve	RM of Pipestone
Earl Porter, Reeve	RM of Portage la Prairie
Lonn Dunlop, Reeve	RM of Riverside
Jim Campbell, Reeve	RM of Rockwood
Edward Levandoski, Reeve	RM of Rosedale
Edward Mychasiw, Reeve	RM of Rossburn
Frances Smee, Reeve	RM of Rosser
Robert Muir, Reeve	RM of Russell
Robert Sharpe, Reeve	RM of Saskatchewan
Alvin Zimmer, Reeve	RM of Shellmouth-Boulton
Rick Plaisier, Reeve	RM of Sifton
Barry Zacharias, Reeve	RM of Siglunes
Fred Dunn, Reeve	RM of Silver Creek
Earl Malyon, Reeve	RM of South Cypress
Craig Spencer, Reeve	RM of South Norfolk
Roger Poitras, Reeve	RM of St. Francois Xavier
Earl Zotter, Reeve	RM of St. Laurent
Maurice Maguet, Reeve	RM of Ste. Rose
Ken Wozney, Reeve	RM of Strathclair
Dennis Schram, Reeve	RM of Strathcona
Lorne Henkelman, Reeve	RM of Swan River
Harold Purkess, Reeve	RM of Victoria
Don Neufeld, Reeve	RM of Wallace
David Single, Reeve	RM of Westbourne
Bruce Henley, Mayor	RM of West St. Paul
Wayne Dobbie, Reeve	RM of Whitehead
Blair Woods, Reeve	RM of Whitewater
Michael Dillabough, Reeve	RM of Winchester
Donald Walsh, Reeve	RM of Woodlands
Denis Carter, Reeve	RM of Woodworth
Cities, Towns, & Villages	
Shari Hirst, Mayor	City of Brandon
Eric Irwin, Mayor	City of Dauphin
Earl Porter, Mayor	City of Portage la Prairie

Invitation letters sent May 31, 2013

Stakeholder Name	**Affiliation/Organization**
Sam Katz, Mayor	City of Winnipeg
Cities, Towns, & Villages	
Dwight Stewart, Mayor	Town of Birtle
Edward Anderson, Mayor	Town of Boissevain
Wayne Blair, Mayor	Town of Carberry
Brian Franklin, Mayor	Town of Deloraine
Val Soltys, Mayor	Town of Erickson
Eileen Clarke, Mayor	Town of Gladstone
Tom Bohun, Mayor	Town of Grandview
Lori Taylor, Mayor	Town of Hartney
Bob Walker, Mayor	Town of Melita
Henry Barkowski, Mayor	Town of Minitonas
Ray Orr, Mayor	Town of Minnedosa
Ken Waddell, Mayor	Town of Neepawa
Jeff Sigurdson, Mayor	Town of Oak Lake
Orest Woloski, Mayor	Town of Rapid City
Todd Gill, Mayor	Town of Rivers
Betty Nykyforak, Mayor	Town of Roblin
Shirley Kalyniuk, Mayor	Town of Rossburn
Christopher Radford, Mayor	Town of Russell
Donald Yanick, Mayor	Town of Shoal Lake
Darryl Jackson, Mayor	Town of Souris
Glen McKenzie, Mayor	Town of Swan River
Jeff McConnell, Mayor	Town of Virden
Marion Meadows, Mayor	Village of Benito
Dale Sawchuk, Mayor	Village of Binscarth
Leanne Hutman, Mayor	Village of Bowsman
Roland Gagnon, Mayor	Village of Elkhorn
Mitch Michaluk, Mayor	Village of Ethelbert
Lyle Smith, Mayor	Village of Gilbert Plains
Robert Jewsbury, Mayor	Village of Glenboro
Larry Oakden, Mayor	Village of Hamiota
William Wiebe, Mayor	Village of MacGregor
Linda Cripps, Mayor	Village of McCreary
Gilbert Mabon, Mayor	Village of Somerset
Rene Maillard, Mayor	Village of Ste. Rose du Lac
Martin Dupont, Mayor	Village of St-Lazare
Gary Williams, Mayor	Village of Waskada
Bruce Gullet, Mayor	Village of Wawanesa
Dan Brown, Mayor	Village of Winnipegosis
First Nations & Metis	
Chief Nelson Burn	Birdtail Sioux First Nation
Chief Frank Brown	Canupawakpa Dakota First Nation
Chief Clarence Easter	Chemawawin Cree Nation

Invitation letters sent May 31, 2013

Stakeholder Name	**Affiliation/Organization**
Chief Orville Smoke	Dakota Plains First Nation
First Nations & Metis	
Chief David Pashe	Dakota Tipi First Nation
Chief John Stagg	Dauphin River First Nation
Chief Nelson Houle	Ebb & Flow First Nation
Chief David Ledoux	Gamblers First Nation
Chief Gilbert Andrews	God's Lake First Nation
Chief James Plewak	Keeseekoowenin Ojibway Nation
Chief Barry Swan	Lake Manitoba First Nation
Chief Adrian Sinclair	Lake St. Martin First Nation
Chief Hector Shorting	Little Saskatchewan First Nation
Chief David Meeches	Long Plain First Nation
President David Chartrand	Manitoba Metis Federation
Chief Eugene Eastman	O-Chi-Chak-Ko-Sipi First Nation
Chief Garnet Woodhouse	Pinaymootang First Nation
Chief Charlie Boucher	Pine Creek Anishinabe Nation
Chief Morris Swan-Shannacappo	Rolling River Anishinabe Nation
Chief Ken Henry Jr.	Roseau River Anishinabe First Nation
Chief Russell Beaulieu	Sandy Bay First Nation
Chief Nelson Genaille	Sapotaweyak Cree Nation
Chief Vincent Tacon	Sioux Valley Dakota Nation
Chief Cameron Catcheway	Skownan First Nation
Chief Francine Meeches	Swan Lake First Nation
Chief Barry McKay	Tootinaowaziibeeng Treaty Reserve
Chief Melville Wabash	Waywayseecappo First Nation
Chief Albina Brass	Wuskwi Sipihk First Nation (Indian Birch)

Manitoba
Infrastructure and Transportation
Steven D. Topping, P. Eng.
Executive Director
Hydrologic Forecasting and Water Management
Water Management and Structures Division
Box 11, 200 Saulteaux Crescent
Winnipeg, Manitoba, CANADA R3J 3W3
T 204-945-7488 **F** 204-945-7419

[Date]

[Name]
[Address]
[City], Manitoba [Postal Code]

Dear [Name]:

RE: ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

Manitoba Infrastructure and Transportation is conducting a conceptual level study of the Assiniboine River and Lake Manitoba watersheds to help better understand and address potential options that could be used to lessen effects from future floods. The study will identify flood vulnerabilities and examine potential mitigation options for communities, as well as, identify potential improvements to existing infrastructure.

This study includes a public engagement process designed to share and receive information and perspectives on potential flood mitigation measures. Three open house events are planned during June within the study area. This engagement process will communicate the scope of the study and offer opportunities for people to provide input to the study.

As an elected official for your community, you or a representative are invited to an information session prior to the start of the open house event. The session will be an opportunity for you to receive the information that will be made available to the general public at the open houses which are scheduled at 5:00 PM after the information sessions.

The information sessions are scheduled for:

- Dauphin: June 18, 2013, 1:00-3:00 PM
  - Super 8 Dauphin, 1457 Main Street South, Highway 10
- Brandon: June 19, 2013, 1:00-3:00 PM
  - Keystone Centre, 1-1175 18th Street
- Portage La Prairie: June 20, 2013, 1:00-3:00 PM
  - Canada Inns, 2401 Saskatchewan Ave W

I have included with this letter a map of the broader study area for your information along with an invitation to the open house event that can be posted in your administration offices.

If you have any questions or comments about the study or information session, please feel free to contact Steve Topping at 204-945-7488 or email at steve.topping@gov.mb.ca.

Yours truly,

<original signed by>

Steve D. Topping, P. Eng.
Executive Director
Hydrologic Forecasting and Water Management

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FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\RevA\Open_House\12-0300-011-F01_RevA.mxd
8.5"x11" PLOT SCALE 1:1

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Open House

Share Your Views concerning the Assiniboine River and Lake Manitoba Basins Flood Mitigation Study

Manitoba Infrastructure and Transportation is conducting a conceptual level study for the Assiniboine River and Lake Manitoba watersheds to determine how future flood risks can be mitigated. The study will identify flood vulnerabilities and examine potential mitigation options for communities, as well as, identify improvements to existing infrastructure.

Public open house events have been organized to introduce the study and provide you with an opportunity to provide input on potential options for mitigating flood risk in the region. This consultation process will describe the scope of the study, including:

- hydrologic and hydraulic analyses,
- flood control and infrastructure options,
- non-structural land use and development options,
- benefit-cost analyses, and
- environmental and social considerations.

Public open houses have been planned at the following locations:

- **Dauphin: June 18, 2013, 5:00-8:00 PM**
  - Super 8 Dauphin, Large Meeting Room, 1457 Main Street S Highway 10
- **Brandon: June 19, 2013, 5:00-8:00 PM**
  - Keystone Centre, Salon A, 1-1175 18th Street
- **Portage la Prairie : June 20 2013, 5:00-8:00 PM**
  - Canada Inns, Ambassador Room H, 2401 Saskatchewan Ave W

Refreshments will be served. Everyone is welcome to attend.

For more information contact:

Steve Topping
Executive Director, MIT
Email: steve.topping@gov.mb.ca

Colin Siepman, P. Eng.
Senior Project Manager, KGS
Email: CSiepman@kgsgroup.com

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2013 PRINT SUMMARY

			Revised
CLIENT:	**Manitoba Infrastructure & Transportation**	**SUMMARY DATE:**	**June 3, 2013**
BRANCH:	**Water Management and Structures**	**REQUISITION:**	**ESR-015479**
CAMPAIGN:	**Assiniboine River & Lake Manitoba Basins Flood Mitigation Study - Open Houses**	**COST CENTRE:**	**200034663**
DESCRIPTION:	**TBD**		
POSITION:	**Well Fwd News - Right Hand Page**		

Publication	Issue Date	Size			(Width)	Lines	Rate	1X Cost	Ins	Net Total	PO	Material Deadline	e-mail
		col	x	agates									
Daily Newspapers:													
Winnipeg Free Press	Sat. June 8, 15	4	x	100	4 9/16"	400						Wed. June 5	ads@freepress.mb.ca
Brandon Sun	Sat. June 8, 15	3	x	100	5.687"	300						Wed. June 5	ads@brandonsun.com
Weekly Newspapers:													
La Liberte (FR)	Wed. June 12	2	x	100	4 1/16"	200						Wed. June 5	production@la-liberte.mb.ca
Portage The Graphic	Tues. June 11, 18	5	x	100	5.041"	500						Thurs. June 6	cindy.makarchuk@sunmedia.ca
Parkland Shopper	Fri. June 14	3	x	100	5.083"	300						Tues. June 4	dhdisplay@wcgwave.ca
MCNA (12 Members)													
	w/o June 3												
Gimli Interlake Spectator	Thurs. June 6	4	x	100	4.033"	400						Mon. June 3	anne@mcna.com
Melita New Era	Fri. June 7	3	x	100	4.875"	300						Mon. June 3	"
Neepawa Banner	Fri. June 7	3	x	100	5.083"	300						Mon. June 3	"
Portage Central Plains Herald	Thurs. June 6	4	x	100	4.033"	400						Mon. June 3	"
Stonewall Argus/Teulon Times	Thurs. June 6	5	x	100	5.083"	500						Mon. June 3	"
Virden Empire Advance	Fri. June 7	3	x	100	4.875"	300						Mon. June 3	"
	w/o June 10												
Brandon Westman Journal	Wed. June 12	3	x	100	4.875"	300						Wed. June 5	anne@mcna.com
Dauphin Herald	Tues. June 11	3	x	100	5.083"	300						Wed. June 5	"
Grandview Exponent	Tues. June 11	3	x	100	5.083"	300						Wed. June 5	"
Melita New Era	Fri. June 14	3	x	100	4.875"	300						Wed. June 5	"
Neepawa Banner	Fri. June 14	3	x	100	5.083"	300						Wed. June 5	"
Neepawa Press	Wed. June 12	3	x	100	4.875"	300						Wed. June 5	"
Russell Banner	Tues. June 11	3	x	100	5.083"	300						Wed. June 5	"
Virden Empire Advance	Fri. June 14	3	x	100	4.875"	300						Wed. June 5	"
Gimli Interlake Spectator	Thurs. June 13	4	x	100	4.033"	400						Wed. June 5	"
Portage Central Plains Herald	Thurs. June 13	4	x	100	4.033"	400						Wed. June 5	"
Stonewall Argus/Teulon Times	Thurs. June 13	5	x	100	5.083"	500						Wed. June 5	"
Swan River Star & Times	Tues. June 11	2	x	100	4.166"	200						Wed. June 5	"
	w/o June 17												
Brandon Westman Journal	Wed. June 19	3	x	100	4.875"	300						Wed. June 5	anne@mcna.com
Dauphin Herald	Tues. June 18	3	x	100	5.083"	300						Wed. June 5	"
Grandview Exponent	Tues. June 18	3	x	100	5.083"	300						Wed. June 5	"
Neepawa Press	Wed. June 19	3	x	100	4.875"	300						Wed. June 5	"
Russell Banner	Tues. June 18	3	x	100	5.083"	300						Wed. June 5	"
Swan River Star & Times	Tues. June 18	2	x	100	4.166"	200						Wed. June 5	"

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APPENDIX B2
ROUND 1 PUBLIC CONSULTATION MATERIALS

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Manitoba

Assiniboine River and Lake Manitoba Basins Flood Mitigation Study:

Providing Greater Protection For Manitobans From Flood Risks

Manitoba

Welcome!

Manitoba Infrastructure and Transportation is studying flooding in the Assiniboine River and Lake Manitoba watersheds and how we can provide greater protection from flooding risks.

The study will help the province better understand the flood risks and the possible options that can be used to reduce effects from future flood events. This study is focused on:

- The main stems of the Assiniboine River and Souris River
- Lake Manitoba
- Lake St. Martin
- Dauphin Lake
- Shoal Lakes

We value your feedback.

- Ask us any questions you have about the study.
- Share your experience with different types of flood protection works.
- Fill out a comment form.

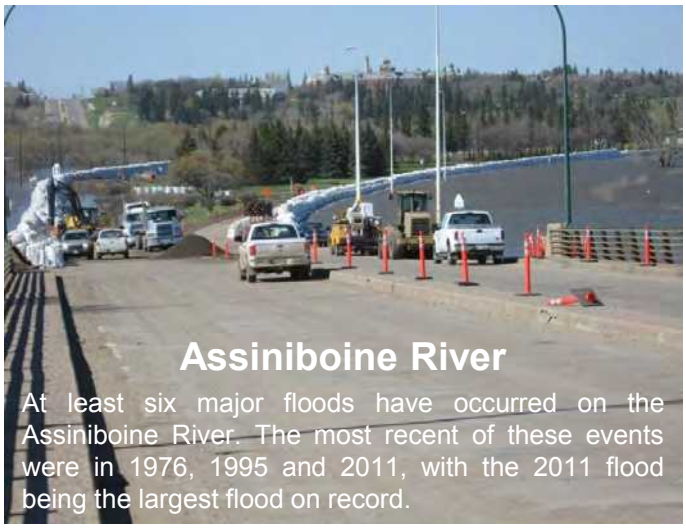
Your input and feedback will help improve the study through local knowledge and experience.

Manitoba

Background

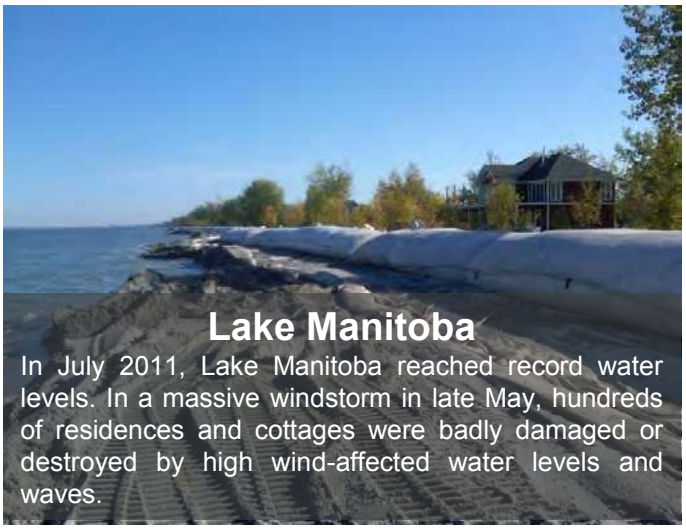
In the last hundred years, Manitoba has experienced several major floods. The flood of 2011 was unique. High flows were recorded on almost all streams and rivers in the Assiniboine River and Lake Manitoba watersheds. For a flood event like this to occur on one or two major rivers is rare, let alone for a flood to occur over all of western Manitoba.

The flood of 2011 highlighted several potential weak links in some of the existing flood control systems.



Assiniboine River

At least six major floods have occurred on the Assiniboine River. The most recent of these events were in 1976, 1995 and 2011, with the 2011 flood being the largest flood on record.



Lake Manitoba

In July 2011, Lake Manitoba reached record water levels. In a massive windstorm in late May, hundreds of residences and cottages were badly damaged or destroyed by high wind-affected water levels and waves.

Souris River

In 2011, the Souris River was affected by several major rainstorms in May and June. The spring peak related to snowmelt runoff occurred in late April and the summer, record-high peak occurred in early July as a result of the significant rain events in May and June.

Dauphin Lake

In 2011, water levels began to rise in early April as a result of the snowmelt. Lake levels continued to rise due to several major rain events in May and June, reaching a record level in mid-June. Very significant damages occurred in the late-May wind storm.

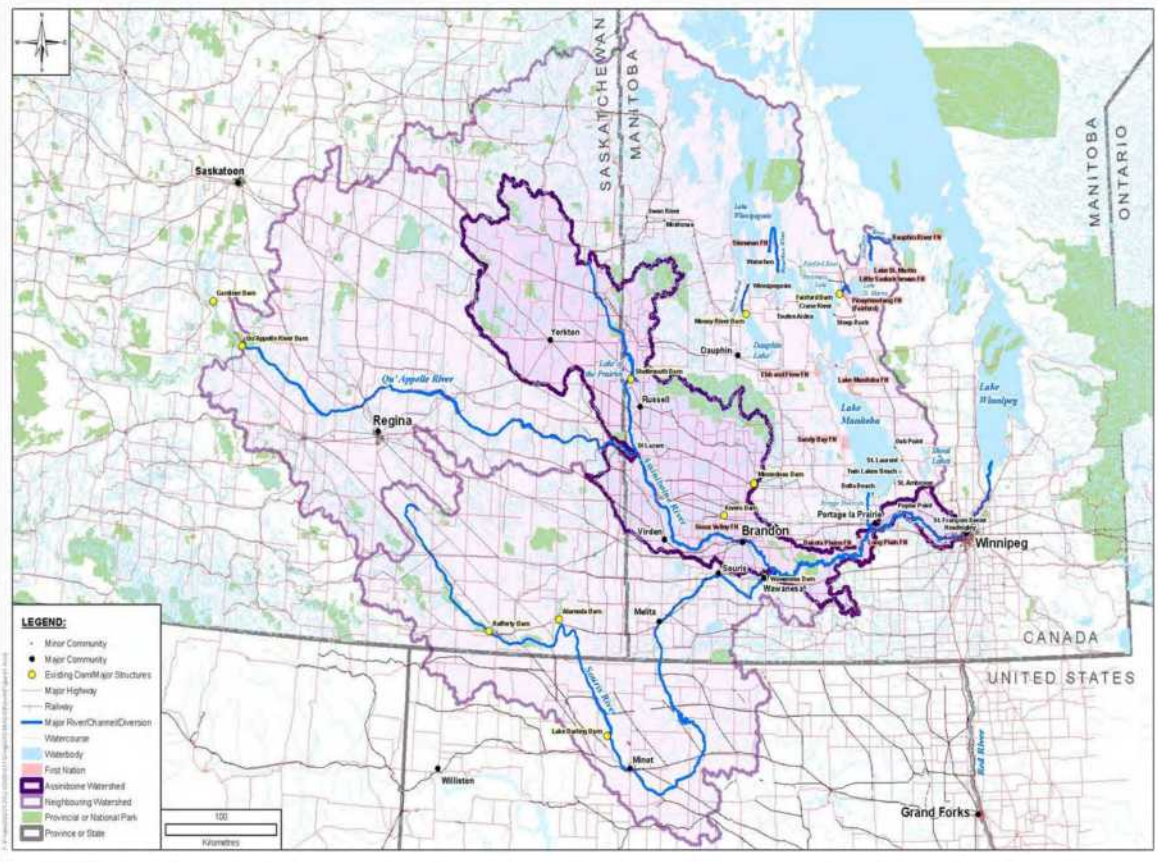
In May 2013, a wind and ice event resulted in significant damages at Ochre Beach.

Study Objectives

The study will identify the flood risk, and assess potential options to reduce flood risk for communities and major infrastructure along the following lakes and rivers.

- Assiniboine River
- Souris River
- Qu'Appelle River
- Dauphin Lake
- Lake Manitoba
- Fairford River
- Lake St. Martin
- Dauphin River
- Shoal Lakes

A number of activities to reduce flood effects are currently well underway and will be considered in the study. Results from this study will be used to plan for and carry out future activities.



Watershed

Manitoba

Other Completed Provincial Activities

The Manitoba Government recently released various studies and reviews including the following:

- **The 2011 Manitoba Flood Review Task Force**

  The Task Force reviewed and provided recommendations on the following components of the 2011 Flood:

  - Flood forecasting.
  - Emergency preparedness, flood fighting ability, and response.
  - Operation of flood control infrastructure.
  - Environmental impacts.
  - First Nations.

- **Lake Manitoba, Lake St. Martin Regulation Review**

  The review considered:

  - The need for additional water control works.
  - The most acceptable and practical range of control for the levels of Lake Manitoba and Lake St. Martin.
  - Land use and zoning rules for areas around the water bodies that are vulnerable to flooding.

- **Lake St. Martin Flood Mitigation Alternatives Study**

  This study reviewed options to reduce flood effects for each of the four First Nations affected by operation of the Fairford River Water Control Structure.

Other Provincial Activities

Other studies and reviews commissioned by the Manitoba Government include:

- **Lake Manitoba Outlet Channel – Conceptual Design Study**

  This study is evaluating options for enhanced Lake Manitoba outlet capacity to better control water levels.

- **Lake St. Martin Outlet Channel – Conceptual Design Study**

  This study is evaluating options for a permanent Lake St. Martin outlet channel, including a review of the approval and licensing requirements of The Environment Act.

- **Surface Water Management Strategy**

  This strategy will develop an integrated approach to improve the management of surface water in Manitoba, considering the needs of Manitobans and the importance of water to the environment.

Lake St. Martin Emergency Outlet (photo courtesy of KGS)

Manitoba

Potential Infrastructure Options

- **Diversion channels**: Review existing and potential diversion channel projects, including improvements to the Portage Diversion and the Fairford River Water Control Structure.
- **Community dikes**: Review existing and proposed dikes, and consider potential new diking options.
- **Flood storage dams and reservoirs**: Review existing and potential dam sites to store water and reduce the risk of flooding.

Shellmouth Dam *(photo courtesy of KGS)*

Potential Policy and Planning Options

- **Development rules and regulations**: Consider ways that future development could be controlled, regulated, and/or restricted to reduce future flood damages.
- **Land use**: Examine the pros and cons of land use changes and identify opportunities to preserve, expand, or enhance key areas to reduce runoff.
- **Wetland retention and restoration**: Consider potential ways of improving and increasing wetlands throughout western Manitoba, and the flood-reduction benefits of these wetlands.

Wetlands *(photo courtesy of KGS)*

Manitoba

Other Flood Reduction Options

Temporary flood mitigation options: Sometimes permanent options to reduce flood effects may not be possible or economical. Temporary options to reduce flood effects will be considered in these cases.

Temporary options may include:

- Sandbags
- Water filled barriers such as Aquadams and Tiger Dams
- Geotubes
- Hesco cage-type barriers

Sandbags

Flood Tubes at Lake Manitoba Basin

(Photos courtesy of KGS)

Manitoba

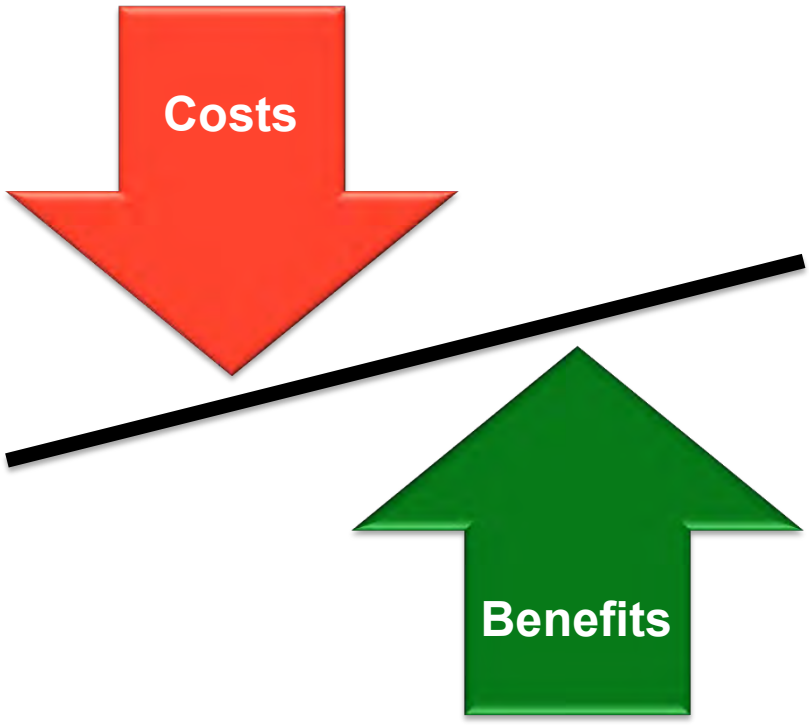
Benefits, Costs, and Priorities

The study will identify potential benefits and costs of the various options to reduce flood effects.

Potential benefits include minimizing:

- Damage to home, cottage, business and farm structures.
- Loss of agricultural land and/or losses in productivity.
- Effects of temporary relocation due to evacuation.
- Transportation restrictions.
- Damage to public infrastructure.
- Temporary mitigation measures (e.g., sandbags, flood tubes).
- Costs associated with flood defence.

The assessment of benefits and costs will help determine the priority of the studied flood reduction and flood protection options.



Manitoba

Study Process

Several steps are required to complete the study.

Data Collection

- Collect and review existing reference reports and data (water levels, flows, flooded areas, rating curves, frequency curves).
- Collect survey information (topographic maps, river cross sections, road and railway elevations).

Identifying Alternatives

- Identify potential options for flood control and reducing weak links in the system.
- Options include: development regulation and rules, diversion channels, dikes, storage dams and reservoirs, land use, wetlands, etc.

Modeling/ Evaluation

- Develop computer models of the river and lake systems and watersheds.
- Evaluate options to reduce flood effects using the models.

Benefit/Cost Analysis

- Identify and assess benefits, costs and other implications of all the alternatives studied at a conceptual level.
- Consider a wide range of flood events including 100-year event, 500-year event, and flood of record.

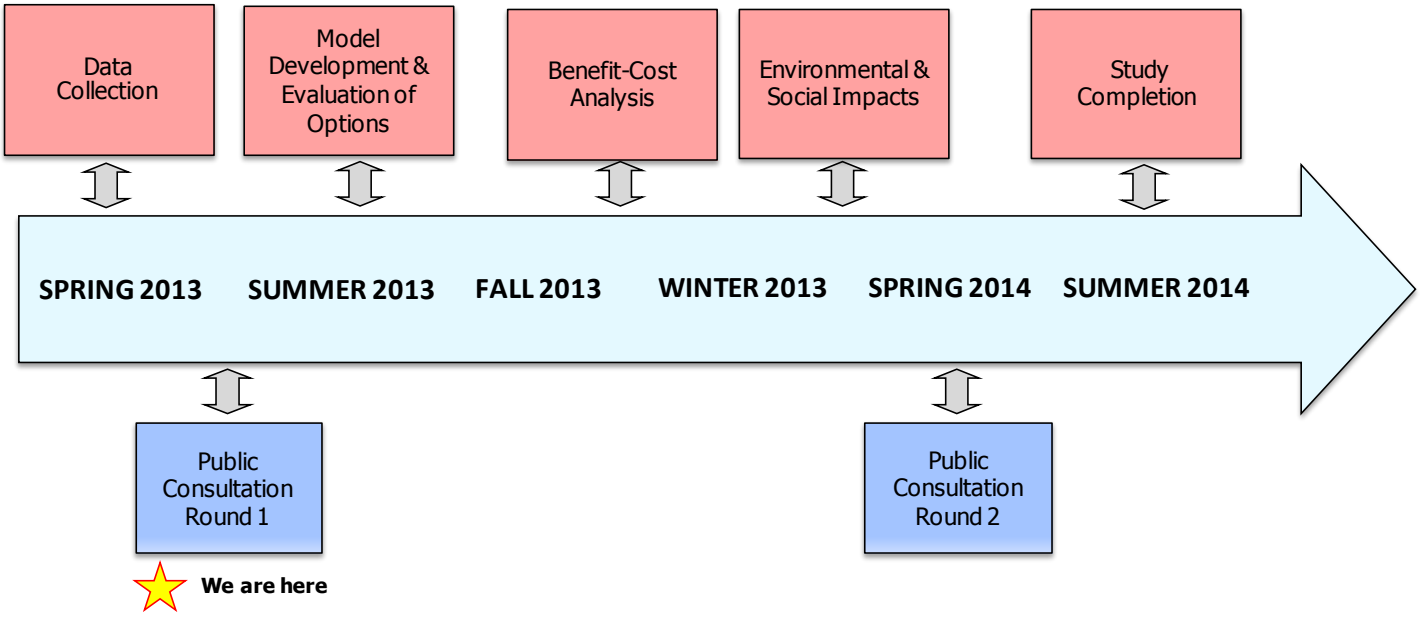
Impacts of Alternatives

- Identify environmental and social impacts of the most feasible options including all potentially impacted resources (fish, wildlife, agriculture, water quality, etc.)

Final Report

- Document options that have been considered, including benefits, costs, impacts, and recommendations.

Study Schedule



Manitoba

Public Consultation

Round 1 Consultation

Today is the first part of the public consultation process. This session is intended to:

- Communicate the scope of the study; and
- Provide opportunity for people to provide thoughts on what options should be considered to reduce flood effects and how these options should be prioritized.

Round 2 Consultation

The second round will be an opportunity to review preliminary results and provide feedback prior to the development of the final study report.

Manitoba

Sharing your opinions, knowledge, and experience

The study team would like to know:

- Based on what you have observed and heard today, what are your thoughts about the study?
- What are your thoughts about the considered flood mitigation options?
- What have been your past experiences with different forms of flood mitigation, particularly those presented today?

Comment forms are available for you to fill out.

Forms can also be submitted electronically or by mail, so long as they are received before July 15, 2013.

Details are provided on the comment form.

Please feel free to approach a study team member to help you through the documentation process.

Contact information: feedback@floodstudy.ca

Mailing address: **Colin Siepman, P. Eng, KGS Group**

KGS Place
3rd Flr - 865 Waverley St.
Winnipeg, MB, R3T 5P4

Thank you!

ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

Please take this opportunity to fill-out this brief questionnaire based on the information provided at this event. Please drop the completed comment form in the box provided. Comments may also be mailed or emailed to the addresses provided on the back of this form.

Overall, was this information session helpful in providing you with a general understanding of the study process?

☐ Yes ☐ No

If no, what additional information would have been helpful?

Based on what you have observed today, what are your thoughts about the study?

What do you think about the considered flood mitigation options (particularly those discussed today)?

What have been your past experiences with different forms of flood mitigation?

Do you have any suggestions on how we could improve the public involvement process?

CONTACT INFORMATION (information will not be used for any purposes other than the study and will not be shared outside the study team)

Name: _____

Address: _____

Telephone: _____ Email: _____

Connection to the study area (check all that apply)

☐ Local area resident

☐ Farmer

☐ Other (please specify _____)

MAILING ADDRESS

Comments may also be submitted via mail or email no later than **July 15 2013.**

Email: feedback@floodstudy.ca

Mailing address: Colin Siepman, P. Eng KGS Group

KGS Place

3rd Floor-865 Waverley St.

Winnipeg, MB R3T 5P4

THANK YOU FOR TAKING THE TIME TO FILL OUT THE COMMENT FORM.
YOUR PARTICIPATION IS VERY IMPORTANT TO THE STUDY PROCESS.

APPENDIX B3
ROUND 1 PUBLIC CONSULTATION SUMMARY INFORMATION

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Stakeholder Mtg 1-3pm.

SIGN IN SHEET FOR THE

ASSINIBOINE RIVER & LAKE MANITOBA BASINS STUDY OPEN HOUSE

JUNE 18, 2013

DAUPHIN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
BRIAN SIGFUSSON	<original signed by>	RM of Coldwell
HOPE SIGFUSSON	<original signed by>	" Coldwell
WES T. BERNAT	<original signed by>	City of Dauphin
CLINTON CLEAVE	<original signed by>	RM of Ochre River
Lyle Finney	<original signed by>	RM of Alonsa

SIGN IN SHEET FOR THE

ASSINIBOINE RIVER & LAKE MANITOBA BASINS STUDY OPEN HOUSE

JUNE 18, 2013

DAUPHIN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
DON WHITE	<original signed by>	DPHW
BOB CRADOCK	<original signed by>	DPHW
Gary & Darryl Warne		DPW
Gary Medwechuk/ Betty		
Betty MEDWECHUK	<original signed by>	Dpw Lake
Roland Magnuson	<original signed by>	Lake Manitoba
Jessica Rawluk	<original signed by>	Dauphin Herald.
Kate Storey	<original signed by>	Grandview
Dawn English		MB MLG
Chuck Lund	<original signed by>	MIT- Retired.

Stakeholder Meeting ended 3:15pm

SIGN IN SHEET FOR THE

ASSINIBOINE RIVER & LAKE MANITOBA BASINS STUDY OPEN HOUSE

JUNE 19, 2013

BRANDON, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
RICK SALEK	<original signed by>	TOWN OF MINNEDOSA
Ken Jenkins	<original signed by>	" " " CAO
Wendy Davidson	<original signed by>	R.M. of Archie (Reeve)
Olive McKean	<original signed by>	R.M. of MINIOTA, Reeve
ALVIN ZIMMER	<original signed by>	R.M of SHELLMOUTH-BOULTON
DENIS CARTIER	<original signed by>	R.M of WOODWORTH
Sandy Sanderson	<original signed by>	RM of Glenwood
SANDRA COULING	<original signed by>	R.M. of Whitehead
Joe Goodwill	<original signed by>	RM of Gleichen
Rick Placsick	<original signed by>	R.M of Sifton
STAN COCHRANE	<original signed by>	R M of SIFTON
HERB MAHOOD	<original signed by>	M I T -

SIGN IN SHEET FOR THE Open House

ASSINIBOINE RIVER & LAKE MANITOBA BASINS STUDY OPEN HOUSE

JUNE 19, 2013

BRANDON, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
SHARI DECTER HIRST	<original signed by>	City of BRANDON
Reg Atkinson	<original signed by>	RM of Cornwallis
Rick Plaun	<original signed by>	AMM Rural Water Director
Keith Tearo	<original signed by>	Farmer - Assiniboine Valley
Fred Taylor		Farmer " "
Rob Mclean	<original signed by>	ReTired
Gerald Smith	<original signed by>	producer
Stan Taylor	<original signed by>	Farmer
Daniel Wa	<original signed by>	Manitoba Co-operato
Ted Saure	<original signed by>	C.T. of Brandon Engineeri
Steve Topping	<original signed by>	MIL
WELDON NEWTON	<original signed by>	
CLIFF REINDER.	<original signed by>	AVP. NORTH
Stacey McBride	<original signed by>	MIT

Stakeholder Meeting 1-3pm

ASSINIBOINE RIVER & LAKE MANITOBA BASINS STUDY OPEN HOUSE

JUNE 20, 2013

PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
EARL J. PORTER	<original signed by>	MAYOR City of P.la.P.
HAM BUGHT	<original signed by>	REEVE RM of PORTAGE
Wanny Mulyk	<original signed by>	Town of Rivers
Doug Pickell	<original signed by>	R.M. of ROCKWOOD
David Single	<original signed by>	RM Westbourne
DARYL HREHIRCHUK	<original signed by>	RM of Portage
HAROLD PUNKESS	<original signed by>	RM VICTORIA
L. Bruneau	<original signed by>	R.M. Victoria.
Don Walsh	<original signed by>	RM of Woodlands
Eileen Lynxleg	<original signed by>	TOOTINAOWAZIIBEENG TREATY RESERVE
Robert Thiessen	<original signed by>	RM of St Francois
Roland Rasmussen	<original signed by>	R.M Cartier
Randi Thomas	<original signed by>	MB Metis Fed.

SIGN IN SHEET FOR THE

ASSINIBOINE RIVER & LAKE MANITOBA BASINS STUDY OPEN HOUSE

JUNE 20, 2013

PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Cheryl Kennedy (Councillor)	<original signed by>	WILDLIFE/FISH
Gerald + Pat Fauvelle		
Lorne Berry		Irrigators rep.
Eric Tosh	<original signed by>	PROVINCIAL SEC.
Susan Yeake	<original signed by>	Delta
Joan Moorhouse	<original signed by>	
Gerry Dell	<original signed by>	Lake MB Narrows
Allen Dell	<original signed by>	Lake MB Narrows
Maureen Cawes	<original signed by>	MB Beef Producers
Robert Tregnell	<original signed by>	Delta
MYRNA HANCOCK	<original signed by>	Delta
DON EASTON	<original signed by>	Twin Lakes Beach
ALAN OWENS	<original signed by>	DELTA BEACH HOME OWNER
RICHARD GRAY	<original signed by>	MANITOBA LOCAL GOVERNMENT
GERRY GRETSINGER	<original signed by>	
RANDY PAINTER	<original signed by>	COTTAGE AT DELTA + FARM PORT. ALONE DIVERS.
Arthur Wiebe	<original signed by>	Former farmer Oakville
Hank Finlayson	<original signed by>	Former farmer
DOUG MCMAHON	<original signed by>	MLT
BARRY RUD	<original signed by>	COTTAGE OWNER
MARLENE RUD		

SIGN IN SHEET FOR THE

ASSINIBOINE RIVER & LAKE MANITOBA BASINS STUDY OPEN HOUSE

JUNE 20, 2013

PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
DAVID W. FAURSCHOU	<original signed by>	PORTAGE ADEL FARMER.

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Assiniboine River & Lake Manitoba Basins Flood Mitigation Study: Round 1 Public Engagement

Title:	**Assiniboine River & Lake Manitoba Basins Flood Mitigation**		
Date:	June 18, 2013	**Minutes Status:**	FINAL
Attendees:	**Participants** Brian Sigfusson, Reeve, RM of Coldwell Hope Sigfusson, RM of Coldwell Wes T. Bernat, Councillor, City of Dauphin Clinton Cleave, Reeve, RM of Ochre River Lyle Finney, Councillor, RM of Alonsa **Study Team** Fishaha Unduche, Manitoba Infrastructure and Transportation Eugene Kozera, Manitoba Infrastructure and Transportation Colin Siepman, KGS Group Patrice Leclercq, KGS Group John Osler, InterGroup Consultants Yadili Okwumabua-Gbakima, InterGroup Consultants		
Meeting Location:	Super 8 Hotel - Dauphin, MB	**Meeting Time:**	1:00 PM – 3:00 PM
Meeting Purpose	The objective of the meeting was to share with local leadership current information regarding the study in advance of the public open house. This will allow community leadership to be informed of the study process should questions arise from their constituents. As well, perspectives provided by participants would be integrated into the study process.		
Introduction	John Osler welcomed participants to the information session, noting this was the first opportunity for people to be consulted on the study. He thanked people for attending and encouraged them to ask questions throughout the presentation. The Study Team was introduced to the group.		

Presentation	John Osler (IG) and Colin Siepman (KGS) presented the storyboards outlining the flood mitigation study. The study will allow the province to gain a better understanding of flood risks and the possible mitigation efforts that can be used to reduce effects of future flood events. Round One communicates the scope of the study, presents an opportunity for stakeholders to provide thoughts on proposed mitigation efforts to reduce flood effects and how these efforts should be prioritized. This information will be considered by the study team in the development of the hydraulic model and the benefit/cost analysis.

SUMMARY OF QUESTIONS AND DISCUSSION

As the presentation was made, a number of questions were raised by those in attendance. Questions and comments are summarized below.

Q. Is the 2011 flood still being considered a 300-year event for Dauphin Lake?

A. At Dauphin Lake, the 2011 flood is currently considered an 80-year event.

Q. What type of year event was the ice storm at Ochre River?

A. This has not been confirmed but it may be a 50-year flood event. The intensity of the ice storm was due to multiple variables aligning at the same time.

C. Would have thought that the ice storm would have occurred in spring of 2012, when the lake levels were high.

Q. What is meant by 'benefits'? Who is benefiting and from what?

A. Damage prevention efforts are considered to be benefits. These efforts may be stakeholder specific. (e.g., storage reservoir could be considered beneficial to downstream residents).

Q. What environmental review process, if any, will be required?

A. When the Portage Diversion was created, rules and regulations were not as stringent as they are today. Depending on what form(s) of mitigation the Province decides to proceed with, each option would likely go through an environmental regulatory process.

Q. Will a cost analysis be included in the final report?

A. A high-level cost review will be included in the benefit/costs analysis.

Q. What effect will global warming have in this study? Is it part of what is driving the increase in weather events?

A. Do not have a clear answer to the question of the influence of global warming on adverse weather events yet.

Q. Will climate change be incorporated into the study?

A. A detailed review of climate change is beyond the scope of the study. A potential upward trend in flood peaks will be considered but the study will not speculate on the specific causes (i.e. climate change, climate variability, land use and drainage changes, etc.).

Q. Will buyouts be an option?

A. It will be included in the review.

C. Buyouts would not be a good option and instead consideration should be made to road closures. Buyouts might make some sense but it would also reduce the overall tax base.

C. A study of this size can only take this alternative so far; further discussion would be required on the buyout options during implementation so potential issues can be flagged; cost was noted as just one component in the final decision on how best to proceed.

Q. Why are Alberta and Saskatchewan allowed to drain water without regulation?

A. Regulation of water drainage in Alberta and Saskatchewan falls outside the scope of the current study.

Q. If Lake Winnipegosis was lowered by 6 feet; will water levels at another lake elsewhere in the province be increased by 6 feet?

A. The impact of altering water levels will be included in the scope of the current study

Q. What impact would Lake Winnipegosis experience if Dauphin Lake was lowered by 6 feet?

A. The impact of altering water levels will be included in the scope of the current study.

Q. How has Mossey River Dam been historically operated? For example, in 2012, it was dry but the dam was still opened. Also, the dam is not wide open and is restricting flow.

A. Follow-up: The Mossey River Dam has maintained maximum cut flows for over two months (June 28, 2013 Media Bulletin-Manitoba). The target level is around 855 fasl (260.60m).

Additional comments:

- Rural Municipalities (RM) along the Assiniboine River should be included in this study. These RMs continue to build as they are protected by insurance while RMs like Alonsa do not benefit from insurance protection (due to restrictions) resulting in a loss of tax revenue.
- Super sandbags are preferred over traditional sandbags because they are easier to handle. The smaller sandbags are labour intensive and costly to install and remove.
- The government does not provide residents with potential flood event information in a timely manner.
    - A participant commented that people should not complain about lack of information because there is a lot of material available to the public on this issue.
    - Another participant commented that Manitoba Infrastructure and Transportation (MIT) was fairly accurate with their 2011 flood forecast. Participant had attended a meeting prior to the 2011 flood where MIT informed attendees of the expected flooding. Unfortunately, people in attendance chose not to listen.
- The desired water level at Dauphin Lake is 854 feet.
- Lake Manitoba experienced flooding three times in the 1900s.
- Manitoba Infrastructure and Transportation is committed to using information collected from these sessions and the public open houses to help identify solutions. Ultimately, elected officials will make the final decisions on what options to advance, how they will be funded and the timing.
- If buyouts of high flood risk properties are recommended they also need to consider the long-term economic impacts on the area (e.g. reduced tax base).
- Tiger Tube temporary dikes can move and float, making them less effective.

Action items:

Manitoba Infrastructure and Transportation to discuss approving a Dauphin Lake information questionnaire that can be distributed to stakeholders. Follow-up: declined by MIT, not in study scope.

Assiniboine River & Lake Manitoba Flood Mitigation Study

Community Leadership Meeting

Tuesday, June 18, 2013, 1-3 pm

Super 8 Hotel, Dauphin, MB

Meeting Objectives:

- To communicate the scope of the study;
- To provide the opportunity for participants to share thoughts on what options should be considered to reduce flood effects and how these options should be prioritized; and
- To receive feedback from participants.

Agenda Items:

1. Introductions
2. Storyboard Presentation
3. Questions and discussion

Assiniboine River & Lake Manitoba Basins Flood Mitigation Study: Round 1 Public Engagement

Title:	**Assiniboine River & Lake Manitoba Basins Flood Mitigation**		
Date:	June 19, 2013	**Minutes Status:**	FINAL
Attendees:	**Participants** Rick Saler, Councillor, Town of Minnedosa Ken Jenkins, CAO, Town of Minnedosa Wendy Davidson, Reeve, RM of Archie Olive McKean, Reeve, RM of Miniota Alvin Zimmer, Reeve, RM of Shellmouth Denis Carter, Reeve, RM of Woodworth Sandy Sanderson, Reeve, RM of Glenwood Sandra Couling, Councillor, RM of Whitehead Joe Goodwill, Deputy Reeve, RM of Glenwood Rick Plaisik, Reeve, RM of Sifton Stan Cochrane, Councillor, RM of Sifton Herb Mahood, Director Regional Operations, MIT **Study Team** Fishaha Unduche, Manitoba Infrastructure and Transportation Eugene Kozera, Manitoba Infrastructure and Transportation Steve Topping, Manitoba Infrastructure and Transportation Rob Kenyon , KGS Group Colin Siepman, KGS Group Patrice Leclercq, KGS Group John Osler, InterGroup Consultants Yadili Okwumabua-Gbakima, InterGroup Consultants		
Meeting Location:	Keystone Centre – Brandon, MB	**Meeting Time:**	1:00 PM – 3:15 PM
Meeting Purpose	The objective of the meeting was to share with local leadership current information regarding the study in advance of the public open house. This will allow community leadership to be informed of the study process should questions arise from their constituents. As well, perspectives provided by		

	participants would be integrated into the study process.
Introduction	John Osler welcomed participants to the information session, noting this was the first opportunity for people to be consulted on the study. He thanked people for attending and encouraged them to ask questions throughout the presentation. The Study Team was introduced to the group.
Presentation	Presented by John Osler (IG) and Colin Siepman (KGS) presented the storyboards outlining the flood mitigation study. The study will allow the province to gain a better understanding of the flood risks and the possible mitigation efforts that can be used to reduce effects of future flood events. Round One communicates the scope of the study, presents an opportunity for stakeholders to provide thoughts on what flood reduction options should be considered and how these efforts should be prioritized. This information will be considered by the study team in the development of the hydraulic model and the benefit/cost analysis.

SUMMARY OF QUESTIONS AND DISCUSSION

As the presentation was made, a number of questions were raised by those in attendance. Questions and comments are summarized below.

Q. How long and wide is the Lake St. Martin Outlet Channel?

A. 6 km long, base width of 45m with 3 to 1 side slopes.

Q. Is there a control at the Lake St. Martin Outlet Channel or does the water move freely?

A. There is no control structure. The only way to change flows is to install or remove a cofferdam at the channel entrance.

Q. What is the design flow of the Lake St. Martin Outlet Channel?

A. The channel was originally designed for a maximum flow of 5,000 cfs but that design was ultimately reduced by 75% due to time constraints.

Q. What is the difference in elevation between Lake Manitoba and Lake Winnipeg?

A. Approximately 100 feet.

Q. Can the Lake St. Martin Channel be closed?

A. Emergency operation status for the channel expired in November 2012. The channel opening is currently closed though the necessary equipment remains onsite. The Assiniboine River and Lake Manitoba Flood Mitigation Study will consider making the outlet channel permanent as a mitigation option.

Q. What are the future plans for the Lake St. Martin Outlet Channel?

A. The Province is currently looking at the entire system, including the Lake St. Martin Outlet Channel, and will consider making it permanent as a mitigation option.

C. A participant mentioned that the Province needs to remember that water from the 2011, 2012, and 2013 floods came from Saskatchewan, so throwing money into the Manitoba system will not solve the real problem.

C. It was noted that this issue needs to be approached from a watershed perspective.

C. An attendee suggested that the flows in the creeks that run into the Lake of the Prairies increased by 2,000%, and were mostly from Saskatchewan. This is the issue that must be addressed before land in the Assiniboine River Basin is completely washed away.

C. KGS commented that the political aspect of Manitoba's flood issue is beyond the scope of the Assiniboine River and Lake Manitoba River Basins Flood Mitigation Study. The study will focus on Manitoba (rather than other provinces); however, the potential for water retention in Saskatchewan will be reviewed at a cursory level based on past studies.

Q. Is there any effort within the government to coordinate all of the flood-related studies that are underway?

A. Yes, the 2011 flood reviews will be used as inputs into the current study.

Q. Where does the data used to estimate losses come from?

A. KGS is working with Emergency Measures Organization, Manitoba Agricultural Service Corporation and others to collect data. The data will also be cross-referenced directly with some RM's.

Q. Some people do not get compensated for their losses. How will this study account for unregistered losses?

A. This is an area where public information is especially helpful. It is possible to approach municipalities for damage data.

C. It was suggested that a standard information request form be sent to all municipalities so that the same data is collected.

KGS is working with MIT to draft and approve a form that is similar to what was suggested above.

Q. It was mentioned that all of the recent studies will feed into this study. I've heard that in the near future Manitoba will experience a water shortage. How will this be incorporated into the study?

A. Water supply would be a benefit to some of these projects; however, it is extremely complicated to quantify and beyond the scope of the current study.

Q. Is it conceivable to suggest that we can afford to build another dam similar to the Shellmouth Dam? Is it too expensive? A. The study will investigate all components of this option including land acquisition, environmental impact, construction costs, etc.
Q. Studies were conducted for potential dams along the Assiniboine River. Where were the proposed locations of these dams? A. This study will review and update past studies and also consider new dam sites. It was noted that as the economy and market values change, benefit and costs estimates have to be revised to reflect a different environment.
Q. Why is there more silt sitting in our fields now as compared to 2011. A. This may be due to the smaller flood not moving fast enough to clear the silt out.
Q. Are you saying that the river will clear out all of those trees? A. No.

Additional comments:

- An attendee commented that larger floods heavily impact stakeholders as the negative effects are felt for weeks after the water recedes.
- A KGS representative expressed that Western provinces cannot fully be blamed for Manitoba's flooding. He shared that he recently was in Saskatchewan helping put up dikes. This year, Regina and Saskatchewan endured a very wet spring, yet Manitoba did not have to fight floods this year.
- An attendee questioned the overall flood protection process because not everyone makes the current regulations work. When the water comes, the RM takes care of it but they also need advice on what to do when surface water is the cause of flooding.
- A participant mentioned that draining was the cause of 2012 and 2013 flooding and added that current water levels suggest a 2014 flood.
- It was suggested that farmers could protect themselves from the type of flooding that occurred in 2012 and 2013 if the frequency of these types of floods were known.
- KGS noted that there will be situations where temporary flood mitigation measures will still be necessary. It is not practical to mitigate every scenario with permanent structures.
- A participant suggested looking at the province by quarters when considering mitigation options so that all Manitobans are covered (regional balance).
- Another participant expressed concern that the southwest corner of the province will not be considered in the study.
- A participant suggested that Manitoba has 20% of North America's fresh water and that Manitobans should to stop blaming Saskatchewan and figure out how to benefit from the excess water. It is difficult to get a permit to pump water out of the Assiniboine River. It seems as if government departments are working against each other.
- When considering the value of water, the Province should sue for increased membership/representation in the international board that is comprised of American, Saskatchewan, Alberta, and Manitoba representatives. Currently, Manitoba has only one member.
- A participant commented that the Province has many studies but is slow at starting projects.
- It was mentioned that residents within the Assiniboine Valley are still waiting for compensation and are not even sure what type of compensation will be received. In fact, forms have not been received. It would be appreciated if the Province could share this information if it comes out during this study process.
- A participant expressed that the problem with compensation is that there should not be a difference between losing a residence or losing cropland.
- Concern was expressed regarding the large number of trees lost along the river and it will soon cause problems. The debris needs to be cleared out.
- A participant mentioned that silt deposits should be factored into this study. Silt removal has saved land along the river.
- A participant suggested that the regulatory process for the construction of a dam should be reviewed. This process should be simplified so that it doesn't take so long to implement and start construction.

Assiniboine River & Lake Manitoba Flood Mitigation Study

Community Leadership Meeting

Wednesday, June 19, 2013, 1-3 pm

Keystone Centre, Brandon, MB

Meeting Objectives:

- To provide communicate the scope of the study;
- To provide the opportunity for participants to provide thoughts on what options should be considered to reduce fold effects and how these options should be prioritized; and
- To receive feedback from participants.

Agenda Items:

1. Introductions
2. Storyboard Presentation
3. Questions and discussion

Assiniboine River & Lake Manitoba Basins Flood Mitigation Study: Round 1 Public Engagement

Title:	**Assiniboine River & Lake Manitoba Basins Flood Mitigation**		
Date:	June 20, 2013	**Minutes Status:**	FINAL
Attendees:	**Participants:** Earl J. Porter, Mayor, City of Portage la Prairie Kam Blight, Reeve, RM of Portage la Prairie Wally Melnyk, CAO, Town of Russell Doug Pickell, Councillor, RM of Rockwood David Single, Reeve, RM of Westbourne Daryl Hrehirchuk, CAO, RM of Portage la Prairie Harold Purkess, Reeve, RM of Victoria Ivan Bruneau, CAO, RM of Victoria Don Walsh, Reeve, RM of Woodlands Eilleen Lynxleg, Tootinaowaziibeeng Treaty Reserve Robert Thiessen, Councillor, RM of St. François Xavier Roland Rasmussen, Reeve, RM of Cartier Randi Thomas, Manitoba Metis Federation **Study Team:** Fishaha Unduche, Manitoba Infrastructure and Transportation Steve Topping, Manitoba Infrastructure and Transportation Colin Siepman, KGS Group Dave MacMillan, KGS Group John Osler, InterGroup Consultants Yadili Okwumabua-Gbakima, InterGroup Consultants		
Meeting Location:	Canad Inns Room H – Portage la Prairie, MB	**Meeting Time:**	1:00 PM – 3:00 PM
Meeting Purpose	The objective of the meeting was to share with local leadership current information regarding the study in advance of the public open house. This will allow community leadership to be informed of the study process should questions arise from their constituents. As well, perspectives provided by participants would be integrated into the study process.		
Introduction	John Osler welcomed participants to the information session, noting this was the first opportunity for people to be consulted on the study. He thanked people for attending and encouraged them to ask questions		

	throughout the presentation. The Study Team was introduced to the group.
Presentation	John Osler (IG) and Colin Siepman (KGS) presented the storyboards outlining the flood mitigation study. The study will allow the province to gain a better understanding of the flood risks and the possible mitigative efforts that can be used to reduce effects of future flood events. Round One communicates the scope of the study, presents an opportunity for stakeholders to provide thoughts proposed mitigative efforts to reduce flood effects and how these options should be prioritized. This information will be considered by the study team in the development of the hydraulic model and the benefit/cost analysis.

SUMMARY OF QUESTIONS AND DISCUSSION

As the presentation was made, a number of questions were raised by those in attendance. Questions and comments are summarized below.

Q. Will you be releasing a copy of the water management study? When will it be available?

A. Yes, will be posted on Provincial website in 2013.

Q. Are you looking at tributaries that come from Duck Mountain? There is one dam near Duck mountain and one river that flows through our community. Last time the dam was flooded out, contaminating community water supply with mercury. These actions are detrimental to human and the environmental health.

A. KGS has a list of tributaries that will be examined.

Q. Are you collecting data on your own or will you be collaborating with RMs, towns, elected officials, and community members for data.

A. Primarily from provincial sources but also some collaboration to cross-reference the data.

Q. Will the results of the study be distributed prior to Round 2 consultations? Other studies provide general information but do not allow much room for public input.

A. The intent of Round Two is to share the preliminary results of the study with the public prior to completing the report so that the study can benefit from these additional perspectives.

Q. Is there an executive summary available for all of these studies? It would be helpful to know about all of the studies summarized together.

A. An executive summary was included with the 2011 Flood Review Task Force Report. Other studies, such as the Lake Manitoba and Lake St. Martin Regulation Review, do not include an executive summary. The 2011 Flood Review Task Force Report also includes a description of the other studies currently contemplated or underway. Both reports are on the province's website.

Q. Why is the Province repeating what was done for the Lake Manitoba and Lake St. Martin studies?

A. The previous studies were high level reviews. The current study will evaluate potential mitigation options in greater detail.

Q. Concerned that no Aboriginal communities have been formally consulted. When does the Province intend on doing this considering it is a legal obligation.

A. The current flood mitigation study is focused on identifying and considering flood mitigation options. No decisions are being made at this time on specific options. When decisions are being made about specific mitigation options, it is expected that another process will be implemented to determine the nature and form of consultations with Aboriginal communities.

C. It would not necessarily mean that Manitoba would have to consult on each potential mitigation option, but this study will ultimately make recommendations on specific options and implementation timetables.

C. The duty to consult is an international law held by the Crown and needs to be recognized.

Q. Two years ago, a community diking program was announced but we have yet to hear anything. Has any of this been done?

A. There were 72 applicants which was short-listed to 13. Now eight feasibility studies are nearing completion.

Q. Are you looking at creating jobs in affected areas?

A. The study will try to factor in the regional and local benefits associated with the various flood mitigation options. This will be reflected in the benefit/cost ratio comparison.

Q. What percentage of the project has been set aside for local people?

A. This study is focused on what should be done rather than how best it can be done.

C. The way I see it, there are no social benefits from this project. The Province should look at these areas and identify employment opportunities.

Q. Will the study look at the Shoal Lakes Outlet?

A. Yes. Shoal Lakes Outlet has been identified and will be reviewed in this study.

Q. Is there a program to stabilize current dikes and can this be considered on the riverside?

A. The feasibility of stabilizing current dikes is under review.

Q. Will you look at impacts on the north? If water is being drained to the north, then there needs to be some sort of review.

A. The impacts are anticipated to be minor but they will be considered.

Q. Has the Province spoken to First Nations along Lake Winnipeg who will be affected when water is diverted?

A. No formal discussions at this time.

Additional comments:

- Hay and crop land was damaged in 2011 by failsafe overflow.
- The University of Manitoba research station has been abandoned, moving some buildings east of the Portage Diversion.
- There are roughly a half dozen lodges & homes in the failsafe area; temporary protection in 2011 was successful.
- The 2012 failsafe inundation due to ice jam was of short duration; mostly confined to marsh and not farmland.
- A participant requested that the Province ensure that regardless of staff turnover this study and any associated projects will move forward.
- If you are going to look at benefits and costs they should be considered for all Manitobans and not just focused in one area. A youth employment strategy should be considered.
- Along with Whitemouth River, we have the Conservation District that helps with limited flood mitigation. It feels as though there are not enough votes to do anything. Not sure how this will be addressed.
- With regard to RM Whitemouth, if we get as much water as we did in 2009 and 2011, everyone will flood regardless of drainage.

Assiniboine River & Lake Manitoba Flood Mitigation Study

Community Leadership Meeting

Tuesday, June 20, 2013, 1-3 pm

Canad Inns, Portage la Prairie, MB

Meeting Objectives:

- To provide communicate the scope of the study;
- To provide the opportunity for participants to provide thoughts on what options should be considered to reduce fold effects and how these options should be prioritized; and
- To receive feedback from participants.

Agenda Items:

1. Introductions
2. Storyboard Presentation
3. Questions and discussion

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APPENDIX Q 2
LAKE MANITOBA AND LAKE ST. MARTIN OUTLET CHANNEL CONCEPUTAL DESIGN PRELIMINARY OPTIONS ASHERN OPEN HOUSE SUMMARY REPORT NOVEMBER 2014

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LAKE MANITOBA AND LAKE ST. MARTIN

OUTLET CHANNEL CONCEPTUAL DESIGN

PRELIMINARY OPTIONS

ASHERN OPEN HOUSE SUMMARY REPORT

Prepared by:

InterGroup Consultants Ltd.

500-280 Smith Street

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Submitted to:

KGS Group

865 Waverly Street, 3rd Floor

Winnipeg, MB R3T 5P4

November 2014

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- MEDIA MATERIALS
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- ASHERN OPEN HOUSE INVITATION LETTER AND EMAIL TEXT
- ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY AREA MAP
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APPENDIX B ASHERN OPEN HOUSE MATERIALS

- STORYBOARDS
- COMMENT FORM
- DRAFT PROJECT SCHEDULE FOR LAKE MANITOBA AND LAKE ST. MARTIN CHANNELS

APPENDIX C ASHERN OPEN HOUSE SIGN IN SHEETS AND COMMENTS RECEIVED

- SIGN-IN SHEETS
- COMMENT SHEETS AND SUBMISSIONS (CONFIDENTIAL)

1.0 BACKGROUND

Manitoba Infrastructure and Transportation (MIT) engaged KGS Group to develop a two-stage process to advance the Lake Manitoba and Lake St. Martin Outlet Channels Conceptual Design Study. Stage 1 of this process included the following scope of work:

- Identify Outlet Options for Lake Manitoba and Lake St. Martin;
- Develop Screening Level Designs and Cost Estimates;
- Economic Assessment;
- Stage/Damage Relationships;
- Summary Report; and
- Public Consultation.

This report documents the process, information shared and feedback received during the consultation process. It is expected that information will help inform future stages of the design and project development process.

2.0 OPEN HOUSE SUMMARY

The open house was held on September 18, 2014, at the Centennial Hall in Ashern, Manitoba from 5:00 pm to 8:00 pm. Material shared at the event provided information about Manitoba's flood protection model, including the current Assiniboine River and Lake Manitoba Basins Flood Mitigation Study, and how the Lake Manitoba and Lake St. Martin outlet channels will form part of a larger system for the Assiniboine River and Lake Manitoba basins flood mitigation. Input from participants was solicited on six options for the Lake Manitoba outlet channel and two options for the Lake St. Martin outlet channel. The information presented provided conclusions and recommendations on two options for people to consider for the Lake Manitoba outlet channel (options C and D) and a preferred option for Reach 3 of the Lake St. Martin outlet channel (Willow Point). Stakeholders were provided with an opportunity to review the storyboards (attached in Appendix B) and to speak with senior staff from Manitoba Infrastructure and Transportation (MIT) and design engineers from KGS Group to learn about the options, ask questions and provide their comments and perspectives. At the sign in table those who attended were given a copy of the Comment Form and a Draft Project Schedule (copies provided in Appendix B).

Study team members were present to guide attendees through the storyboards, provide answers to questions, and receive input on the presented options. Participants were encouraged to fill out comment forms. Total attendance for the Ashern Open House was approximately 250 people.

2.1 EVENT COMMUNICATION

Community leadership, First Nations and Aboriginal communities, and interested individuals and organizations within the study region were identified and mailed an invitation to the Open House event (copies provided in Appendix A). An email with the invitation was sent to stakeholders that had provided an email address or for which an email address could be found.

Advertisements were placed in various newspapers to notify the public of the upcoming open house (copies are provided in Appendix A).

On the day of the event the Open House Storyboards and the Comment Form were posted on the MIT website where previous flood study documents have been posted (http://www.gov.mb.ca/mit/floodinfo/floodproofing/reports/index.html) (copies are provided in Appendix B).

2.2 ASHERN OPEN HOUSE MATERIALS

Materials prepared for the Ashern Open House included the following:

- **Storyboards:** Storyboards were used to communicate key messages of the conceptual design. These included: information on why the study is being conducted; an overview of the Manitoba flood protection model; description of preliminary options as well as an overview of the benefits to Lake Manitoba and Lake St. Martin water levels; summary information on estimated costs and

potential environment concerns; preliminary conclusions; and the anticipated next steps of the project.

- **Comment Forms:** Open house attendees were asked to share their concerns/issues about the conceptual design preliminary options and the public involvement process. For some discussions with open house participants, a study team member documented information provided by participants. Comment Forms were distributed during the event and some attendees took additional copies for those who were unable to attend the event. Participants were given the option to fill out a form at the event, mail the form in, or email comments to the project email address – feedback@floodstudy.ca.

- **Draft Project Schedule:** A one page handout that lists the steps that the province of Manitoba needs to follow to plan and eventually build and operate new outlet channels for Lake Manitoba and Lake St. Martin.

Copies of the Storyboards, Comment Forms and Draft Project Schedule used at the Ashern Open House are provided in Appendix B.

2.3 SUMMARY OF ROUND 1 PUBLIC CONSULTATION INTERESTS AND CONCERNS

Stakeholders came to the Ashern Open House to share their personal perspectives and to discuss the different options presented with the Open House Team. Stakeholders shared different perspectives on the design options and several common themes emerged throughout the discussions. Despite frustration with the current flooding situation for many regional participants, the majority of people noted appreciation of the effort that had been made to share the information and receive information from stakeholders. The presence and participation of Minister Steve Ashton and senior staff from MIT similarly enhanced the credibility of the process and the discussions that took place.

In total there were 231 people on the sign in sheets and several people were noted to have walked in without signing in, for an estimate of 250 people in attendance. A total of 50 Comment Forms were submitted by attendees at the event, an additional nine were received by mail and six were emailed after the event. The following is a summary of the information received from the Comment Forms.

- Personal connection to the process included:
    - 28 homeowners;
    - 16 farmers/ranchers;
    - 13 cottage owners;
    - 8 elected municipal officials;
    - 4 business owners;
    - 3 First Nation members;

  - 2 municipal employees; and
  - 5 other.

- 30 people participated in the Individual Flood Protection Initiative.
- 21 people participated in the Financial Assistance to Cottage Owners.
- How people heard about the event:
  - 23 by newspaper;
  - 22 by word of mouth;
  - 18 by email;
  - 15 by radio/public service announcement;
  - 1 by letter of invitation; and
  - 7 by other (town sign and Facebook).
- 30 people indicated that the information presented in the boards and/or their discussion with Open House staff was helpful.
- The Manitoba Beef Producers provided a formal submission addressing the Open House materials and their broader concerns.

Copies of both the submitted comment forms and received submissions were provided confidentially to MIT to use as a reference in future planning but have not been appended to the public report. Information shared in the Comment Form, emails received after the event and the notes taken by the Open House staff are summarized in Table 2-1. This summary is organized around the open ended questions on the Comment Form and other common themes heard at the Ashern Open House.

Table 2-1: Summary of Ashern Open House Comments

Question / Theme	**Comments Received** (most common comment are first for each set)
How could the Open House event have been more helpful to you?	Give an overview of the materials at the beginning of the Open House.
	Provide an open microphone or have a question and answer session.
	Provide the information to people in advance of the Open House so that people can come prepared for the discussions (link to material in advertising or invitation).
	Describe the format of the event in the advertising.
	The event was crowded, hard to get at the storyboards.
	Identify the Open House experts better.
	Have more printouts of the material available.
	Respect the elderly and have more chairs for them to sit down (and do a presentation).
	Show fewer options – One person noted that media had suggested that only Options C and D were being pursued, so more options created confusion.
	Have more government officials, speakers, people on hand that know about Lake Manitoba, people with more answers.
	Need to acknowledge that people have been living in a state of duress.
	Publicize the event better and more widely.
Why did you decide to attend the Open House today?	To see the scope of the project, to get more information and some answers.
	Came hoping that a plan would be announced.
	To get some answers about the current flood management.
	To become more informed to help local residents.
	Individuals land is flooded and wanted to know what is being planned.
	To give their opinion or share some information about the routes.
	Concerned about delays.

Question / Theme	**Comments Received** (most common comment are first for each set)
	Live in the area or have a cottage in the area.
	To learn when issues will be addressed to help deal with the anxiety local people are feeling.
	The Manitoba Beef Producers attended to learn about the proposed designs and to offer feedback to help draw down Lake Manitoba and Lake St. Martin.
What perspectives would you like to share about the options that have been shared today? Do you have a preference and why?	18 Comment Forms listed Option D – it is on private and crown land, less political, most natural option, less intrusive and more direct.
	4 Comment Forms listed Option C – so you can start sooner and deal with private land owners.
	Noted it is easier to deal with private land owners.
	Proposed ditch does not have enough capacity, need to be much bigger channel, needs to match the Portage Diversion.
	Build it to handle 10,000 cfs, 5,000 to 7,500 is not enough.
	Go with the best options from an engineering perspective, least amount of bedrock.
	Start with mitigation measures that would reduce reliance on the Portage Diversion.
	Close the Portage Diversion, Assiniboine River too polluted.
	The Manitoba Beef Producers will not support one single option for all of their members, but notes that cattle operations will continue to be flooded under all of the proposed options.
	Why is it taking so long, get it done.
What other comments or questions would you like to provide today?	Why is it taking so long, get it done.
	Remove as much water as the diversion puts in.
	Would like to see the Lake St. Martin control structure opened fully immediately to get the lake down by 3 feet.
	Only use the Portage Diversion as originally intended.
	Portage Diversion should not be opened beyond 25,000 cfs.
	Do it right and do it soon.

Question / Theme	**Comments Received** (most common comment are first for each set)
What other comments or questions would you like to provide today? (continued)	Give it the attention it deserves.
	You are putting the citizens around the lake through Hell.
	A flood in 2015-2016 could cost much more than a solution now.
	If there is more flooding, most ranches will be out of business.
	Why isn't the province doing more for people around Lake Manitoba?
	We had more questions and were given better answers than in 2011.
	Treat those affected fairly.
	Appreciate all the work put into flood protection.
	Keep us informed.
	Thank you for letting us express our opinion.
	More needs to be done in Saskatchewan and down the Assiniboine River, need to work with the entire system to balance flows.
	Put a series of culverts on the approaches to the Fairford Water Control Structure and lower the Fairford bridge, low cost, maximum effect.
	Remove the same amount of water that the diversion puts in.
	Do more to restore the sloughs and potholes upstream.
	Need to increase the capacity of the Lower Assiniboine River to 24,000 cfs.
	Need to lower Lake Manitoba to 810 feet so that it has the capacity to accept flood waters.
	Divert some flows into the La Salle River and then into the Red River. The Red River peaks earlier and can handle the later flows from the Assiniboine River.
What other comments or	The Manitoba Beef Producers agree that there needs to be a review of the entire province's water management system along with plans for a long term strategy, but it would like this to happen much more quickly, not in seven years.

Question / Theme	**Comments Received** (most common comment are first for each set)
questions would you like to provide today? (continued)	The Manitoba Beef Producers question if the Lake Manitoba and Lake St. Martin Regulation Review Committee recommendations of lowering the operating range of Lake Manitoba will be followed. This would reduce the operating range from 810.5 feet to 812.5 feet by half a foot to 810.0 feet to 812.0 feet for a period of five years.
	The Manitoba Beef Producers would like clarification about why flows of 5,000 cfs and 7,500 cfs for the channel options to lower Lake Manitoba were used and if larger flows were evaluated and taken off the table.
	The Manitoba Beef Producers would like the Province to understand that loss of forage crops due to flooding is more than a one year event, it can take two to three years for forage crops to recover. Their view is that compensation should consider this longer period of impact.
	The Manitoba Beef Producers recognize that some of their members with operations around Lake Manitoba were flooded to help protect other Manitobans. They feel that they deserve fair treatment for their sacrifices, and lasting solutions to this water management problem.
	Quite a few people felt that they were being artificially flooded by the inflows down the Portage Diversion and did not seem to recognize or account for the benefits of the enlarged Fairford structure.
	Several people expressed strong concerns about the water quality impacts of the Portage Diversion. Some people noted that the water used to be clear and now it is very murky.
	Quite a few people said that the government had done very little to address the impacts on flooded agricultural land around Lake Manitoba.
Other recorded comments	Several people said they did not accept that it would take so many years to build the Lake Manitoba Outlet. They contrasted this with the government's decision to use the Portage Diversion in the spring. People also questioned the level of flows allowed through the Portage Diversion.
	Several individuals took exception to the plot showing the regulated and unregulated water levels.
	Several individuals had concerns about water quality and the phosphorus loading of Lake Manitoba attributed to inflows from the Portage Diversion.

Question / Theme	**Comments Received** (most common comment are first for each set)
Other recorded comments (continued)	Some individuals inquired why the inlet to the Fairford Control Structure had to be dredged in the last decade. In their opinion the lake could have been brought down closer to 810 feet if this dredging had occurred.
	Many people asked if dredging the Assiniboine River was being looked at. This was thought to be a better solution than just expanding the dikes.
	A few individuals claimed that the statement "The design of this structure [FRWCS] considered the total inflow to Lake Manitoba including the Portage Diversion, which was completed later in 1970" was an outright lie.
	A majority of people from the agricultural industry strongly suggested that the 7,500 cfs limit be increased. They compared this to the 34,000 cfs input from the Portage Diversion. They start to lose their crops [mostly hay – although some had grain crops] around 812.5 to 813 feet.
	Farmers expressed concern that damage calculations are not being properly considered. An individual noted that it took three years to recover after the 2011 flood and compensation for one year's damages was not the right way to consider damages.
	An individual noted that he had talked with the federal minister and heard that the federal government had committed to financially supporting a portion of the project. The individual also noted that the environmental approval process was changed and felt the project could be expedited.
	Landowners that were directly affected by an alternative alignment wanted input into the fine tuning of the alignment as it affected their farming operations. Landowners indicated alignment C and D would either sever their farmyard from their land or was passing through their farmyard.
	General opinion was alternatives should be chosen to minimize FN impacts.
	Counsellor from PFN indicated they were not consulted on any of the alignments.
	Route C and B could influence ground water and public safety on reserve.
	Public knowledge about projects is inadequate. Owners along preferred options should have been notified earlier.
	Drainage along Route D is an issue right now.

Question / Theme	**Comments Received** (most common comment are first for each set)
Other recorded comments (continued)	7,500 cfs won't help, need outflow to equal inflow.
	Cottage raised to 822 feet, but access is at 817 feet, still have to evacuate.
	Several comments were provided about keeping water level on Lake Manitoba below 812 feet to prevent loss of hay land.

APPENDIX A

LAKE MANITOBA & LAKE ST. MARTIN OUTLET CHANNEL DESIGN OPTIONS OPEN HOUSE SUMMARY REPORT

OPEN HOUSE PLANNING MATERIALS

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PUBLIC OPEN HOUSE

Share Your Views

on the Lake Manitoba and Lake St. Martin Outlet Channels Conceptual Design Study

The Manitoba government invites you to attend an open house to learn more about the Lake Manitoba and Lake St. Martin Outlet Channels Conceptual Design Study.

This event will present the preliminary findings and results of the study, and provide you with an opportunity to share your views on the presented preliminary outlet channel options. The open house will be held at:

Location
Centennial Hall
Ashern, MB
September 18, 2014
5:00 p.m. to 8:00 p.m.

For more information about the Lake Manitoba and Lake St. Martin Outlet Channels Conceptual Design Study, please email: **feedback@floodstudy.ca** or call **204-896-1209.**

Manitoba

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Invitation/Contact list for the Lake Manitoba and Lake St. Martin Outlet Channels Design Open House in Ashern Manitoba on September 18, 2014

Title and Name	Organization
CAO Susan Boyachek	RM of Gilbert Plains
Chief Adrian Sinclair	Lake St. Martin First Nation
Chief Albina Brass	Wuskwi Sipihk First Nation (Indian Birch)
Chief Barry McKay	Tootinaowaziibeeng Treaty Reserve
Chief Cameron Catcheway	Skownan First Nation
Chief Charlie Boucher	Pine Creek Anishinabe Nation
Chief Clarence Easter	Chemawawin Cree Nation
Chief Cornell McLean	Lake Manitoba First Nation
Chief David Ledoux	Gamblers First Nation
Chief David Meeches	Long Plain First Nation
Chief David Pashe	Dakota Tipi First Nation
Chief David Crate	Fisher River First Nation
Chief Delbert Cruise	Canupawakpa Dakota First Nation
Chief Eugene Eastman	O-Chi-Chak-Ko-Sipi First Nation
Chief Francine Meeches	Swan Lake First Nation
Chief Garnet Woodhouse	Pinaymootang First Nation
Chief Glenn Husdon	Peguis First Nation
Chief Hector Shorting	Little Saskatchewan First Nation
Chief James Plewak	Keeseekoowenin Ojibway Nation
Chief John Stagg	Dauphin River First Nation
Chief Mel Wabash	Waywayseecappo First Nation
Chief Morris Swan-Shannacappo	Rolling River Anishinabe Nation
Chief Nelson Bunn	Birdtail Sioux First Nation
Chief Nelson Genaille	Sapotaweyak Cree Nation

Chief Nelson Houle	Ebb & Flow First Nation
Chief Orville Smoke	Dakota Plains First Nation
Chief Russell Beaulieu	Sandy Bay First Nation
Chief Vincent Tacon	Sioux Valley Dakota Nation
Councillor Alfred Hayden	Roseau River Anishinabe First Nation
General Manager Maureen Cousins	Manitoba Beef Producers
Mayor Betty Nykyforak	Town of Roblin
Mayor Bob Walker	Town of Melita
Mayor Brian Franklin	Town of Deloraine
Mayor Bruce Gullet	Village of Wawanesa
Mayor Bruce Henley	RM of West St. Paul
Mayor Christopher Radford	Town of Russell
Mayor Dale Sawchuk	Village of Binscarth
Mayor Dan Brown	Village of Winnipegosis
Mayor Darryl Jackson	Town of Souris
Mayor Donald Yanick	Town of Shoal Lake
Mayor Dwight Stewart	Town of Birtle
Mayor Earl Porter	City of Portage la Prairie
Mayor Edward Anderson	Town of Boissevain
Mayor Eileen Clarke	Town of Gladstone
Mayor Einarsson Helgi	Dauphin River (Community of)
Mayor Eric Irwin	City of Dauphin
Mayor Gary Williams	Village of Waskada
Mayor Gilbert Mabon	Village of Somerset
Mayor Glen McKenzie	Town of Swan River
Mayor Henry Barkowski	Town of Minitonas
Mayor Jeff McConnell	Town of Virden
Mayor Jeff Sigurdson	Town of Oak Lake

Mayor Ken Waddell	Town of Neepawa
Mayor Larry Oakden	Village of Hamiota
Mayor Leanne Hutman	Village of Bowsman
Mayor Linda Cripps	Village of McCreary
Mayor Lori Taylor	Town of Hartney
Mayor Lyle Smith	Village of Gilbert Plains
Mayor Marion Meadows	Village of Benito
Mayor Martin Dupont	Village of St-Lazare
Mayor Mitch Michaluk	Village of Ethelbert
Mayor Orest Woloski	Town of Rapid City
Mayor Ray Orr	Town of Minnedosa
Mayor Rene Maillard	Village of Ste. Rose du Lac
Mayor Robert Jewsbury	Village of Glenboro
Mayor Roland Gagnon	Village of Elkhorn
Mayor Sam Katz	City of Winnipeg
Mayor Shari Hirst	City of Brandon
Mayor Shirley Kalyniuk	Town of Rossburn
Mayor Todd Gill	Town of Rivers
Mayor Tom Bohun	Town of Grandview
Mayor Val Soltys	Town of Erickson
Mayor Wayne Blair	Town of Carberry
Mayor William Wiebe	Village of MacGregor
President David Chartrand	Manitoba Metis Federation
Reeve Alvin Zimmer	RM of Shellmouth-Boulton
Reeve Arne Lindell	RM of Eriksdale
Reeve Art Potoroka	RM of Ethelbert
Reeve Aurel Pantel	RM of Lorne
Reeve Barry Zacharias	RM of Siglunes

Reeve Blair Woods	RM of Whitewater
Reeve Bob Conibear	RM of Argyle
Reeve Brent Fortune	RM of Blanshard
Reeve Brian Sigfusson	RM of Coldwell
Reeve Calvin Jacobson	RM of Minto
Reeve Cliff Kutzan	RM of Grandview
Reeve Clinton Cleave	RM of Ochre River
Reeve Craig Atkinson	RM of Park
Reeve Craig Spencer	RM of South Norfolk
Reeve David Inkster	RM of Oakland
Reeve David Single	RM of Westbourne
Reeve Denis Carter	RM of Woodworth
Reeve Dennis Forbes	RM of Dauphin
Reeve Dennis Schram	RM of Strathcona
Reeve Derek Klassen	RM of Glenella
Reeve Diane Price	RM of Grahamdale
Reeve Don Neufeld	RM of Wallace
Reeve Donald Walsh	RM of Woodlands
Reeve Duncan Stewart	RM of Brenda
Reeve Earl Malyon	RM of South Cypress
Reeve Earl Zotter	RM of St. Laurent
Reeve Edward Levandoski	RM of Rosedale
Reeve Edward Mychasiw	RM of Rossburn
Reeve Evan Smith	RM of Daly
Reeve Frances Smee	RM of Rosser
Reeve Fred Dunn	RM of Silver Creek
Reeve Fred Taylor	RM of Lawrence
Reeve Garry Wasylowski	RM of Armstrong

Reeve Guy Huberdeau	RM of Ellice
Reeve Harold Purkess	RM of Victoria
Reeve James Andersen	RM of Odanah
Reeve James Boyd	RM of Elton
Reeve James Trewin	RM of Arthur
Reeve Jim Campbell	RM of Rockwood
Reeve Kam Blight	RM of Portage la Prairie
Reeve Kathy Jasienczyk	RM of Langford
Reeve Ken Wozney	RM of Strathclair
Reeve Kim Taylor	RM of Pembina
Reeve Larry McLauchlan	RM of McCreary
Reeve Larry Zurba	RM of Mossey River
Reeve Lonn Dunlop	RM of Riverside
Reeve Lorne Henkelman	RM of Swan River
Reeve M.E. Sanderson	RM of Glenwood
Reeve Maurice Maguet	RM of Ste. Rose
Reeve Michael Dillabough	RM of Winchester
Reeve Michael McIntosh	RM of Minitonas
Reeve Murray Davies	RM of Harrison
Reeve Neil Christoffersen	RM of North Norfolk
Reeve Olive McKean	RM of Miniota
Reeve Philip Thordarson	RM of Lakeview
Reeve Ralph Oliver	RM of North Cypress
Reeve Ralph Wang	RM of Edward
Reeve Randy Lints	RM of Hamiota
Reeve Ray Franzmann	RM of Grey
Reeve Reg Atkinson	RM of Conwallis
Reeve Richard Funk	RM of Lansdowne

Reeve Richard Hyde	RM of Fisher
Reeve Rick Plaisier	RM of Sifton
Reeve Robert Hanson	RM of Mountain
Reeve Robert McCallum	RM of Morton
Reeve Robert Misko	RM of Hillsburg
Reeve Robert Muir	RM of Russell
Reeve Robert Sharpe	RM of Saskatchewan
Reeve Roger Poitras	RM of St. Francois Xavier
Reeve Roger Wilson	RM of Birtle
Reeve Roland Rasmussen	RM of Cartier
Reeve Ross Tycoles	RM of Pipestone
Reeve Stan Asham	RM of Alonsa
Reeve Tom Campbell	RM of Albert
Reeve Victor Baraniuk	RM of Clanwilliam
Reeve Wayne Dobbie	RM of Whitehead
Reeve Wayne Drummond	RM of Cameron
Reeve Wendy Davidson	RM of Archie
Reeve Wilfred Taillieu	RM of Headingly
Bill Finney	Westlake Grazing Club
Dave Wolfe	
Fred Hartmann	
Joe Johnson	Lake Manitoba Flood Rehabilitation Committee
Larry Gereles	Manitoba Beef Producers District 7
Pat Dunlop	Manitoba Association of Cottage Owners
Sadley Marcinyk	
Scott Greenlay	Association of Lake Manitoba Stakeholders
Sgnd. James A. Bauer	
Tom Teichroeb	Lake Manitoba Flood Rehabilitation Committee

Manitoba
Infrastructure and Transportation

Steven D. Topping, P. Eng.
Executive Director
Hydrologic Forecasting and Water Management
Water Management and Structures Division
Box 11, 200 Saulteaux Crescent
Winnipeg, Manitoba, R3J 3W3
T 204-945-7488 **F** 204-945-7419

August 29, 2014

To Whom It May Concern,

RE: LAKE MANITOBA and LAKE ST. MARTIN OUTLET CHANNELS CONCEPTUAL DESIGN STUDY OPEN HOUSE RESCHEDULED

Earlier this summer, Manitoba Infrastructure and Transportation (MIT) needed to postpone the planned open house to share information regarding options for enhanced channel outlet capacity. Staff resources were required to respond to high water events on Lake Manitoba and Lake St. Martin and the re-opening of the emergency channel.

The open house has now been rescheduled for September 18, 2014 at the Centennial Hall in Ashern. This event will present the preliminary channel options being studied and offer opportunities for people to provide their views on all aspects of the study.

As a stakeholder with possible interest in the project, we would like to personally invite you to either the open house or to respond to material that will be posted on the MIT website after the open house (http://www.gov.mb.ca/mit/floodinfo/floodproofing/reports/index.html).

The public open house will be held at: **Centennial Hall, Ashern, Manitoba**
September 18, 2014
5:00 p.m. – 8:00 p.m.

I have included with this letter a map of the study area for your information, along with an invitation to the open house event that can be posted in your community.

If you have any questions or comments about the study or information session, please feel free to contact me at 204-945-7488 or email at steve.topping@gov.mb.ca.

Yours truly,

<original signed by>

Steve D. Topping, P. Eng.
Executive Director
Hydrologic Forecasting and Water Management

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From: Lake Manitoba & Lake St. Martin Outlet Channel Study
Sent: Friday, September 05, 2014 4:23 PM
To: Lake Manitoba & Lake St. Martin Outlet Channel Study
Subject: Invitation to Open House - Lake Manitoba and Lake St. Martin Outlet Channel Conceptual Design
Attachments: L_MB_L_SM_poster_map 2014 05 09.pdf

RE: LAKE MANITOBA and LAKE ST. MARTIN OUTLET CHANNELS CONCEPTUAL DESIGN STUDY OPEN HOUSE RESCHEDULED

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The public open house will be held at: **Centennial Hall, Ashern, Manitoba**
September 18, 2014

5:00 p.m. – 8:00 p.m.

Included with this email is a map of the study area for your information, along with an invitation to the open house event that can be distributed to other persons who may have an interest in participating.

If you have any questions or comments about the study or information session, please contact feedback@floodstudy.ca.

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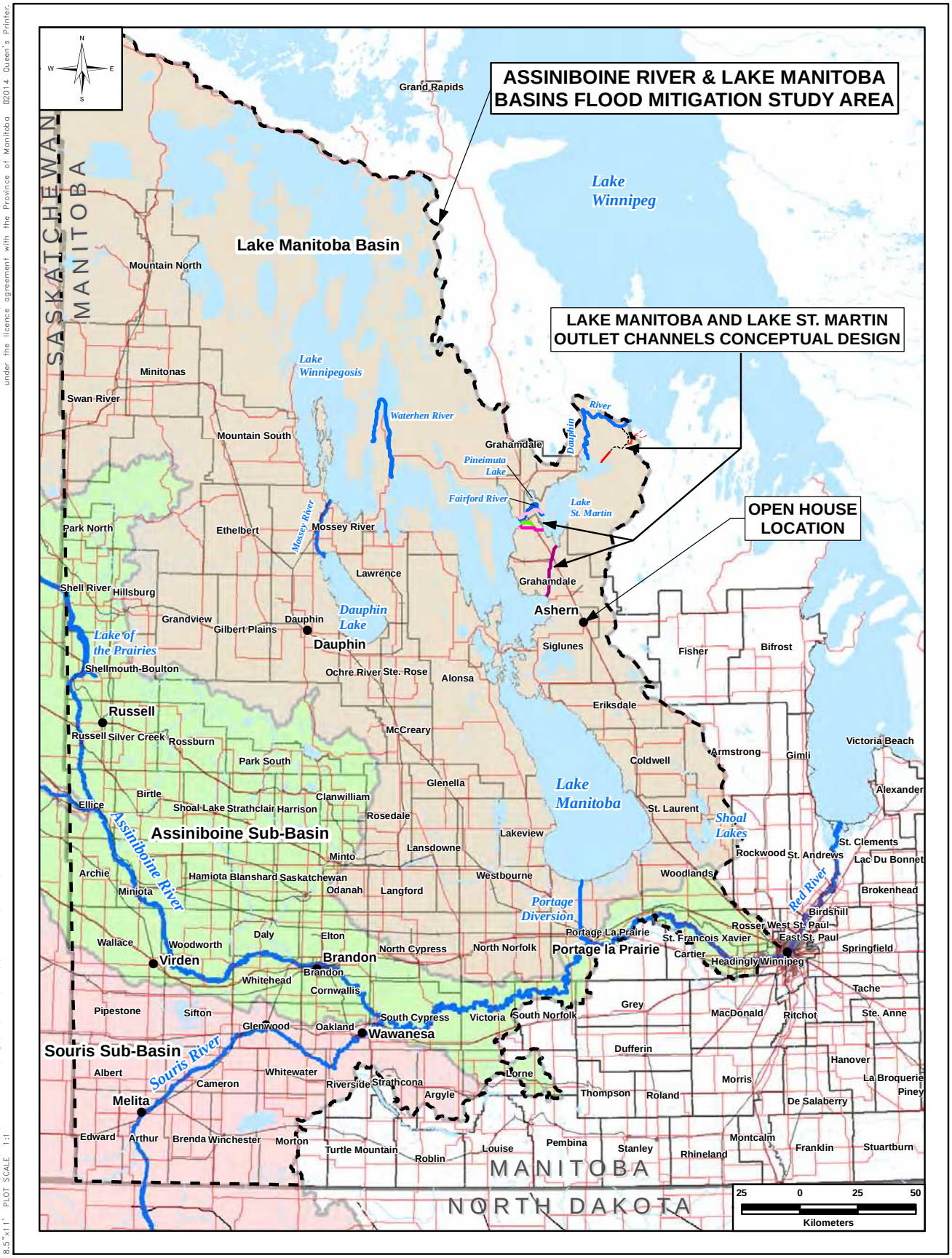

Portions of data presented are owned by the Province of Manitoba and are produced under the licence agreement with the Province of Manitoba ©2014 Queen's Printer.

FileName: P:\Projects\2012\12-0300-011\Dwg\GIS\MXDs\RevA\Open_House\Ashern\12-0300-011-F01_RevA.mxd
8.5"x11" PLOT SCALE 1:1

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Public Open House

Share Your Views on the
Lake Manitoba and Lake St. Martin
Outlet Channels Conceptual Design Study

Manitoba Infrastructure and Transportation is conducting a conceptual-level study for the Assiniboine River and Lake Manitoba watersheds to determine how future flood risks can be mitigated. Part of this study is to identify and evaluate options for enhanced Lake Manitoba and Lake St. Martin outlet capacity.

A public open house has been organized to provide information on the options that are being studied, and to provide you with an opportunity to provide input and your views on this subject. The open house will provide information about:

- study scope and objectives
- preliminary outlet options for Lake Manitoba and Lake St. Martin
- screening-level designs and cost estimates for these options
- effectiveness and impacts of the outlet options

The public open house will be held at:

Centennial Hall

Ashern, Manitoba

September 18 2014, 5:00 p.m.- 8:00 p.m.

Refreshments will be served. Everyone is welcome to attend.

For more information contact:

Colin Siepman, P. Eng.
Senior Project Manager, KGS
Phone: 204-896-1209
Email: CSiepman@kgsgroup.com

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APPENDIX B

LAKE MANITOBA & LAKE ST. MARTIN OUTLET CHANNEL DESIGN OPTIONS OPEN HOUSE SUMMARY REPORT

OPEN HOUSE MATERIALS

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Manitoba

LAKE MANITOBA AND LAKE ST. MARTIN OUTLET CHANNELS CONCEPTUAL DESIGN

Preliminary Options

Lake St. Martin Channel - (May 2012)

Manitoba

WELCOME!

The Province is committed to enhancing Lake Manitoba and Lake St. Martin outlet capacities to better regulate water levels on these lakes. The purpose of this open house is to solicit public opinion on the conceptual design options.

This initiative will help the Province better understand the flood risks and the possible options that can be used to reduce the effects from future flood events. The objective is to evaluate options for an enhanced Lake Manitoba and Lake St. Martin outlet capacity.

Manitoba Infrastructure and Transportation is also evaluating the Assiniboine River and Lake Manitoba watersheds and how we can provide greater protection from flooding risks. Additional public consultations for the Assiniboine River and Lake Manitoba Basins Flood Mitigation Study are planned for this fall.

Fairford River Water Control Structure at PTH 6 (December 2011)

Fairford River Downstream of Fairford Water Control Structure (July 2011)

PTH 6 and the Fairford River Water Control Structure (December 2011)

WE VALUE YOUR FEEDBACK!

Information is being shared today on each of the conceptual options.

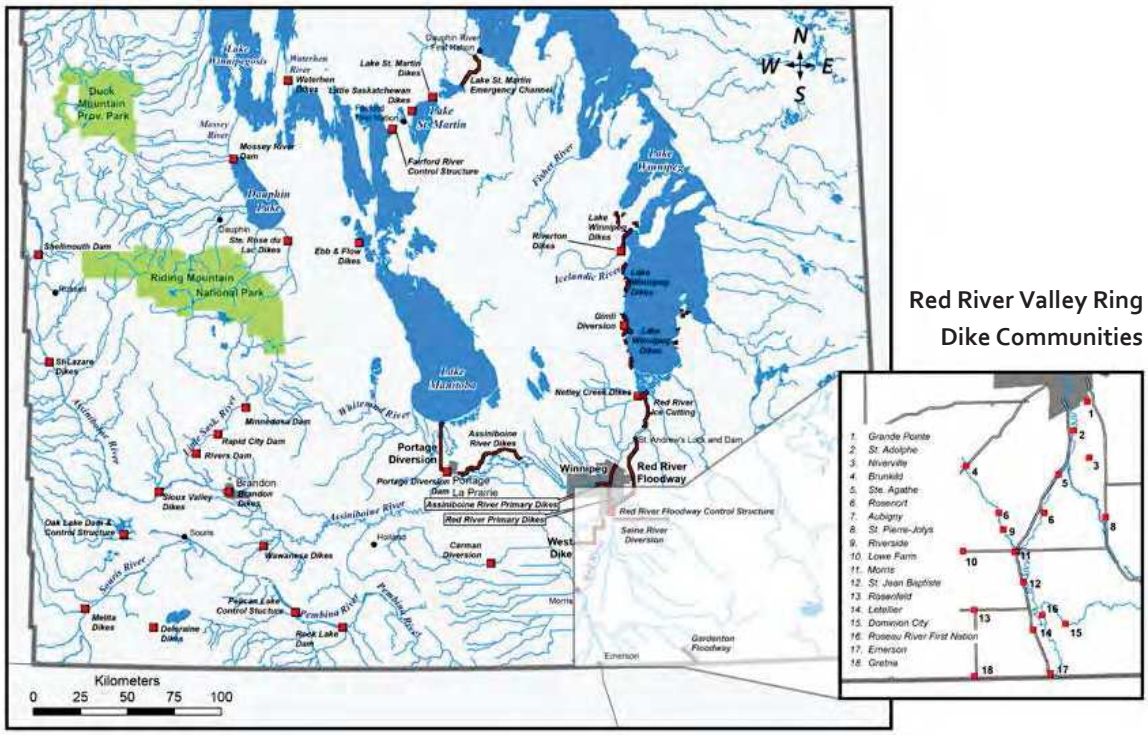
- Ask us any questions you have about this initiative.
- Share your thoughts on the presented options.
- Fill out a comment form.

Manitoba

MANITOBA'S FLOOD PROTECTION MODEL

- Since the1950 flood, Manitoba has developed an extensive integrated flood protection system consisting of the following components:
  - Floodways and diversions
  - Dams
  - Community diking
  - Linear diking
  - Individual flood protection
  - Development controls
- This integrated flood mitigation strategy has worked well as a system since that time and has avoided billions of dollars of damages.

Manitoba's Flood Control Infrastructure System



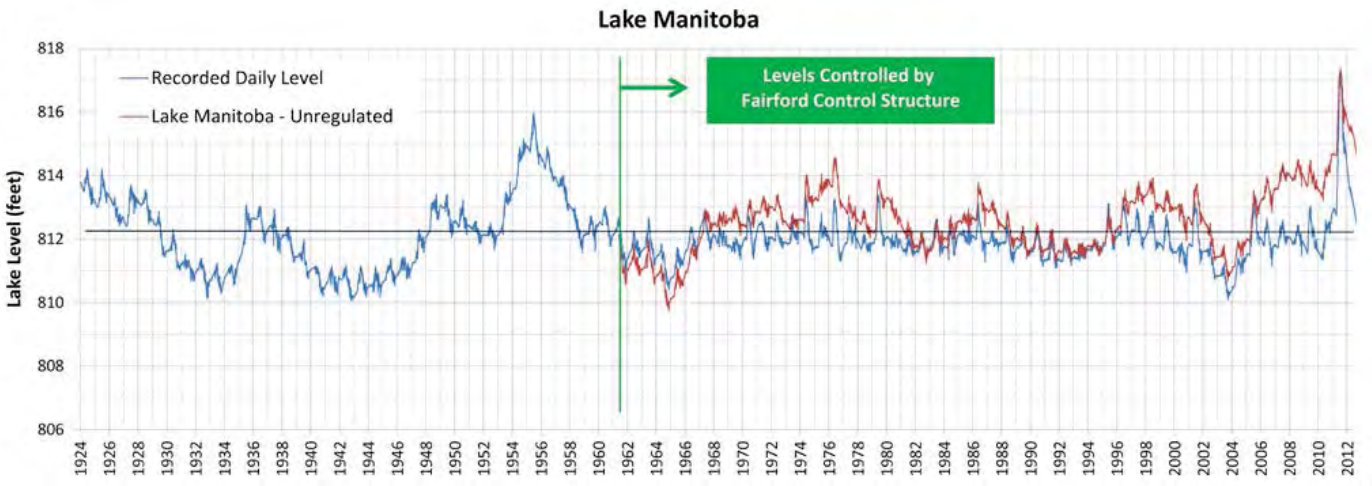
Red River Valley Ring Dike Communities

Manitoba

BACKGROUND / HISTORY

Lake Manitoba Regulation

- In the first half of the 20th century Lake Manitoba experienced significant fluctuations in water levels.
- Lake Manitoba was above flood stage for approximately 3 years in the 1950's.
- The Fairford River Water Control Structure was put into operation in 1961 to better regulate water levels and was the Province's first major flood control infrastructure.
- The design of this structure considered the total inflow to Lake Manitoba including the Portage Diversion, which was completed later in 1970.
- Today, approximately 3000 homes and cottages surround Lake Manitoba.



Lake Manitoba Recorded and Unregulated Levels (source: 2011 Lake Manitoba & Lake St. Martin Regulation Review Report)

BACKGROUND / HISTORY
(Continued)

Lake St. Martin

- Additional outlet capacity for Lake St. Martin was not part of the original Fairford River Water Control Structure Project.
- Options to alter the water level regimes on Lake St. Martin, including additional Lake St. Martin outlet capacity, were previously studied by the Manitoba Water Commission in 1978 but were not recommended.

Portage Diversion

- The Portage Diversion was completed and put into service in 1970.
- The Portage Diversion provides significant water level benefits to:
  - City of Winnipeg.
  - City of Portage la Prairie.
  - Downstream communities on the Assiniboine and La Salle Rivers.
- The benefits for each of the areas vary depending on the size and extent of each flood.

The current system has worked to maintain Lake Manitoba water levels below flood stage between 1961 and 2011.

Manitoba

OTHER FLOOD MITIGATION INITIATIVES

- **Assiniboine River and Lake Manitoba Basins Flood Mitigation Study.**

- **Review of operating guidelines for key provincial flood control infrastructure:**
  - Red River Floodway.
  - Portage Diversion.
  - Fairford River Water Control Structure.

- **Significant investments in flood mitigation:**
  - Individual flood protection province-wide including Lake Manitoba.
  - Upgrades to the Provincial Assiniboine River dikes and reviewing options to increase flows downstream of Portage la Prairie.
  - Community flood protection province wide including Brandon.
  - Upgrades and rehabilitation to existing flood control infrastructure.

- **Evaluation of many additional flood mitigation demands.**

Manitoba

LAKE MANITOBA & LAKE ST. MARTIN FLOOD MITIGATION STRATEGY

The overall strategy for Lake Manitoba and Lake St. Martin flood mitigation includes multiple components that are intended to work together:

- Additional Lake Manitoba and Lake St. Martin outlet capacity.
- Individual home, cottage and business & community flood protection at 2011 flood protection levels (FPL's) 6 to 8 feet above the 814.0 feet flood stage.
- Enhancing Assiniboine River Dikes to increase reliable flows east of Portage la Prairie.
- Shellmouth Dam enhancements.
- Land use planning initiatives.

Home / Cottage Raised to FPL

Lake St. Martin Channel – (October 2013)

Manitoba

NEED FOR ADDITIONAL OUTLET CAPACITY

Lake Manitoba & Lake St. Martin:

- Record flooding in 2011 and 2014 resulted in unprecedented inflows.
- Both lakes peaked well above flood stage.
- Evacuations were required and significant damage occurred to hundreds of properties.
- An emergency outlet channel was constructed at Lake St. Martin in 2011 to lower both lakes. The channel was closed in November 2012 as required by the Federal regulatory authorization.
- Heavy rains in 2014 resulted in Lake Manitoba and Lake St. Martin levels peaking above flood stage.
- The emergency outlet channel was re-opened in July 2014.

Post 2011 Flood Review:

- After the 2011 flood, the Manitoba Government commissioned the Lake Manitoba & Lake St. Martin Regulation Review Committee and also the 2011 Flood Review Task Force.
- The Committee and the Task Force both made specific recommendations for permanent additional outlet capacity from Lake Manitoba and Lake St. Martin to improve the control of water levels on both lakes. The Province accepted this recommendation.

Manitoba

OBJECTIVES

The objectives of the conceptual design include:

- Identify outlet channel options to increase outflow capacity for Lake Manitoba and Lake St. Martin;
- Develop conceptual designs and cost estimates for these options;
- Determine the benefit of an outlet channel to reduce water levels on both lakes for a range of outflow capacities;
- Perform a preliminary review of potential environmental concerns;
- Recommend which outlet options should be pursued; and
- Share information with the public and seek input.

Main Market Road Bridge downstream of the Fairford River Water Control Structure - (July 2011)

Manitoba

PRELIMINARY OPTIONS

Lake Manitoba Outlet Channel

Option A – Twinning the Fairford River

- Twinning of the existing Fairford River from Lake Manitoba to Lake St. Martin; crosses through the Fairford River at two locations.
- Situated primarily on Pinaymootang First Nation Land.
- New upstream and downstream control structure and bridge; new municipal bridges.
- 12.5 km channel; considered for a range of flows.

Option B – Channel south of Pinaymootang First Nation

- A new channel connecting Portage Bay on Lake Manitoba with Lake St. Martin.
- Situated on privately held land and crown leased land.
- New control structure and bridge; new municipal bridges.
- Requires rock excavation.
- 11.5 km channel; considered for a range of flows.

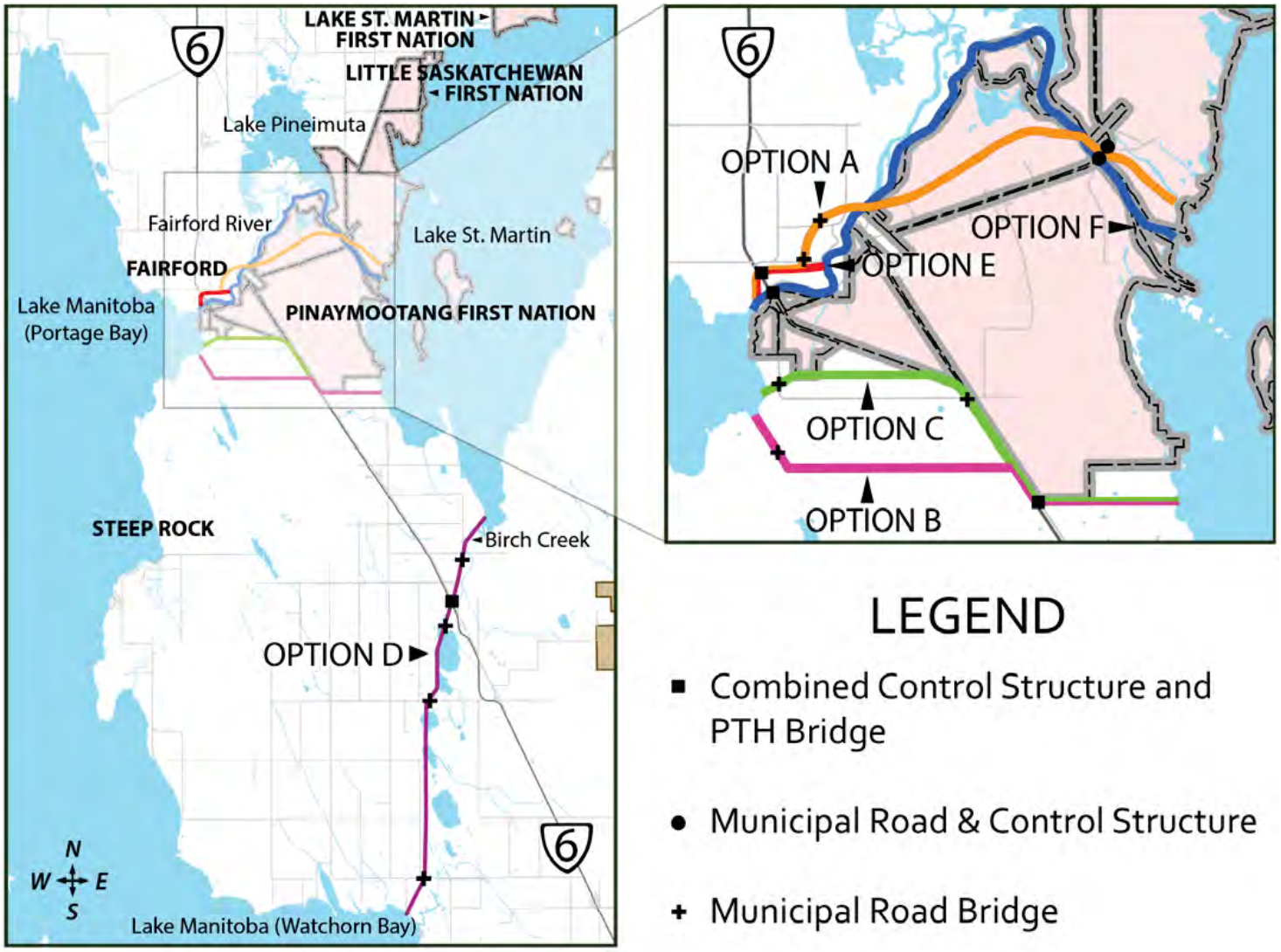
Option C – Channel slightly less south of Pinaymootang First Nation

- A new channel connecting Portage Bay on Lake Manitoba with Lake St. Martin.
- Situated on privately held land and crown leased land.
- New control structure and bridge; new municipal bridges.
- More earth and rock excavation than Option B.
- 11.6 km channel; considered for a range of flows.

Manitoba

PRELIMINARY OPTIONS

Lake Manitoba Outlet Channel



Map showing Lake Manitoba Outlet Channel options

Manitoba

PRELIMINARY OPTIONS

Lake Manitoba Outlet Channel

Option D – Channel following Birch Creek

- A new channel connecting Watchorn Bay on Lake Manitoba to the outlet of Birch Creek on Lake St. Martin.
- Situated on privately held land and crown leased land.
- New control structure and bridge; new municipal bridges.
- 22.8 km channel; considered for a range of flows.

Option E – Bypass channel north of the Fairford River Water Control Structure

- New channel around the Fairford River Water Control Structure to the north and then merges with the Fairford River a short distance downstream.
- Situated on privately held land and crown leased land.
- New control structure and bridge.
- 2.15 km channel; considered for a range of flows to a maximum 3,750 cfs (greater flows not feasible).

Option F – Expansion of the Fairford River and Fairford River Water Control Structure

- Widening of the Fairford River and Fairford River Water Control Structure (including dredging of the inlet channel).
- Situated primarily on Pinaymootang First Nation land.
- 18.7 km channel; considered for a range of flows.

Manitoba

PRELIMINARY OPTIONS
Lake St. Martin Outlet Channel

- The Lake St. Martin Outlet Channel includes the following three reaches:
  - **Reach 1** – New inlet control structure and existing 6 km channel from Lake St. Martin to Big Buffalo Lake Bog.
  - **Reach 2** – 2.9 km length of Buffalo Creek downstream of Big Buffalo Lake.
  - **Reach 3** – New control structure and channel from Buffalo Creek to Lake Winnipeg.
    - **Option JB** – 8 km channel to Lake Winnipeg at Johnson Beach.
    - **Option WP** – 9.8 km channel to Lake Winnipeg south of Willow Point.

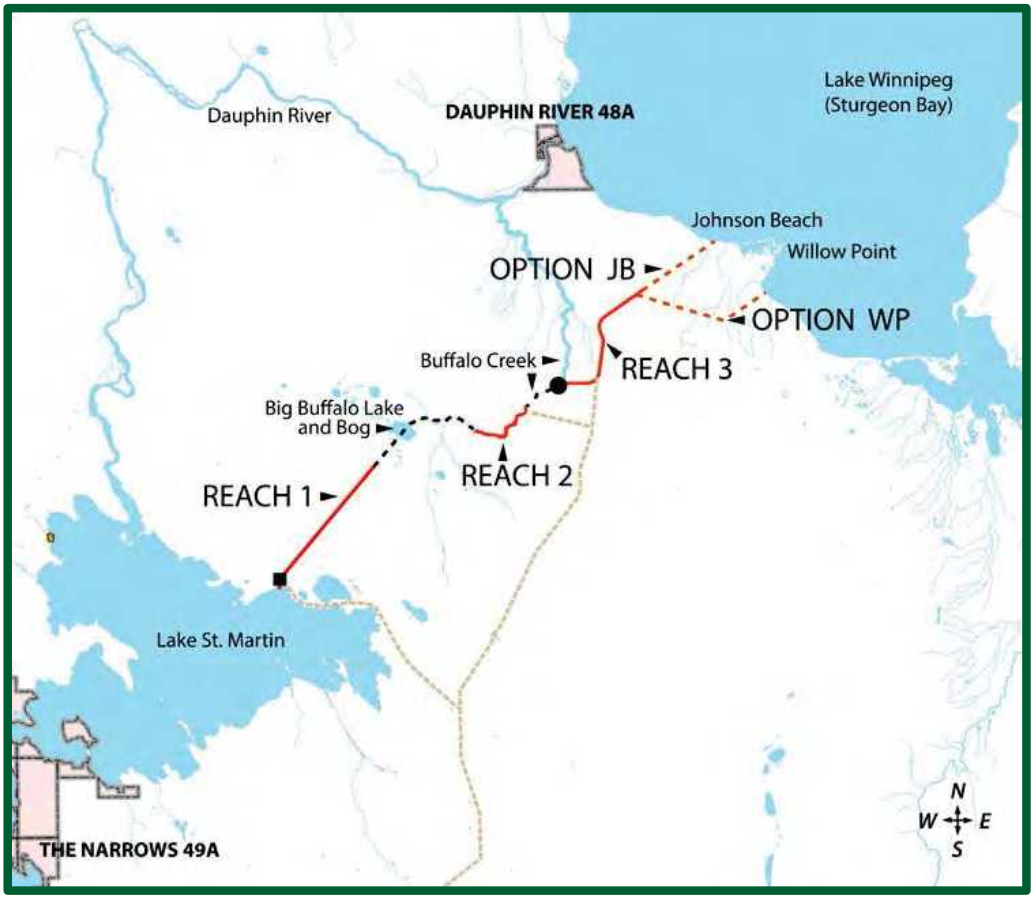
- The existing emergency outlet channel capacity is required to mitigate the effects of the 1961 increase to the Fairford River Water Control Structure outflows.

- Therefore, the existing Lake St. Martin outlet channel must be increased in capacity to accommodate an additional Lake Manitoba outlet channel.

Manitoba

PRELIMINARY OPTIONS

Lake St. Martin Outlet Channel



LEGEND

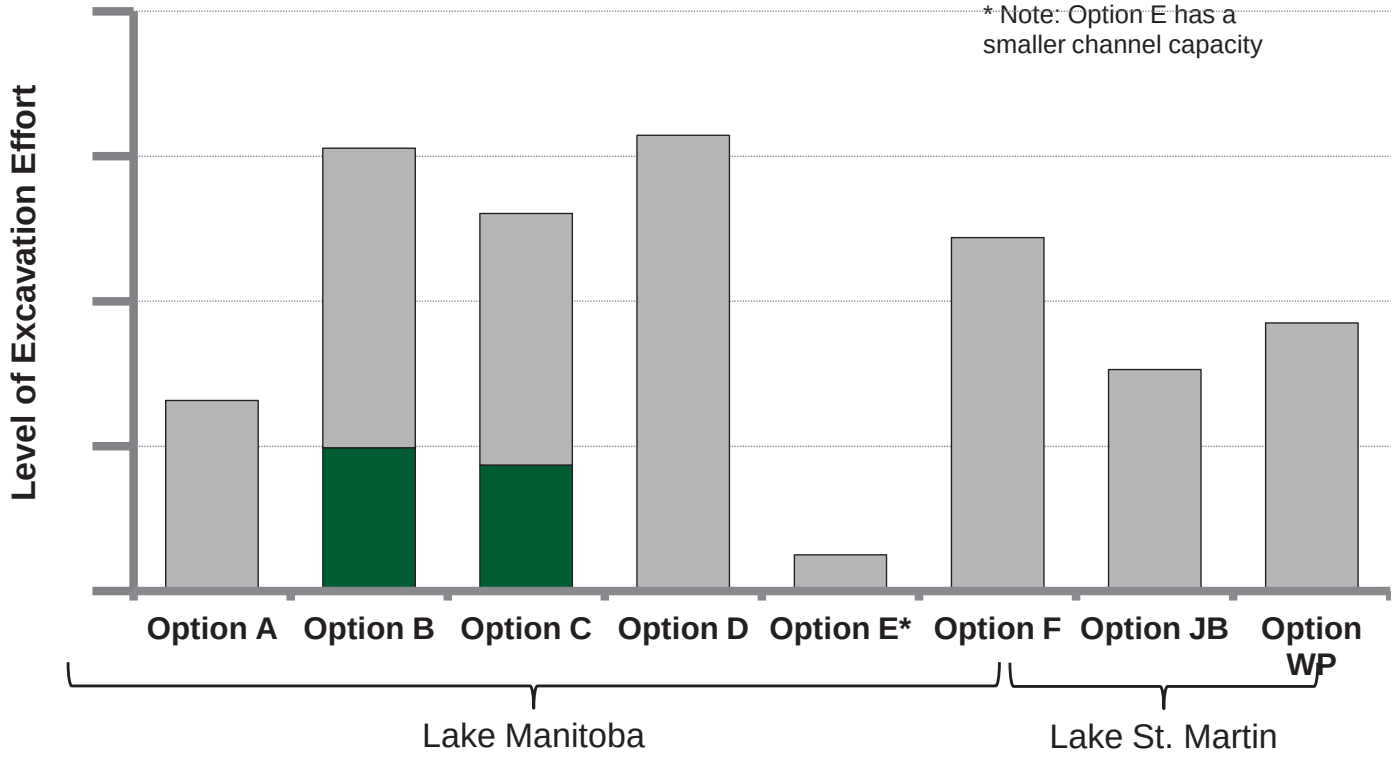
- ■ Inlet Control Structure
  - Regulates flow from Lake St. Martin to Reach 1
- ▬ ▬ Proposed Access Road
- ● Control Structure
  - Diverts flow from Buffalo Creek to Reach 3 during flood operation

Map showing Lake St. Martin Outlet Channel options

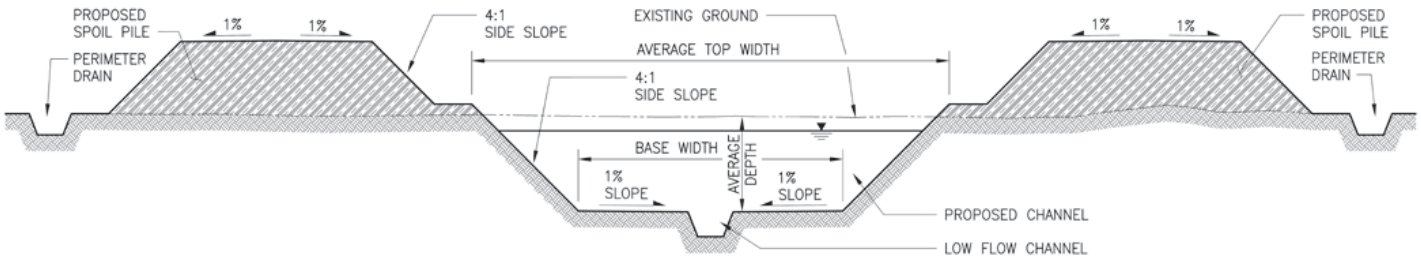
Manitoba

OUTLET CHANNELS

Relative Excavation Effort for Channel Options



Typical Cross Section in Earth



Manitoba

CHANNEL CAPACITY

- Manitoba is committed to the construction of additional outlet capacity for both Lake Manitoba and Lake St. Martin.

- While the conceptual design process considered a range of flows, the assessment indicates that a new channel on Lake Manitoba with a capacity in the range of 5,000 to 7,500 cfs should be pursued.

- The assessment considered:
  - The overall Lake Manitoba / Lake St. Martin flood mitigation strategy.
  - Flood protection standards that are consistent with the recommendations of the Lake Manitoba / Lake St. Martin Review Committee.
  - Economic aspects.

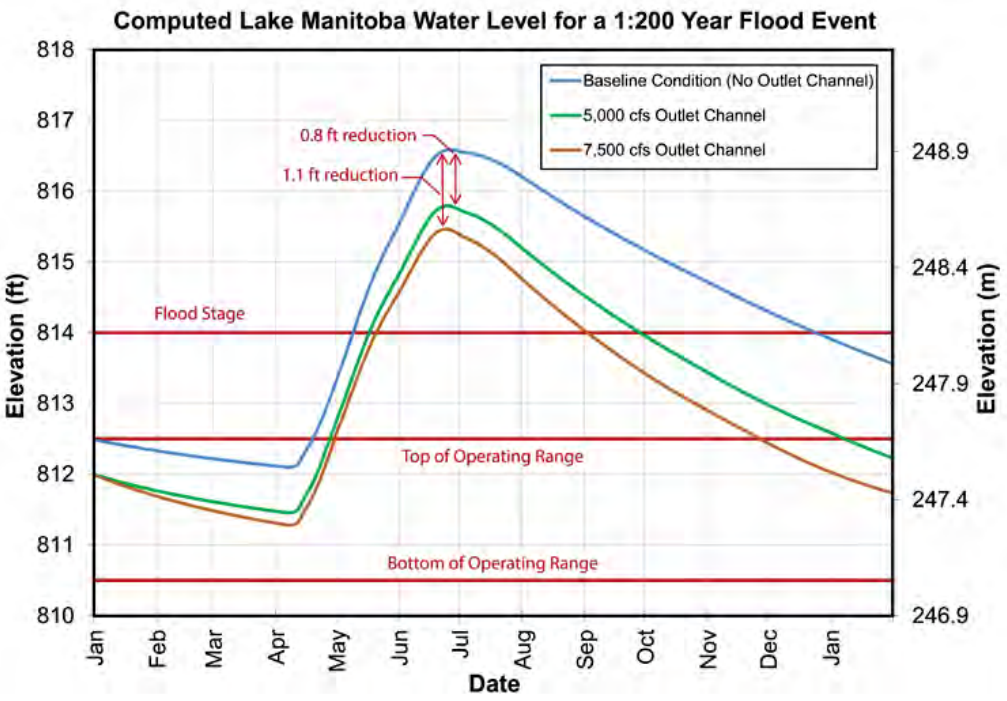
- The combination of additional outlet capacity, individual and community protection, increased Lower Assiniboine River capacity and other initiatives will make a significant difference in mitigating impacts of large floods such as 2011 and 2014.

- Large scale outlets alone, such as one twice the proposed capacity, will not reduce water levels on Lake Manitoba below the 814 feet flood stage in a 1:200 year event.

Manitoba

BENEFIT TO LAKE MANITOBA WATER LEVELS

- For a 200-year event, the peak water level without wind on Lake Manitoba <u>without a new outlet channel</u> would be about 248.9 m (816.6 ft).
- A new outlet channel would reduce the peak Lake Manitoba water level by:
  - 0.24 m (0.8 ft) for a 5,000 cfs channel.
  - 0.34 m (1.1 ft) for a 7,500 cfs channel.



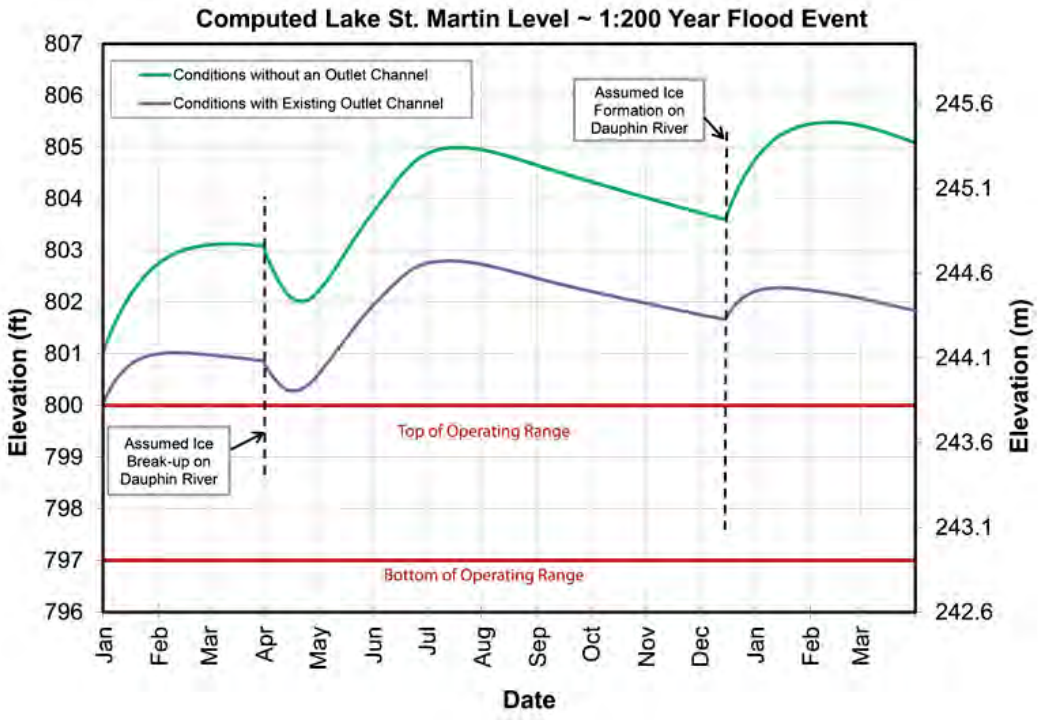
Note: Additional lowering of Lake Manitoba water levels may be achieved by increasing the current capacity of the Lower Assiniboine River.

The magnitude of these peak level reductions would be very similar for floods ranging from a 100 to a 500-year event.

The 2011 flood on Lake Manitoba was estimated to be greater than a 300-year event. The peak water level without wind in 2011 was 249.1 m (817.2 ft).

BENEFIT TO LAKE ST. MARTIN WATER LEVELS

- For a 200-year event, the peak water level without wind on Lake St. Martin without the emergency outlet channel would be about 245.5 m (805.4 ft).
- The existing emergency outlet channel will reduce the 200-year peak Lake St. Martin level by 0.8 m (2.7 ft).
- This peak level reduction would be very similar for all options since a Lake Manitoba outlet channel would be combined with an expanded Lake St. Martin outlet channel.



The magnitude of these peak level reductions would be very similar for floods ranging from a 100 to a 500-year event.

Manitoba

ADDITIONAL BENEFITS OF LAKE MANITOBA AND LAKE ST. MARTIN CHANNELS

- Additional Lake Manitoba and Lake St . Martin outlet capacity provides greater flexibility in operating the provincial water control system and results in benefits to the Lower Assiniboine River, including the City of Winnipeg .

- Additional outlet capacity and individual flood protection around Lake Manitoba helps mitigate the effects of the Portage Diversion on Lake Manitoba water levels.

Lake St. Martin Channel – (December 2011)

Manitoba

ESTIMATED COSTS ($MILLIONS) OF OUTLET CHANNEL OPTIONS

Lake Manitoba Flow Range of 5,000 to 7,500 cfs
Lake St. Martin Flow Range of 9,000 to 11,500 cfs

		Bridge/ Control Structures	Excavation	Access	Land Acquisition	Construction Camp	Mitigation/ Revegetation	Eng. & App./ Contingency	Total Costs
Lake Manitoba	**Option A**	9 - 14	32 - 51	7 - 9	1 - 2	2 - 3	7 - 11	27 - 44	**85 - 134**
	Option B	6 - 8	91 - 143	2 - 3	1 - 2	4 - 7	7 - 11	58 - 92	**169 - 266**
	Option C	6 - 8	78 - 124	4 - 6	1 - 2	4 - 6	7 - 10	51 - 82	**151 - 238**
	Option D	7 - 10	63 - 97	8 - 11	1 - 2	4 - 5	10 - 14	48 - 73	**141 - 212**
	Option E[(1)]	8 - 14	7 - 11	--	0 - 1	0 - 1	2 - 3	5 - 15	**22 - 45**
	Option F	16 - 27	112 - 190	4 - 7	2 - 3	6 - 10	16 - 27	83 - 141	**239 - 405**
Lake St. Martin	**Option JB**	15 - 21	32 - 52	35 - 40	--	4 - 5	8 - 11	48 - 67	**142 - 196**
	Option WP	15 - 21	38 - 60	35 - 40	--	4 - 6	9 - 12	52 - 73	**153 - 212**

Notes: (1) Cost estimate for Option E based on a flow range of 2,500 cfs to 3,750 cfs.
(2) Costs include land purchase, earth and rock excavation, rock riprap to reduce erosion, control structures, bridges, access roads and environmental mitigation costs.
(3) Total cost includes 20% for Engineering & Approvals and 30% for Contingency.
(4) A combination of one Lake Manitoba & one Lake St. Martin option is required.
(5) Lake St. Martin options include expansion to existing emergency channel to accommodate Lake Manitoba outlet channel options.

Manitoba

POTENTIAL ENVIRONMENTAL CONCERNS

Construction and operation of an outlet channel between Lake Manitoba and Lake St. Martin will result in various physical, bio-physical and social environmental concerns.

Each channel option was assessed and rated on a preliminary understanding of the potential risks and concerns. This information will be examined in greater detail with the intention of modifying the design to help mitigate identified concerns.

Potential Environmental Risks/Concerns	Lake Manitoba						Lake St. Martin	
	Option A	Option B	Option C	Option D	Option E	Option F	Option JB	Option WP
● low ● medium ● high **Rating**	6	1	2	3	4	5	2	2
Access Disturbance to PTH 6 and municipal roads during construction	●	●	●	●	●	●	●	●
Fish Habitat Alteration or loss of fish habitat due to structures, shoreline excavation, altered flow patterns, flooding, and water quality changes	●	●	●	●	●	●	●	●
Flooding Flooding of lakes, creeks, and wetland complexes	●	●	●	●	●	●	●	●
Groundwater Seepage of groundwater into the channel may lower the water table and affect groundwater wells in the vicinity of the channel	●	●	●	●	●	●	●	●
Land Use Land acquired for the project would affect current and adjacent land use	●	●	●	●	●	●	●	●
Sedimentation Potential increases in shoreline, lake bed and creek erosion can have an adverse effect on fish and other aquatic biota	●	●	●	●	●	●	●	●
Vegetation Loss due to clearing, excavation, blasting, introduction of weed species and dust settling on adjacent vegetation during construction	●	●	●	●	●	●	●	●
Water Quality Increases in organic matter, changes in dissolved oxygen, pH levels and methylmercury	●	●	●	●	●	●	●	●

Manitoba

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Lake Manitoba Outlet Channel

Channel Capacity

- Designs should consider capacities in the 5,000 to 7,500 cfs range.
- To be refined in the next stage of design.

Alignments

- Options C and D
  - Both options should be considered for more detailed evaluation as part of the next stage of design based on their cost effectiveness and high environmental ratings.
  - Preliminary costs range between $141 Million and $238 Million.

Option C Overview Towards Lake Manitoba (June 2011)

Option D Overview Looking South (June 2011)

Option D at Watchorn Bay (June 2011)

Manitoba

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Lake St. Martin Outlet Channel

Permanent Channel

- The Lake St. Martin Emergency Outlet Channel should be made permanent with additional capacity to accommodate the increased inflows from a new Lake Manitoba outlet channel.
- A control structure and permanent access road should be constructed.
- Preliminary costs range between $142 Million and $212 Million.

Reach 3 Options

- Options for the Reach 3 outlet at Johnson Beach and south of Willow Point have similar ratings, therefore, both should proceed for more detailed evaluation.
- The Dauphin River First Nation has indicated concerns with the Johnson Beach option.
- At this stage, the Willow Point option is preferred.

Big Buffalo Lake – (July 2011)

Buffalo Creek – (August 2013)

Johnson Beach & Willow Point – (October 2013)

Manitoba

NEXT STEPS SCHEDULE

The next stages of the project include:

- Aboriginal consultations.
- Environmental and regulatory reviews.
- Land acquisition.
- Completion of the design.
- Further public engagement.
- Construction.

Lake St. Martin Channel – (May 2012)

Main Market Road Bridge downstream of the Fairford River Water Control Structure – (July 2011)

Manitoba

SHARING YOUR PERSPECTIVES

The project team would like to know your thoughts based on the information that has been shared here today.

Your input/feedback will help us as we proceed with further work on the design.

- Comment forms are available for you to fill out.
- Forms can also be submitted electronically or by mail, so long as they are received before October 18, 2014.
- Details are provided on the comment form. Please ask a project team member to help you if you have any questions.

Contact information: feedback@floodstudy.ca

Mailing address:

Colin Siepman, P.Eng., KGS Group
KGS Place
3rd Floor - 865 Waverley St.
Winnipeg, MB R3T 5P4

THANK YOU!

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Manitoba

Lake Manitoba and Lake St. Martin Outlet Channels Conceptual Design

Open House Feedback Form

Please take this opportunity to fill in this brief questionnaire based on the information provided at this event. Please drop the completed comment form in the box provided. Comments may also be mailed or emailed to the addresses provided on the back of this form.

Your Name (optional): _____

Which of the following best describes your personal connection to this process (select all that apply)?

- ☐ Homeowner
- ☐ Cottage Owner
- ☐ Farmer/Rancher
- ☐ Business Owner
- ☐ First Nations Member
- ☐ First Nations Leader
- ☐ Municipal Elected Official
- ☐ Municipal Employee
- ☐ Provincial Elected Official
- ☐ Provincial Employee
- ☐ Other: _____

Did you participate in the Individual Flood Protection Initiative (IFPI)? ☐ Yes ☐ No

Did you participate in the Financial Assistance for Cottage Owners (FACO)? ☐ Yes ☐ No

Section 1: Open House Feedback

Now that you have reviewed the information shared here and/or spoken with open house staff, please share your feedback on the open house today.

How did you hear about the event?

- ☐ Newspaper
- ☐ Radio/public service announcement
- ☐ Letter Invitation
- ☐ Email
- ☐ Word of Mouth
- ☐ Other_____

Was the information presented in the boards and/or your discussions with open house staff helpful?

- ☐ Yes
- ☐ No

How could this Open House event been more helpful to you?

_____

_____

_____

_____

_____

Why did you decide to attend the Open House today?

What perspectives would you like to share about the options that have been shared today? Do you have a preference and why?

What other comments or questions would you like to provide today?

If you would like us to forward you a copy of the open house boards please include your email address:

Please send me a copy of the open house boards. Email Address:

THANK YOU FOR TAKING THE TIME TO PROVIDE YOUR FEEDBACK.
YOUR PARTICIPATION IS VERY IMPORTANT TO THE DESIGN PROCESS.

If you would prefer, comments may also be submitted via mail or email no later than October 18, 2014.

Email: feedback@floodstudy.ca

Mailing address:
Colin Siepman, P. Eng KGS Group
KGS Place
3rd Floor-865 Waverley St.
Winnipeg, MB R3T 5P4

Draft Project Schedule
Lake Manitoba & Lake St. Martin Channels

1 Provincial Press Release
2 Assiniboine R and LMB Basins Study
3 Proposed Open House (June 18)
4 Open House (Updated)
5 Functional Design of Alternatives
6 Optimize Hydraulic Design of Preferred Alternatives
7 Geotech/survey Field Investigations
8 Refine Costs and Quantities of Preferred Alternatives
9 Identify Environmental Assessment (EA) Requirements for Future Licensing
10 Environmental Field Programs
11 Fish Passage Assessment - Fairford R. Water Control Structure
12 Analysis of Social Impacts of Preferred Alternatives
13 Aboriginal Consultation
14 Selection of Preferred Alternative
15 Environmental Impact Statement (EIS) Process
16 Develop Project Description (Consultant)
17 Review of Project Description (Canadian Environmental Assessment Agency - CEAA)
18 EA Commencement (CEAA)
19 Develop EIS (Consultant); Including Any Additional Field Work
20 Submit EIS to CEAA
21 EA Analysis and Report (CEAA)
22 EA Decision (CEAA)
23 Detailed Design
24 LSM Access Rd - Environmental Permitting
25 LSM Access Rd - Geotech Investigations
26 LSM Access Rd - Design
27 LSM Access Rd - Tender
28 LSM Access Rd - Construction
29 Tender LMB & LSM
30 Work Permits/Construction Camp Permitting
31 Construction of the LMB Channel
32 Revegetate LMB Channel
33 Expansion of the LSM Channel
34 Revegetate LSM Channel
35 Operation of LMB and LSM Channels

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APPENDIX C

LAKE MANITOBA & LAKE ST. MARTIN OUTLET CHANNEL DESIGN OPTIONS OPEN HOUSE SUMMARY REPORT

SIGN-IN SHEETS AND OPEN HOUSE COMMENTS

(CONFIDENTIAL)

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OPEN HOUSE SIGN IN SHEET

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
ARNE LINDELL	<original signed by>	ERIKSDALE
REG SCHWARTZ	<original signed by>	ASHERN
Frank Kenyon	<original signed by>	Fairford
Richard YANKE	<original signed by>	Ashern
NORINE YANKE	<original signed by>	ASHERN
DON MEADOWS	<original signed by>	~~SLK~~ MB NARROWS
William Sinclair	<original signed by>	Little Saskatchewan
DAVE OLSON	<original signed by>	PEONAN POINT
RANDY ELCAESON	<original signed by>	SIGLUNES
Gerald Falk	<original signed by>	Ashern
Richard Postlethwaite	<original signed by>	Ashern
Brian Thompson	<original signed by>	Moosehorn
HARRY FREDERICK	<original signed by>	ST. LAURENT
TED FUCHS	<original signed by>	Ashern
RICHARD EBBERS	<original signed by>	ASHERN
Marlene METNGR	<original signed by>	Moosehorn
Donna Steinthorson	<original signed by>	Vogar
R. Linden	<original signed by>	L. Sask.
Karen Williamson	<original signed by>	Ashern
Larry Williamson	<original signed by>	Ashern
Harlan Beckman	<original signed by>	Eriksdale
Albert Murray	<original signed by>	Fairford
Brad Dreger	<original signed by>	Grahamdale

OPEN HOUSE SIGN IN SHEET

2 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Lyle Finney	<original signed by>	R.M. of Alonsa
Bob & Wendy Hoffman	<original signed by>	Sugar Point Lundar
Robert H Mutner	<original signed by>	Westborn Bay Woodbury
Michael Sweeney		Vogar MB
Kathy Turner	<original signed by>	Steep Rock
Cal Turner	<original signed by>	Steep Rock
Edith Peterson	<original signed by>	Ashern.
Mark Jacques	<original signed by>	Eddystone
Andy Thibert	<original signed by>	Cayer
Reg Daroch	<original signed by>	Moosehorn MB.
Rose-Marie Bolay	<original signed by>	Fairford, MB
Caron Clarke	<original signed by>	Ashern, MB (MBP director)
Tim Clarke	<original signed by>	" "
Diane Hardy	<original signed by>	Ashern MB
Kevin Yuill	<original signed by>	Lake Manitoba Rehab Committee
ELSIE KIESMAN	<original signed by>	Moosehorn, MB
Kimmy [illegible]	<original signed by>	Fairford MB
Wandis Day	<original signed by>	Eriksdale, MB
MERLE DAY	<original signed by>	Eriksdale
Henry Rosing	<original signed by>	Lake Francis
Leroy Thompson	<original signed by>	Little Saskatchewan F.N
Rhonda Parenteau	<original signed by>	Little Saskatchewan F.N.
BRIAN SANDERSON	<original signed by>	Fair Ford

OPEN HOUSE SIGN IN SHEET

3 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Barry MacBride	<original signed by>	BDM Projects
ORVAL DELORME	<original signed by>	St. Laurent
KAM BLIGHT	<original signed by>	RM PORTAGE
DOROTHY HALLDORSON	<original signed by>	R.M. Siglunes Vogar
Bill KAKEWASH	<original signed by>	LAKE ST MARTIN FN.
SELENA HOGUE	<original signed by>	Lundar, MB.
JEAN ALLARD	<original signed by>	St Laurent
Perry Ewasiuk	<original signed by>	St. Martin
Neil Linden		
Mikey PRICE	<original signed by>	Steep Rock
FRED/BETTY Ann SMITH	<original signed by>	St Laurent
ART/JACKIE JONATSON	<original signed by>	Vogar
DAN MEISNER	<original signed by>	MOOSEHORN
AMANDA STEVENSON	<original signed by>	Rm of Coldwell
Maureen Cabins	<original signed by>	MB Beef Producers
Karen Shoup	<original signed by>	OAK POINT
Tom Terchroeb	<original signed by>	Langruth MB
Lydia Johnson	<original signed by>	Langruth MB
THOMAS JOHNSON	<original signed by>	Langruth MB
DOUG PICKELL	<original signed by>	R.M. of Rockwood
Victoria Hofendorf	<original signed by>	RM Siglunes
DON EASTON	<original signed by>	ST. LAURENT
MARG GRANBERG	<original signed by>	ERIKSDALE

OPEN HOUSE SIGN IN SHEET

4 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Michelle Beaudin	<original signed by>	St Laurent (Sandpiper
Florent Beaudin	<original signed by>	St Laurent (Sandpiper
GARRY ASHMAN	<original signed by>	P. of P.
GREG BROWN	<original signed by>	LUNDAR
Tom Metner	<original signed by>	Moosehorn
LINDA ERICKSON	<original signed by>	Lundar
CARL ERICKSON	<original signed by>	Lundar Beach
BRIAN SIGFUSSON	<original signed by>	Lundar M.B
JOE PAUL	<original signed by>	Vogar Man
Stephen Reykdal		Arborn
Diane Priest	<original signed by>	RM. Grahamdale
Linda Watson		
Ted Watson		Erikdale
Peter Bligh	<original signed by>	OAK POINT
Oli H. Olson	<original signed by>	PEONAN POINT
Bea Rosing	<original signed by>	Lake Francis MB
Sharon Bittner	<original signed by>	Birch Bay
Carolee Keen		Steep Rock
Bengt Keen		Steep Rock
Guy Parent	<original signed by>	Fairford
Gene Henry Holowchak	<original signed by>	Ashern
	<original signed by>	Moosehorn
DUNC GEISLER	<original signed by>	

OPEN HOUSE SIGN IN SHEET

5 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
ED BOAHISH	<original signed by>	
DON FISETTE	<original signed by>	MOOSE HORN
Don SCHEDLER	<original signed by>	MOOSEHORN
Gary Colleen Cook	<original signed by>	Moosehorn
Edna Sandreen	<original signed by>	
Floyd Sandreen	<original signed by>	
Ted Cook	<original signed by>	Comfrey M.B.
Stan Asham	<original signed by>	R.M. of Alonsa
KRIS COOK	<original signed by>	NAC Homebrook Peonan PT
Michael + Marilyn Sveistrup	<original signed by>	Vogar MB
JEFF WARD	<original signed by>	EXPRESS WEEKLY
B. POTTINGER	<original signed by>	ASHERN
Ken Peterson	<original signed by>	Asham
Lee Caytell	<original signed by>	Ninette Road
Kurvis Anderson	<original signed by>	Fairford
JANALEA PROCTOR	<original signed by>	WOODLANDS
KEN MURPHY	<original signed by>	ERRESTYLE MB
John Bday	<original signed by>	Fair Ford
TERRY STEMPSON	<original signed by>	RM of Portage
Richard Woodhouse	<original signed by>	P.I.o.y.m.o.u.t.h. First Nation
LYLE HORNBY	<original signed by>	ASHERN
DORRIE KIESMAN	<original signed by>	MOOSEHORN
Rick Bower. 9	<original signed by>	

OPEN HOUSE SIGN IN SHEET

6 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
CLAIRE GIRARDIN	<original signed by>	
Amanda Classen	<original signed by>	
LILLI SCHNEIDER	<original signed by>	
Tony Dyck	<original signed by>	
ELEANOR KOOP	<original signed by>	
J A Kr	<original signed by>	
KATHY BROOKS	<original signed by>	
GARRY IVERSON	<original signed by>	
M Zittlau		
Nyla Gault	<original signed by>	
Greg Johnson	<original signed by>	
ROB BITTNER	<original signed by>	
Louise MacLeod	<original signed by>	
JIM TOWNSEND	<original signed by>	WOODLANDS
Lenore Brown	<original signed by>	
Glen Clements	<original signed by>	
Mark + Patty Dyck	<original signed by>	ASH
W. Whitmore	<original signed by>	
PHILIP THORDARSON	<original signed by>	RM of Lakeview
BRIAN SNIDAL	<original signed by>	Steep Rock
Ken Sigurdson	<original signed by>	OAKVIEW
Ray Bonk	<original signed by>	MOOSEHORN.
Randy Bjarnason	<original signed by>	ASHERN

OPEN HOUSE SIGN IN SHEET

7 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
DAVE & EILEEN DEVANIE	<original signed by>	Dauphin Beach Cottage Association
Jack K.	<original signed by>	Twin Beach Assoc.
GARRY WAHL	<original signed by>	LAKE MAN NARROWS (ALONSA)
PAT WAHL	<original signed by>	LAKE MAN NARROWS
Lorne Henry		Portage la Prairie
Bryan McLaughlin	<original signed by>	Arden
Reynold Bourdeaud'hui	<original signed by>	R.M. of Lorne Somerset
JoAnn Gittoes	<original signed by>	Hillbre — Captain C
Jack Lewis	<original signed by>	R.M. of Alonsa
Bernice Brandson	<original signed by>	KINOSOTA MB RM of Alonsa
MARY ROSCOE	<original signed by>	Dauphin River
DONALD COOK	<original signed by>	Steep Rock Man.
Bob HAYDEN	<original signed by>	ST LAURENT
FRED HARTMANN	<original signed by>	St-Laurent
John Cabak	<original signed by>	Alonsa
SCOTT FORBES	<original signed by>	U OF WINNIPEG
Mark Lewis	<original signed by>	Rm
[illegible]	<original signed by>	RM OF GRAHAMDALE
[illegible]	<original signed by>	Box 30 STE
R.R. Robinson	<original signed by>	
LARRY WEATHERBURN	<original signed by>	Erickdale
KRIS JONASSON	<original signed by>	ASHERN
Lynnette Beamish	<original signed by>	Steep Rock

OPEN HOUSE SIGN IN SHEET

8 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
LYNA GRUBERT	<original signed by>	TWIN LAKES BEACH.
Cindy Arcos	<original signed by>	Steep Rock Beach
Eileen Meisner	<original signed by>	Steep Rock Beach
BILL BROOKS	<original signed by>	TWIN LAKES BEACH
FRED PISELEKICH	<original signed by>	TWIN
LARRY DECKER	<original signed by>	MOOSEHORN.
LAWRENCE BITTNER	<original signed by>	Hilbre
LORRAINE & RICK ASHDOWN	<original signed by>	Twin Lakes Beach
Carolyn Thorlacius	<original signed by>	Ashern Lower Bay Area
Tom Johnson	<original signed by>	Steep Rock.
Derek Johnson	<original signed by>	Oak Point.
David Single	<original signed by>	RM Westbourne
JOHN BEZEMER	<original signed by>	R.M of Siglunes.
ROLAND VAN DE KERCKHOVE	<original signed by>	R M of Lawrence
Margaret Snidal	<original signed by>	RM of Grahamdale
Marshall B.	<original signed by>	Ashern
GREG VINCE	<original signed by>	ASHERN
RUSSELL KEELER	<original signed by>	ASHERN
Samuel Boles	<original signed by>	St Martin
LARRY MILLER	<original signed by>	Deighton Beach
LYNNE MILLER	<original signed by>	" "
L. COX	<original signed by>	TWIN LAKES BEACH
SID BROOMFIELD	<original signed by>	ST LAURENT LAURENTIA BEACH

OPEN HOUSE SIGN IN SHEET

9 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Sylvia Fisette	<original signed by>	RM Grahamdale
ED LECHELT	<original signed by>	ASHERN
DENNIS TUREK	<original signed by>	Twin Lake Beach Assoc
HENRY SIKORA	<original signed by>	Cf west interlake
Michel Trudel	<original signed by>	PC Caucus
JOHN GRAY	<original signed by>	ASHERN.
Shelly Schwitek	<original signed by>	RM Grahamdale
Wayne Burke	<original signed by>	RM Coldwell
Darlene Beach	<original signed by>	RM COLDWELL
Linda Nickel	<original signed by>	RM Grahamdale
Shawn Burke	<original signed by>	Westlake Conservation
BLAIR OLAFSON	<original signed by>	LMN
Mickey Sturges	<original signed by>	RM of Siglunes
Cheryl Smith	<original signed by>	RM of St Laurent
John Shipley	<original signed by>	MIT
Tom Johnson	<original signed by>	RM. ST. LAURENT
Darlene Michalot-Dumas	<original signed by>	Northern Affairs area rancher
Mary Shading	<original signed by>	Ashern man
Charlie J McKay	<original signed by>	Moosehorn Man
Peter Bolay	<original signed by>	RM of Grahamdale/Fairford
house Kennested	<original signed by>	RM of Siglunes
Hector P Skinner	<original signed by>	
NORA SHORTING	<original signed by>	Little Saint. F.N.

OPEN HOUSE SIGN IN SHEET

10 of 11

SEPTEMBER 18, 2014 - ASHERN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Gail B Holmes	<original signed by>	RM ERIKSDALE
Dennis J. Pike	<original signed by>	WILLOW BAY GRAHAMDALE
Charles Doll	<original signed by>	Moosehorn
Evelyn GALL	<original signed by>	R.M. of Grahamdale
Arlene B. Darknell	<original signed by>	R.M. of Eriksdale
John Wainwright	<original signed by>	RM of Eiksdale
David Gall	<original signed by>	Grahamdale
Allen & Gerry Dell	<original signed by>	Lake Manitoba Narrows
Angela Emms	<original signed by>	RM of Rossen
Mike Emms	<original signed by>	Rossen
ARVID NOTTVEIT	<original signed by>	PEONAN PT
DOUG OLIVER	<original signed by>	R.M. of WOODLANDS
CARL FLEURY	<original signed by>	R.M. of Woodlands
Cecil Rawlyk	<original signed by>	RM of Grahamdale
Charlie Hansen	<original signed by>	RM of Siglunes
DIANNE RIDING	<original signed by>	LAKE FRANCIS
GARY WILKINSON	<original signed by>	LAKE FRANCIS
Paul Kuhn	<original signed by>	ST. LAURENT
Scott Kolomaya	<original signed by>	St. Martin
Ted + Monica Fuchs	<original signed by>	Ashern
Dave + Marla Metner	<original signed by>	Dolly Bay Resort Moosehorn
Teresa JOHNSON	<original signed by>	Vosar, MB
GLEN METNER	<original signed by>	MOOSEHORN

Open House SIGN-IN SHEET 11 of 11
Sept. 18, 2014

NAME (please print)	SIGNATURE	AFFILIATION
Derrick Gould	<original signed by>	Pinaymootang FN

APPENDIX Q 3
ASSINBOINE RIVER & LAKE ST. MARTIN
FLOOD MITIGATION STUDY
ROUND 2 PUBLIC CONSULTATION
JANUARY 2015

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ASSINIBOINE RIVER & LAKE MANITOBA BASINS

FLOOD MITIGATION STUDY

ROUND 2 PUBLIC CONSULTATION REPORT

Prepared by:

InterGroup Consultants Ltd.

500-280 Smith Street

Winnipeg, MB R3C 1K2

Submitted to:

KGS Group

865 Waverly Street, 3rd Floor

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January 2015

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1.0 BACKGROUND

Manitoba Infrastructure and Transportation (MIT) commissioned a conceptual-level study for the Assiniboine River and Lake Manitoba watersheds (the Study Area). The purpose of the study is to provide the Province with recommendations on how future flood events can be mitigated along the main stems of the Assiniboine River and Souris River, Dauphin Lake, Lake Manitoba, Lake St. Martin, and the Shoal Lakes. A map of the Study Area is included in Appendix A1.

The study will include examination of a wide range of possible measures, including non-structural measures, such as land use changes, as well as structural measures, such as large dams. The results of this study will be the foundation for flood mitigation programs for the next few decades.

One major component of the study is the public consultation process, which serves as an opportunity to share information with and obtain input from the public.

To prepare for public consultation activities, a draft Public Consultation Strategy (PCS) was developed to help facilitate internal discussions related to identification of potential stakeholders, proposed consultation activities, and overall study messaging. This included creating opportunities for:

- The Study team to share information with stakeholders;
- Stakeholders to ask questions and request more information;
- The Study team and stakeholders to have discussions about potential effects, enhancement of benefits and mitigation options;
- Addressing interests and concerns as they arise throughout the process; and
- Facilitating the regulatory review and approvals by ensuring that an open and transparent public process is documented for the regulators.

1.1 OBJECTIVES

The following principles guided the development and implementation of the public consultation activities:

- **Accessible Process:** Provide opportunities and sufficient time for potentially affected and interested parties to learn about the study and provide input.
- **Transparent and Open Process:** Encourage open dialogue and provide information in a form easily understood by non-experts.
- **Variety of Communication Mechanisms:** Encourage participation through a variety of mechanisms.
- **Responsive Approach:** Consider results of the consultation process and provide feedback on the results of the process.

- **Adaptive Approach:** Adjust the engagement program in response to issues and concerns.

1.2 STAKEHOLDER IDENTIFICATION

Engagement activities provided information to a broad array of individuals and organizations in the study region. The PCS helped identify an initial list of potential target audiences for engagement activities. This list was later refined by the technical committee and study team. The following categories were targeted for engagement:

- **Community Leadership:** Included Reeves and CAOs (Rural Municipalities), Mayor and Councillors (Cities and Towns), and leadership and representatives from First Nation and Metis communities;

- **Local Residents:** Residents within the Study Area including businesses, landowners, interested groups and other interested individuals; and

- **Other Interested Groups:** Included groups with potential interest in the study including local and regional organizations as well as non-governmental organizations.

1.3 TWO ROUNDS OF ENGAGEMENT

The PCS identified two rounds of consultation activities.

- **Round 1: Study Introduction and Information Collection –** The first round of consultation, held in June 2013, effectively communicated as much information as possible regarding the scope of the study. Input from participants was solicited in order to understand their thoughts on what mitigation options should be considered to reduce flood effects and how these options should be prioritized, as well as their preferences for future consultation processes. The perspectives on preferred consultation mechanisms helped to influence the design of Round Two consultation.

- **Round 2: Preliminary Results –** The second and final round of consultation, held in December 2014, presented preliminary results and provided opportunity for feedback prior to the development of the final study report. This round also informed participants about any changes in the Study since Round 1, respond directly to questions about the study, and show how input received in the previous round influenced the study.

2.0 ROUND 1 SUMMARY

The first round of consultation communicated information about the scope of the study, including the objective of the study, the overall scope of the study, anticipated timelines, and how people can get involved. Input from participants was solicited in order to understand their experiences and concerns, as well as their preferences for future consultation processes.

Information was shared at a total of six events: three meetings with community leadership and three public open houses. Events were held at the following locations:

- Dauphin, MB: Super 8 Hotel, June 18, 2013;
- Brandon, MB: Keystone Centre, June 19, 2013; and
- Portage la Prairie, MB: Canad Inns, June 20, 2013.

A complete summary of the Round One consultation process, including copies of the materials and information shared with the public as well as the perspectives and comments received during the process, are included in the Round One Public Consultation Report found in the appendices to this study report.

3.0 ROUND 2 SUMMARY

The second round of consultation shared the preliminary results of the study and offered opportunities for regional stakeholders including the general public to ask questions and provide feedback on the presented information. The comments and perspectives received during this process helped inform the documentation of the final report.

3.1 EVENT COMMUNICATION

Consistent with the approach used in the earlier Round 1 consultation process, community leadership contact information was confirmed and officials sent an invitation by both surface mail and by email to attend briefing sessions held prior to each public open house event. The invitation package contained an invitation letter as well as a map of the study area.

Advertisements were placed in various newspapers approximately three weeks prior to the scheduled event to notify the public of the upcoming open houses. A separate news release was issued December 5 2015 in advance of the events. Non-government organizations and other interested groups were contacted directly by phone and/or email with information about the timing and location of the public house events.

Copies of the key stakeholder invitation letter packages are included in Appendix A1.

3.2 ROUND 2 PUBLIC CONSULTATION MATERIALS

Materials prepared for Round 2 activities included the following:

- **Storyboards:** Storyboards were used to communicate key messages of the study. Storyboards included information on why the study is being conducted, summaries of potential mitigation options, an overview of potential benefits/costs and development priorities, and the anticipated study process and schedule. Information was also provided on the potential process moving forward, including consultation activities and future opportunities to participate.

- **Comment Forms:** Attendees at both the leadership sessions and open houses were asked to share their concerns/issues about the proposed Study, as well as provide feedback on the public involvement process. Questions and comments received during the stakeholder briefing sessions were recorded and summarized. For discussions with open house participants, a study team member documented summaries of their individual interactions. More than 400 comment forms were distributed during Round 2 public engagement and attendees were encouraged to take additional copies for those who may have had comments but were unable to attend the sessions. Participants were given the option to fill out a form at the event, mail the form in, or email comments to the Study email address – feedback@floodstudy.ca.

Copies of the storyboards and the comment form used during Round 2 are included in Appendix A2.

3.3 ROUND 2 PUBLIC CONSULTATION EVENTS

Information was shared at a total of six events: three meetings with community leadership and three public open houses. Events were held at the following locations:

- Dauphin, MB: Parklands Recreation Centre, Lion's Den, December 8, 2014;
- Brandon, MB: Keystone Centre, December 9, 2014; and
- Portage la Prairie, MB: Herman Prior Centre, December 10, 2014.

3.3.1 Key Stakeholder Briefing Session

December 8-10, 2014, 1:00 pm – 3:00 pm

Key stakeholder briefing sessions were held to provide local leadership (municipal leadership, First Nation Chiefs, and Manitoba Metis Federation) with current information regarding the study in advance of the public open house. It provided community leadership opportunities to be briefed on the study and ask questions. At least two weeks prior to these events, identified stakeholders were mailed invitation letters that included a study area map and an open house poster.

Table 3-1 provides a list of local leadership that participated in the key stakeholder briefing sessions. Copies of the mailing list and the invitation letter are included in Appendix A1. The participant list is included in Appendix A3.

Table 3-1: Key Stakeholder Briefing Session Attendance

Location	Attendees
Dauphin Parkland Recreation Centre Lion's Den December 8, 2014	• Ron Ryz, RM of Dauphin • John Dewarle, RM of Dauphin • Don Dunfield, RM of Dauphin • Dennis Forbe, RM of Dauphin • Arnthor Jonasson, RM of West Interlake • John Bezemer, RM of West Interlake • John Wainwright, RM of West Interlake • Wayne Hildebrand, RM of Langford • Clinton Cleave, RM of Ochre River • Walter Tymchuk, RM of Lawrence-Lakeshore • Maurice Maguet, RM of Ste. Rose
Brandon Keystone Centre December 9, 2014	• Dennis Fouillard, RM of Archie-Ellice • Blair Woods, RM of Grassland • Joe Goodwill, RM of Glenwood • Bob Adriaansen, Reeve, Municipality of North Cypress-Langford • Will Rey, Councillor, RM of Victoria • Reg Marginet, Councillor, RM of Victoria • Harold Purkess, Reeve, RM of Victoria • Tom Mollard, Hamiota, Assiniboine Basin Initiative • Stuart Olmstead, Town of Carberry • Tina Williams, Town of Virden
Portage la Prairie Herman Prior Centre December 10, 2014	• Estelle Thornson, RM of Cartier • Dale Fossay, RM of Cartier • Ken Beaudin, RM of Cartier • Virginia Beckwith, RM of Cartier • Jason Tanguay, RM of Cartier • Andrew Peters, RM of Cartier • Tim deDelley, RM of Cartier • Laura Rogers, RM of Cartier • Rick Plaisor, AMM and RM of Sifton • Kevin Blight, RM of Portage la Prairie • Larry Gibbs, RM of Portage la Prairie • Kam Blight, RM of Portage la Prairie • Daryl Hrehirchuk, RM of Portage la Prairie • Dale Wood, RM of Portage la Prairie • Roy Tufford, RM of Portage la Prairie • Irvine Ferris, City of Portage la Prairie. • Trevor King, RM of Woodlands

3.3.2 Public Open Houses

December 8-10, 2014, 5:00 pm - 8:00 pm

Following meetings with local leadership, public open house events were held to provide opportunities for the general public to learn about the study. These open house events also allowed the public to share their own experience and local knowledge of recent flood events. Study team members were present to walk attendees through the storyboards, provide answers to questions, and receive input to the study. Participants were encouraged to fill out comment forms. Total attendance for the Round 2 public open houses was approximately 390 people.

Copies of the open house storyboards were made available at each of the events to individuals who requested them. During the Portage La Prairie Open House, demand for these copies exceeded supply and additional copies were produced at the event for attendees. As with Round One, copies of the open house materials and comment forms were provided directly on the project website to allow interested individuals and groups the ability to access project information directly (http://www.gov.mb.ca/mit/floodinfo/floodproofing/reports).

During each of the events, summary notes of individual discussions and questions were maintained by staff from MIT, KGS and InterGroup as they occurred. These summary notes were ultimately incorporated into the documentation of the summary report.

Attendance at the leadership and open house sessions was higher than in Round One. There was considerably more feedback provided directly during the events and with comment form submissions. In total, 122 comment forms and submissions were received. Table 3-2 summarizes the number of people who attended the different events along with other information about their participation.

Table 3-2: Round 2 Event Participation

	Dauphin Dec 8, 2014	Brandon Dec 9, 2014	Portage la Prairie Dec 10, 2014	Mailed/ Emailed Comments	Total
Leadership Briefing	11	10	17		38
Public Open House (note 1)	34	70	287		391
Number of Comment Forms	17	19	37	49	122

Note: Public Open House attendance figures reflect individuals who completed the sign-in sheet or were observed participating at each event. At least four individuals in Brandon and five individuals in Portage la Prairie did not complete the sign-in sheet but participated in the event.

3.4 SUMMARY OF ROUND 2 PUBLIC CONSULTATION INTERESTS AND CONCERNS

Stakeholders came to the Leadership and Open House sessions in Dauphin, Brandon and Portage la Prairie to share their personal perspectives and to discuss the different options presented by the Study Team. Many common themes emerged through these discussions. Despite frustration with the current flooding situation for many regional participants, the majority of people noted appreciation of the effort that had been made to share the information. The presence and direct participation of senior MIT staff enhanced the credibility of the process and the discussions that took place and many participants commented that they were pleased MIT was so well represented. Table 3-3 provides a summary of the comment forms received during Round 2.

Table 3-3: Comment Form Profile Information

	Dauphin Dec 8, 2014	Brandon Dec 9, 2014	Portage la Prairie Dec 10, 2014	Mailed/ Emailed Comments	Total
Number of Comment Forms	17	19	37	49	122
Participants identified connection to the process					
Homeowner	11	7	21	20	59
Cottage Owner	4	1	10	10	25
Farmer/Rancher	5	6	11	20	42
Business Owner	3	2	3	3	11
First Nation Leader	1				1
Municipal Elected Official	5	6	2	1	14
Municipal Employee		2		1	3
Provincial Employee	1				1
Other	1	3	8	6	18
How participant heard about event					
Newspaper	8	3	10	15	36
Radio/Public Service Announcement	3	4	7	7	21
Letter Invitation	4	5	3	1	13
Email	2	3	6	4	15
Word of Mouth	2	4	12	9	27
Other	1		5	3	9
Was the information/discussion helpful?					
	Yes 15 No 2	Yes 18 No 2	Yes 24 No 10	Yes 23 No 9	Yes 80 No 23

In addition to comment forms, the Study Team also received numerous formal submissions from groups and individuals. These submissions provided additional perspectives about the information that had been shared at the Open House as well as discussed broader areas of concern and interest. Submissions were received from a number of individuals and organizations such as the Manitoba Beef Producers and the Lake Winnipeg Foundation. The rural municipalities of Cartier, St Francis Xavier and Headingly each provided additional information as well as forwarded submissions they had received directly from constituents. Copies of both the submitted open house comment forms and received submissions were provided confidentially to MIT to for use as a reference in future planning but have not been appended to this public report.

Following the Portage la Prairie Open House event, the RMs of Cartier and St. Francis-Xavier requested and met with MIT and Study Team representatives to discuss proposed mitigation measures and to gain a better understanding to the implications to the municipalities if flows were increased to 23,100 cfs.

While stakeholders shared many different perspectives on the study and many common themes emerged throughout the discussions. Stakeholders interested in learning about and sharing their perspectives about the mitigation options being presented. Section 3.4.1 provides a summary of comments organized into general themes during Round 2, including an indication of the frequency that these comments were identified.

3.4.1 Round 2 Comment Summary

Throughout the engagement process, stakeholders and the general public provided many perspectives on the information that was shared during the events. Discussion, comments and questions raised throughout the process indicated a strong interest in learning about and sharing perspectives about the presented mitigation options, the study process, and the anticipated process in the future. From review of the comment forms and submissions, summary notes of discussions recorded by Study Team staff, comments were consolidated and categorized into a structure that could support the further assessment of mitigation alternatives.

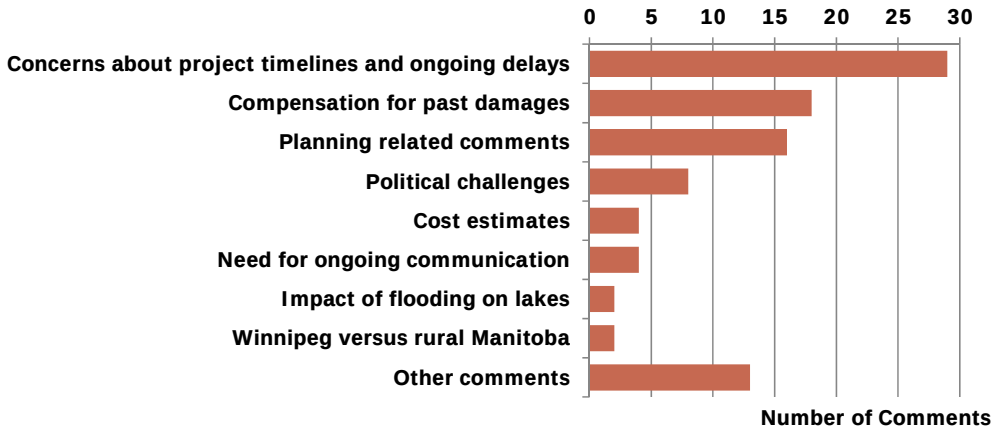
The following sections summarize this information and have been organized in the following manner:

- **General Comments:** Comments and perspectives not specific to particular regions or mitigation alternatives but focused on the overall study.
- **Regional Comments:** Information was presented at the engagement events by region. Many of the received comments reflected the perspective of an individual residing from that region or reflected a mitigation alternative specific to a region. Some mitigation alternative comments could not be attributed to a specific region or initiative and so are categorized as "other flood mitigation options".
    - Upper Assiniboine
    - Lower Assiniboine
    - Lake Manitoba & Lake St. Martin

- Dauphin and Shoal Lake
- Other Flood Mitigation Options

- **Public Consultation Process Comments**: Participants were asked to provide any comments on the overall consultation process.

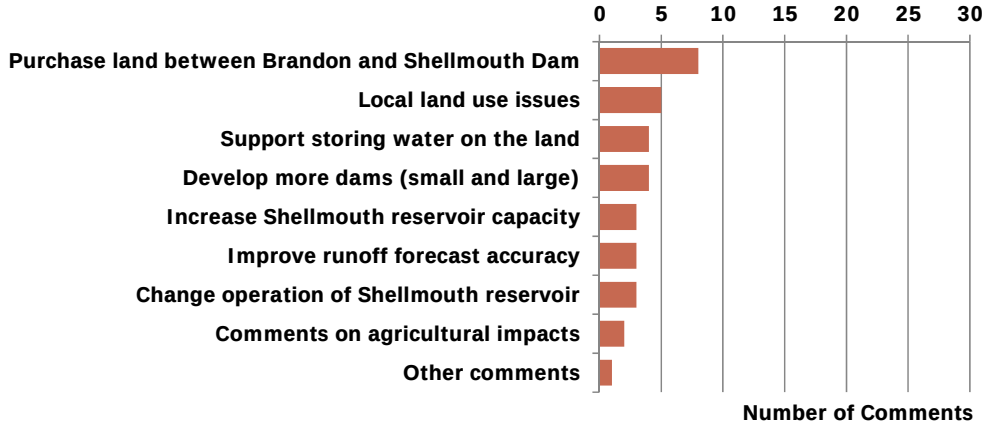
Figure 3-1: Round 2 – Summary General Comments



The largest number of comments recorded in each of the events related to concerns related to project timelines and an interest in having tangible long term solutions in place as soon as possible. Comments were also received identifying or seeking clarification on compensation related to past flooding damages, relfecting that some participants continue to be adversely impacted by previous flood events.

Comments that were related to planning focused on the need to either integrate implementation of mitigation options with other regional/provincial interests and objectives or consider other perspectives.

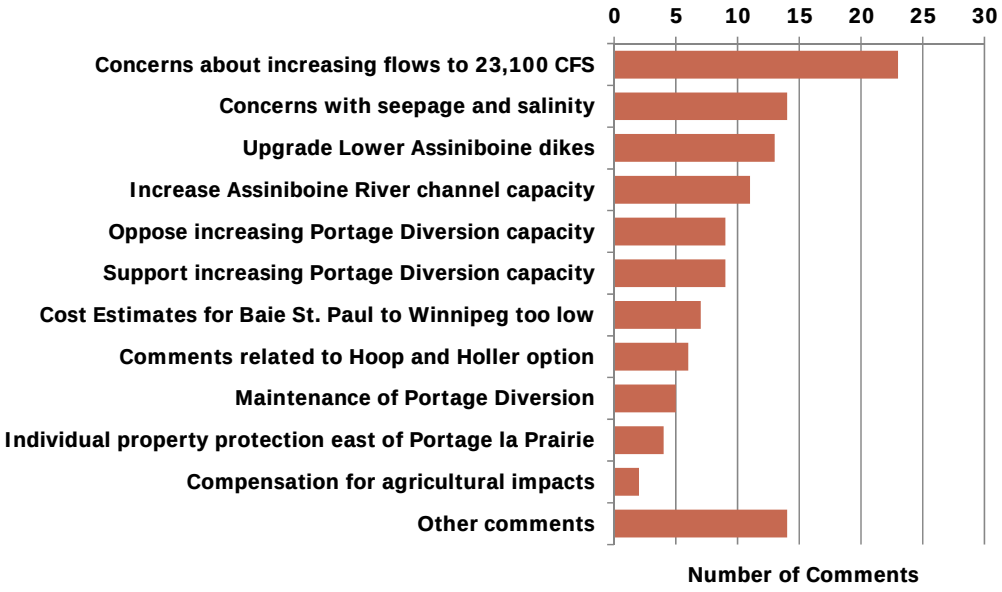
Figure 3-2: Round 2 – Upper Assiniboine Basin Option Comments



Comments that were received on mitigation options within the Upper Assiniboine River basin focused on use of land for temporary storage during flood events and, to a lesser degree, operation of the

Shellmouth Dam. Respondents suggested mitigation of flood events could best be supported through the public purchase of land, securing easement to use land, providing tax incentives for use of land, or working with other groups (Ducks Unlimited) to develop watershed management land. Comments related to the Shellmouth Dam focused on increasing the reservoir capacity, changing the operation, and actions that enhance the operation of the facility (increased forecasting accuracy, use of Shellmouth Dam Compensation Regulation).

Figure 3-3: Round 2 – Lower Assiniboine Option Comments

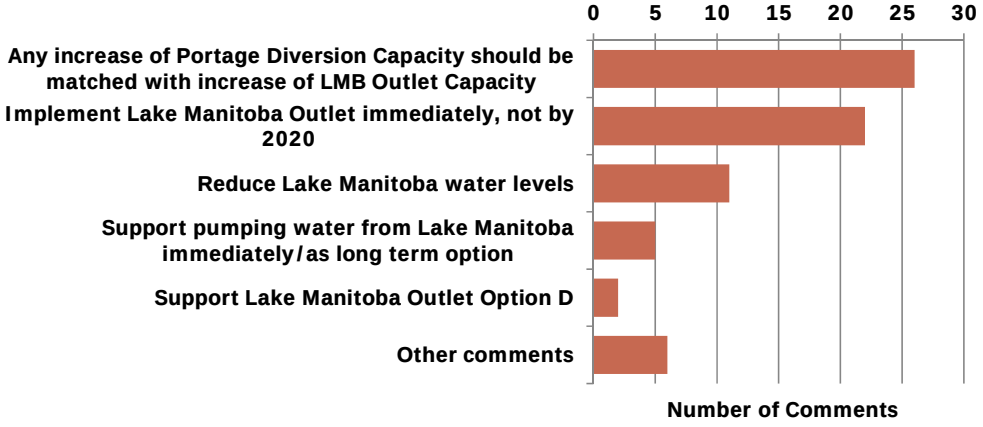


The greatest diversity of comments received during Round 2 was related to flood mitigation options within the Lower Assiniboine River area. This region includes two substantive flood mitigation alternatives (options that increased flows within the Lower Assiniboine River below Portage la Prairie and options that increased flows through the Portage Diversion to Lake Manitoba). Attendance at the Portage la Prairie open house was also the greatest of all events and reflected in the total number of comment forms that were received.

The majority of comments focused on potential increased flows to the Lower Assiniboine River: concerns that these flows (23,100 cfs) could not be safely achieved based on recent experience, dike upgrades should proceed regardless of the whether or not flows are ultimately increased, or the physical river channel should be improved to carry higher flows using only the existing dike system. Other identified concerns referenced the potential for increased basement seepage and salinity with higher water levels.

Comments received either supporting or opposing changes to the capacity of Portage Diversion were divided. Individuals who did provide comments on the Portage Diversion were more likely also to comment that there was a need to balance any increased Portage Diversion capacity with increased Lake Manitoba Outlet capacity (see Figure 3-4).

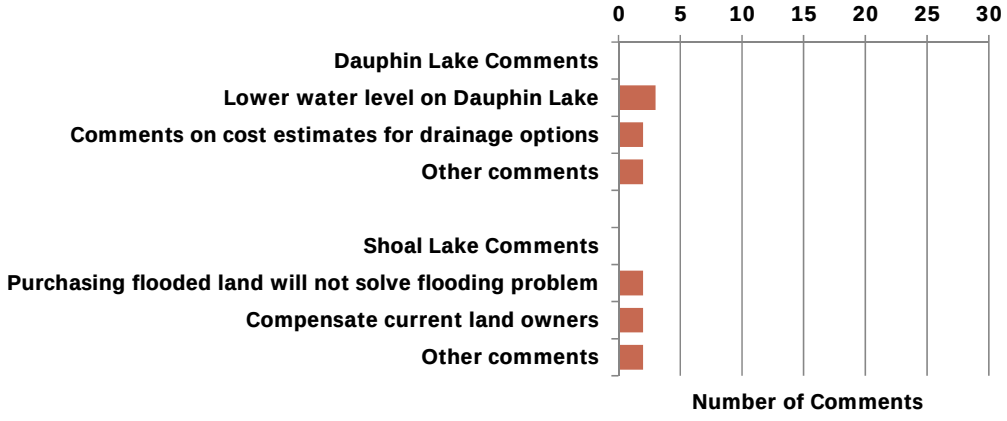
Figure 3-4: Round 2 – Lake Manitoba & Lake St. Martin Option Comments



Comments about the Lake Manitoba and Lake St. Martin options reflected the interest of participants with the Portage Diversion, water levels on Lake Manitoba, and options for enhancing outlet channel capacity between Lake Manitoba and Lake Winnipeg. Participants acknowledged the an earlier consultation process (Fall 2014) had focused specifically on Lake Manitoba and Lake St. Martin Outlet Channel options but for some participants this was their first opportunity to review the information.

Most comments urged immediate action to reduce net Lake Manitoba water levels, either through expediting the environmental approval process and development of Lake Manitoba outlet channels or immediate pumping of water from Lake Manitoba. Respondents did not want water levels on either Lake Manitoba or Lake St. Martin exacerbated by increased Portage Diversion flow contributions unless there was at least an equivalent increase in capacity with Lake Manitoba/Lake St. Martin outlet channels.

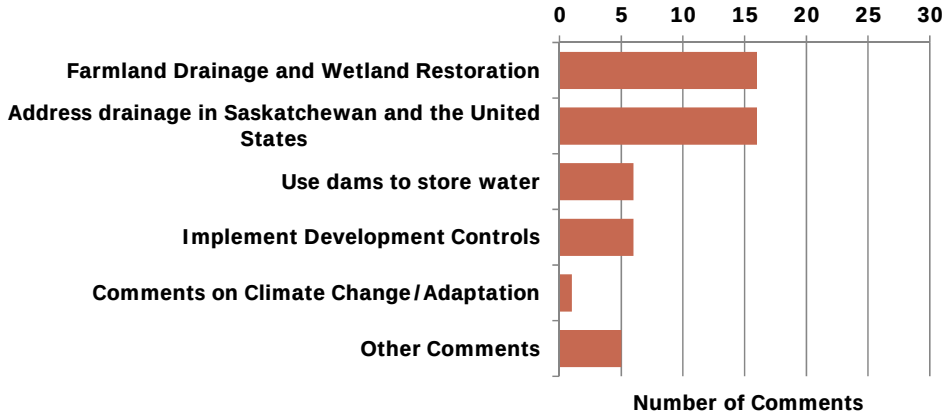
Figure 3-5: Round 2 – Dauphin and Shoal Lake Option Comments



Dauphin Lake comments identified the need to improve drainage capacity of the lake but also identified the decrease volume available for storage because of increased siltation in the lake over the past 50 years. Other comments included the need to revisit cost estimates in light of previous reports that suggested effective drainage of Dauphin Lake could be achieved at a lower cost.

Shoal Lake comments reflected discussions and input received from adjacent landowners. Some comments reflected concerns about landowner compensation and relief and other comments identified a need to develop a viable solution that could support sustainable ranching in the region.

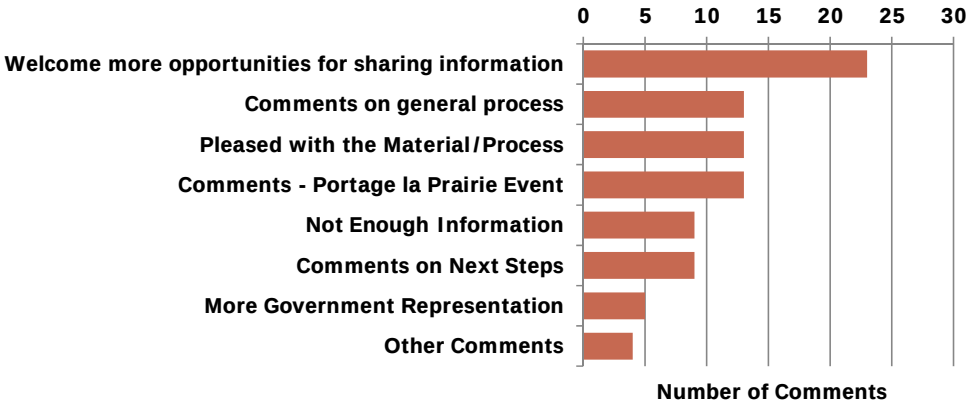
Figure 3-6: Round 2 – Other Flood Mitigation Option Comments



Throughout the consultation process, participants offered perspectives and comments on many flood mitigation options. Some options were potentially applicable to all study regions. The topic of farmland drainage management and wetland restoration was raised throughout the larger study area but received greatest attention and discussion at both the Dauphin and Brandon open houses. Another comment heard repeatedly was the need to address drainage at a watershed basin level and to encourage collaboration on drainage management with both Saskatchewan and the U.S. Border States.

Other comments urged consideration of the use of smaller dams throughout the region to retain water during flood events and provide a water supply during drought events to support agriculture. Individuals commenting on implementation of development controls encouraged limiting urban development within Designated Flood Areas (DFAs) to reduce the likelihood of future flooding and the associated effects.

Figure 3-7: Round 2 – Public Consultation Process Comments



In addition to comments provided on the various flood mitigation options, participants were encouraged to provide comments on the general process, the information that was shared, and their usefulness of both to addressing issues of concern they may have had.

The engagement process was generally well received and both elected officials who participated in the leadership briefing sessions and the general public who participated in the open house events noted an interest in having more opportunities to share information and perspectives. Suggestions included workshops to review and discuss topics in more detail. Despite the advertising and notification that did occur, many commented that increased participation may have occurred had there been more extensive and earlier notification.

Individuals who expressed concern about the lack of information focused mostly on the lack of individual property-specific flooding and compensation information available at the open house. Participants at the Portage la Prairie open house commented that the number of people present made it difficult to find opportunities to speak with consultants and government staff and suggested a larger venue with more resources might have been better.

APPENDIX A
ROUND 2 PUBLIC CONSULTATION

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APPENDIX A1

ROUND 2 STAKEHOLDERS MEDIA PLAN

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List of Key Stakeholders Invited to AR & LMB Rd 2 Information Session

Invitations were distributed November 21, 2014

Title	First Name	Last Name	Organization
Chief	Nelson	Bunn	Birdtail Sioux First Nation
Mayor	Edward	Anderson	Boissevain-Morton Municipality
Reeve	Gary	Williams	Brenda-Waskada, Municipality
Chief	Delbert	Cruise	Canupawakpa Dakota First Nation
Chief	Clarence	Easter	Chemawawin Cree Nation
Mayor	Rick	Chrest	City of Brandon
Mayor	Shari	Hirst	City of Brandon
Mayor	Eric	Irwin	City of Dauphin
Mayor	Irvine	Ferris	City of Portage la Prairie
Mayor	Earl	Porter	City of Portage la Prairie
Mayor	Brian	Bowman	City of Winnipeg
Head of Council	Elgin	Hall	Clanwilliam-Erikson, Municipality
Chief	Orville	Smoke	Dakota Plains First Nation
Chief	David	Pashe	Dakota Tipi First Nation
Mayor	Einarsson	Helgi	Dauphin River (Community of)
Chief	John	Stagg	Dauphin River First Nation
Head of Council	Gordon	Weidenhamer	Deloraine-Winchester Municipality
Chief	Nelson	Houle	Ebb & Flow First Nation
Reeve	Barry	Lowes	Ellice-Archie, Municipality
Reeve	Art	Potoroka	Ethelbert, Municipality
Chief	David	Crate	Fisher River First Nation
Chief	David	Ledoux	Gamblers First Nation
Head of Council	James	Michaluk	Gilbert Plains Municipality
Head of Council	Earl	Malyon	Glenboro-South Cypress Municipality
Reeve	Richard	Funk	Glenella-Lansdowne, Municipality
Head of Council	Lyle	Morran	Grandview, Municipality
Reeve	Blair	Woods	Grasslands Municipality
Reeve	Larry	Oakden	Hamiota Municipality
Reeve	Lloyd	Ewashiko	Harrison Park Municipality
Councillor	Betty	Nykyforak	Hillsburg-Roblin-Shell River, Municipality
Head of Council	Wadw	Schott	Hillsburg-Roblin-Shell River, Municipality
Chief	James	Plewak	Keeseekoowenin Ojibway Nation
Chief	Cornell	McLean	Lake Manitoba First Nation
Chief	Adrian	Sinclair	Lake St. Martin First Nation
Chief	Hector	Shorting	Little Saskatchewan First Nation
Chief	David	Meeches	Long Plain First Nation
Reeve	Aurel	Pantel	Lorne, Municipality
President	David	Chartrand	Manitoba Metis Federation
Head of Council	Larry	McLauchlan	McCreary, Municipality
Reeve	Clinton	Eisner	Minitonas-Bowsman Municipality
Reeve	Craig	Spencer	Norfolk-Treherne Municipality
Reeve	Bob	Adriaansen	North Cypress-Langford Municipality
Head of Council	Neil	Christoffersen	North Norfolk Municipality
Reeve	David	Inkster	Oakland-Wawanesa Municipality
Chief	Eugene	Eastman	O-Chi-Chak-Ko-Sipi First Nation
Chief	Glenn	Husdon	Peguis First Nation
Reeve	Glenn	Shiskoski	Pembina Municipality
Chief	Garnet	Woodhouse	Pinaymootang First Nation
Chief	Charlie	Boucher	Pine Creek Anishinabe Nation
Reeve	Linda	Clark	Prairie View Municipality

Title	First Name	Last Name	Organization
Head of Council	Todd	Gill	Riverdale Municipality
Reeve	Tom	Campbell	RM of Albert
Reeve	Stan	Asham	RM of Alonsa
Reeve	Wendy	Davidson	RM of Archie
Reeve	Bob	Conibear	RM of Argyle
Reeve	Daniel	Martens	RM of Argyle
Reeve	Jack	Cruise	RM of Armstrong
Reeve	Garry	Wasylowski	RM of Armstrong
Reeve	James	Trewin	RM of Arthur
Reeve	Duncan	Stewart	RM of Brenda
Reeve	Wayne	Drummond	RM of Cameron
Reeve	Dale	Fossay	RM of Cartier
Reeve	Roland	Rasmussen	RM of Cartier
Reeve	Victor	Baraniuk	RM of Clanwilliam
Reeve	Brian	Sigfusson	RM of Coldwell
Reeve	Reg	Atkinson	RM of Conwallis
Reeve	Heather	Dalgleish	RM of Cornwallis
Reeve	Evan	Smith	RM of Daly
Reeve	Dennis	Forbes	RM of Dauphin
Reeve	Ralph	Wang	RM of Edward
Reeve	Guy	Huberdeau	RM of Ellice
Reeve	James	Boyd	RM of Elton
Reeve	Farley	Ross	RM of Elton
Reeve	Arne	Lindell	RM of Eriksdale
Reeve	Richard	Hyde	RM of Fisher
Reeve	Shannon	Pyziak	RM of Fisher
CAO	Susan	Boyachek	RM of Gilbert Plains
Reeve	Derek	Klassen	RM of Glenella
Reeve	M.E.	Sanderson	RM of Glenwood
Reeve	Clifford	Halaburda	RM of Grahamdale
Reeve	Diane	Price	RM of Grahamdale
Reeve	Cliff	Kutzan	RM of Grandview
Reeve	Raymond	Franzmann	RM of Grey
Reeve	Randy	Lints	RM of Hamiota
Reeve	Murray	Davies	RM of Harrison
Reeve	Wilfred	Taillieu	RM of Headingly
Reeve	Robert	Misko	RM of Hillsburg
Reeve	Clinton	Cleave	RM of Lakeshore
Reeve	Kathy	Jasienczyk	RM of Langford
Reeve	Fred	Taylor	RM of Lawrence
Reeve	Olive	McKean	RM of Miniota
Reeve	Michael	McIntosh	RM of Minitonas
Reeve	Calvin	Jacobson	RM of Minto
Reeve	James	Andersen	RM of Minto-Odanah
Reeve	Robert	McCallum	RM of Morton
Reeve	Kate	Basford	RM of Mossey River
Reeve	Larry	Zurba	RM of Mossey River
Reeve	Robert	Hanson	RM of Mountain
Reeve	Marvin	Kovachik	RM of Mountain
Reeve	Ralph	Oliver	RM of North Cypress
Reeve	Brent	Fortune	RM of Oakview
Reeve	Craig	Atkinson	RM of Park
Reeve	Kim	Taylor	RM of Pembina
Reeve	Archie	McPherson	RM of Pipestone

Title	First Name	Last Name	Organization
Reeve	Ross	Tycoles	RM of Pipestone
Reeve	Kam	Blight	RM of Portage la Prairie
Reeve	Lonn	Dunlop	RM of Prairie Lakes
Reeve	Barry	Chescu	RM of Riding Mountain West
Reeve	Jim	Campbell	RM of Rockwood
Reeve	Edward	Levandoski	RM of Rosedale
Reeve	Bill	Martin	RM of Rosedale
Reeve	Edward	Mychasiw	RM of Rossburn
Reeve	Frances	Smee	RM of Rosser
Reeve	Robert	Muir	RM of Russell
Reeve	Robert	Sharpe	RM of Saskatchewan
Reeve	Alvin	Zimmer	RM of Shellmouth-Boulton
Reeve	Rick	Plaisier	RM of Sifton
Reeve	Barry	Zacharias	RM of Siglunes
Reeve	Fred	Dunn	RM of Silver Creek
Reeve	Dwayne	Clarke	RM of St. Francois Xavier
Reeve	Roger	Poitras	RM of St. Francois Xavier
Reeve	Cheryl	Smith	RM of St. Laurent
Reeve	Earl	Zotter	RM of St. Laurent
Reeve	Maurice	Maguet	RM of Ste. Rose
Reeve	Ken	Wozney	RM of Strathclair
Reeve	Dennis	Schram	RM of Strathcona
Reeve	Lorne	Henkelman	RM of Swan River
Reeve	Harold	Purkess	RM of Victoria
Reeve	Don	Neufeld	RM of Wallace
Reeve	Denis	Carter	RM of Wallace-Woodworth
Head of Council	Bruce	Henley	RM of West St. Paul
Reeve	Heather	Curle	RM of Whitehead
Reeve	Wayne	Dobbie	RM of Whitehead
Reeve	Michael	Dillabough	RM of Winchester
Reeve	Trevor	King	RM of Woodlands
Reeve	Donald	Walsh	RM of Woodlands
Reeve	Donald	Yanick	RM of Yellowhead
Chief	Morris	Swan-Shannacapp	Rolling River Anishinabe Nation
Councillor	Alfred	Hayden	Roseau River Anishinabe First Nation
Head of Council	Brain	Brown	Rossburn Municipality
Mayor	Len	Derkach	Russell-Binscarth Municipality
Chief	Russell	Beaulieu	Sandy Bay First Nation
Chief	Nelson	Genaille	Sapotaweyak Cree Nation
Chief	Vincent	Tacon	Sioux Valley Dakota Nation
Chief	Cameron	Catcheway	Skownan First Nation
Head of Council	Darryl	Jackson	Souris-Glenwood Municipality
Mayor	Robert	Brunel	St. Rose Municipality
Chief	Francine	Meeches	Swan Lake First Nation
Head of Council	Verne	Scouten	Swan Valley West Municipality
Chief	Barry	McKay	Tootinaowaziibeeng Treaty Reserve
Mayor	Dwight	Stewart	Town of Birtle
Councillor	Roger	Wilson	Town of Birtle - Prairie View Municipality
Mayor	Wayne	Blair	Town of Carberry
Mayor	Stuart	Olmstead	Town of Carberry
Mayor	Brian	Franklin	Town of Deloraine
Mayor	Val	Soltys	Town of Erickson
Mayor	Eileen	Clarke	Town of Gladstone
Mayor	Tom	Bohun	Town of Grandview

Title	First Name	Last Name	Organization
Mayor	Lori	Taylor	Town of Hartney
Mayor	Bob	Walker	Town of Melita
Mayor	Henry	Barkowski	Town of Minitonas
Mayor	Ray	Orr	Town of Minnedosa
Mayor	Adrian	DeGroot	Town of Neepawa
Mayor	Ken	Waddell	Town of Neepawa
Mayor	Jeff	Sigurdson	Town of Oak Lake
LUD	Orest	Woloski	Town of Rapid City
Mayor	Shirley	Kalyniuk	Town of Rossburn
Mayor	Christopher	Radford	Town of Russell
Mayor	Glen	McKenzie	Town of Swan River
Mayor	Jeff	McConnell	Town of Virden
Reeve	Debbie	McMechan	Two Borders Municipality
Mayor	Marion	Meadows	Village of Benito
Mayor	Dale	Sawchuk	Village of Binscarth
Mayor	Leanne	Hutman	Village of Bowsman
Mayor	Roland	Gagnon	Village of Elkhorn
Mayor	Mitch	Michaluk	Village of Ethelbert
Mayor	Lyle	Smith	Village of Gilbert Plains
Mayor	Robert	Jewsbury	Village of Glenboro
Mayor	William	Wiebe	Village of MacGregor
Councillor	Linda	Cripps	Village of McCreary - McCreary Municipality
Mayor	Gilbert	Mabon	Village of Somerset
Mayor	Rene	Maillard	Village of Ste. Rose du Lac
Mayor	Martin	Dupont	Village of St-Lazare
LUD	Laurent	Fouillard	Village of St-Lazare - Elice Archie Municipality
Mayor	Bruce	Gullet	Village of Wawanesa
Mayor	Dan	Brown	Village of Winnipegosis
Chief	Mel	Wabash	Waywayseecappo First Nation
Reeve	Randy	Helgason	West Interlake Municipality
Head of Council	David	Single	Westlake-Gladstone Municipality
Councillor	Philip	Thordarson	Westlake-Gladstone Municipality
Chief	Albina	Brass	Wuskwi Sipihk First Nation (Indian Birch)

Manitoba
Infrastructure and Transportation
Hydrologic Forecasting and Water Management
Water Management and Structures Division
Box 11, 200 Saulteaux Crescent
Winnipeg, Manitoba, R3J 3W3
T 204-945-7488 **F** 204-945-7419

November 21, 2014

[Name]
[Address]
[City], Manitoba [Postal Code]

Dear [Name]:

RE: ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY ROUND 2 OPEN HOUSE

The Manitoba government is holding a second series of open houses where members of the public can learn more about the Assiniboine River and Lake Manitoba Basins Flood Mitigation Study. The study has identified flood vulnerabilities and examined potential mitigation options for communities including potential improvements to existing infrastructure. Three open house events are planned during December within the study area. These open house events will communicate the results of the study and offer opportunities for people to ask questions and provide feedback on the information presented.

We are inviting the leadership of the local communities within the study area to an information session prior to the start of the open house event. The session will be an opportunity for the local government leadership to receive the information that will be made available to the general public at the open houses, which are scheduled at 5:00 PM after the information sessions. We recognize that this is a time of transition for many towns, villages and municipalities due to the municipal amalgamation process. Given that some leaders were elected on October 22, 2014 and will not take office until January 1, 2015, we are inviting the acting leadership and the transitional group or a representative to the information sessions.

The information sessions are scheduled for:

- Dauphin: December 8, 2014, 1:00-3:00 PM
  - Parklands Recreation Complex – Lion's Den, 200 1st Street South East
- Brandon: December 9, 2014, 1:00-3:00 PM
  - Keystone Centre, Salon A, 1175 18th Street
- Portage La Prairie: December 10, 2014, 1:00-3:00 PM
  - Herman Prior Seniors Service Centre, 40 Royal Road North

I have included with this letter a map of the broader study area along with an invitation to the open house event that can be posted in your administration offices. If you have any questions or comments about the study or information session, please feel free to contact me at 204-945-7488 or by email at steve.topping@gov.mb.ca.

Yours truly,

<original signed by>

Steve D. Topping, P. Eng.
Executive Director
Hydrologic Forecasting and Water Management

Manitoba
Infrastructure and Transportation

Steven D. Topping, P. Eng.
Executive Director
Hydrologic Forecasting and Water Management
Water Management and Structures Division
Box 11, 200 Saulteaux Crescent
Winnipeg, Manitoba, R3J 3W3
T 204-945-7488 **F** 204-945-7419

November 21, 2014

To Whom It May Concern,

RE: ASSINIBOINE RIVER & LAKE MANITOBA BASINS FLOOD MITIGATION STUDY ROUND 2 OPEN HOUSE

The Manitoba government is holding a second series of open houses where you can learn more about the Assiniboine River and Lake Manitoba Basins Flood Mitigation Study. The study has identified flood vulnerabilities and examined potential mitigation options for communities including potential improvements to existing infrastructure.

This study includes a public engagement process designed to share and receive information and perspectives on potential flood mitigation measures. Three open house events are planned during December within the study area. These open house events will communicate the results of the study and offer opportunities for people to ask questions and provide feedback on the information presented.

As a stakeholder with possible interest in the project, we would like to personally invite you to either the open house or to respond to material that will be posted on the MIT website at the start of the open houses (www.gov.mb.ca/mit/floodinfo/floodproofing/reports/index.html).

Three communities were identified as suitable locations for open house meetings given their location in the Assiniboine River and Lake Manitoba Basins study area.

The public open houses are scheduled for:

- Dauphin: December 8, 2014, 5:00-8:00 PM
  - Parklands Recreation Complex – Lion's Den, 200 1st Street South East
- Brandon: December 9, 2014, 5:00-8:00 PM
  - Keystone Centre, Salon A, 1175 18th Street
- Portage La Prairie: December 10, 2014, 5:00-8:00 PM
  - Herman Prior Seniors Service Centre, 40 Royal Road North.

I have included with this letter a map of the broader study area for your information, along with an invitation to the open house event that can be posted in your community.

If you have any questions or comments about the study or information session, please feel free to contact me at 204-945-7488 or email at steve.topping@gov.mb.ca.

Yours truly,
<original signed by>

Steve D. Topping, P. Eng.
Executive Director
Hydrologic Forecasting and Water Management

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8.5"x11" PLOT SCALE 1:1

Manitoba
Infrastructure and Transportation

Public Open Houses

Share Your Views on the
Assiniboine River and Lake Manitoba
Basin Flood Mitigation Study

The Manitoba government is holding a second series of open houses where you can learn more about the Assiniboine River and Lake Manitoba Basins Flood Mitigation Study. The study has identified flood vulnerabilities and examined potential mitigation options for communities including potential improvements to existing infrastructure.

Three public open houses have been organized to give you an opportunity to learn about, and provide your input on, the flood protection options that have been studied for reducing flood risk in the region. Round 2 of the engagement process will communicate the results of the study and offer opportunities for people to ask questions and provide feedback on the information presented

You are encouraged to attend the open house closest to you and to make yourself heard on this important issue. The material will be posted on the MIT website at the start of the open houses (http://www.gov.mb.ca/mit/floodinfo/floodproofing/reports/index.html).

The public open houses are scheduled for:

Dauphin: December 8, 2014, 5:00-8:00 PM

Parklands Recreation Complex – Lion's Den, 200 1st Street South East

Brandon: December 9, 2014, 5:00-8:00 PM

Keystone Centre, Salon A, 1175 18th Street

Portage La Prairie: December 10, 2014, 5:00-8:00 PM

Herman Prior Seniors Service Centre, 40 Royal Road North

Refreshments will be served. Everyone is welcome to attend.

For more information please contact:

Colin Siepman, P. Eng.
Senior Project Manager, KGS
Phone: 204-896-1209
Email: CSiepman@kgsgroup.com

APPENDIX A2

ROUND 2 PUBLIC CONSULTATION MATERIALS

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Manitoba

ASSINIBOINE RIVER AND LAKE MANITOBA BASINS FLOOD MITIGATION STUDY

Round 2 Open House

Manitoba

WELCOME!

Manitoba Infrastructure and Transportation is studying flooding in the Assiniboine River and Lake Manitoba watersheds and how we can provide greater protection from flooding risks.

Round 1 Open Houses were held in June 2013 in Dauphin, Brandon and Portage la Prairie to introduce the study terms of reference.

The study has helped the province better understand the flood risks and the possible options that can be used to reduce effects from future flood events. The study focused on:

- The main stems of the Assiniboine River and Souris River
- Lake Manitoba
- Lake St. Martin
- Dauphin Lake
- Shoal Lakes

Assiniboine River Dikes
(May 2011)

Fairford River Water Control Structure
(July 2011)

Lake Manitoba
(July 2011)

WE VALUE YOUR FEEDBACK!

The Round 2 Open House events communicate the results of the study.

Ask us any questions you may have.
Share your thoughts on the presented information.
Fill out a comment form.

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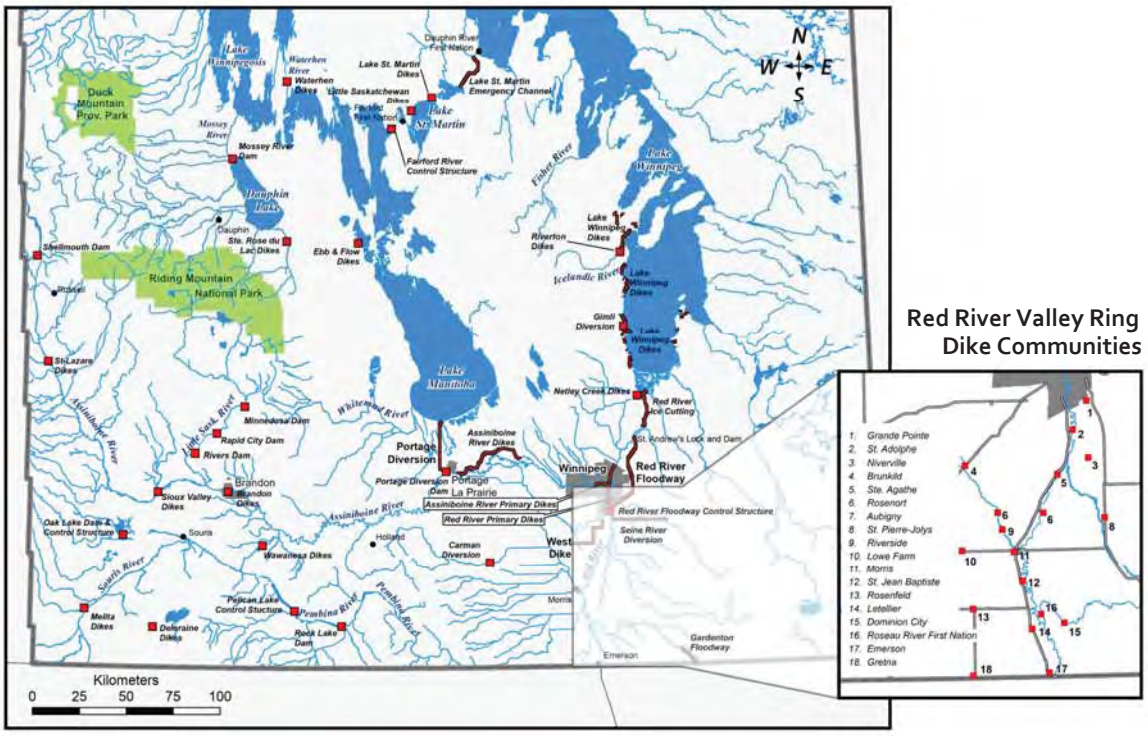
MANITOBA'S FLOOD PROTECTION MODEL

- Since the1950 flood, Manitoba has developed an extensive integrated flood protection system consisting of the following components:
  - Floodways and diversions
  - Dams
  - Community diking
  - Linear diking
  - Individual flood protection
  - Development controls
- This integrated flood mitigation strategy has worked well as a system since that time in the areas protected by the constructed works, and has avoided billions of dollars of damages.

Manitoba's Flood Control Infrastructure System



Manitoba

BACKGROUND

In the last hundred years, Manitoba has experienced several major floods. The flood of 2011 was unique. High flows were recorded on almost all streams and rivers in the Assiniboine River and Lake Manitoba watersheds. For a flood event like this to occur on one or two major rivers is rare, let alone for a flood to occur over all of western Manitoba.

The flood of 2011 highlighted several potential weak links in some of the existing flood control systems. The short comings of the system were emphasized again in the flood of 2014.

Assiniboine River

At least seven major floods have occurred on the Assiniboine River. The most recent of these events were in 2011 and 2014.

Lake Manitoba

Lake Manitoba reached record water levels in July 2011. In 2014, water levels exceeded flood stage for a second time in four years.

Souris River

In July 2011, record-high peaks occurred on the Souris River as a result of significant rain events in May and June.

Dauphin Lake

In 2011, Dauphin Lake began to rise in April due to snowmelt and reached record water levels in mid-June as a result of several major rain events.

Manitoba

STUDY OBJECTIVES

- The study has identified the flood risk, and assessed over 70 potential options to reduce flood risk for communities and major infrastructure along the following lakes and rivers.

  - Assiniboine River
  - Souris River
  - Qu'Appelle River
  - Fairford River
  - Dauphin River
  - Lake Manitoba
  - Lake St. Martin
  - Lake Winnipegosis
  - Dauphin Lake
  - Shoal Lakes

- A massive amount of data and technical input was reviewed as background to this study. In the determination of impacts and the development of a list of mitigation alternatives, a wide range of flood frequencies were considered.

- A number of flood protection works and flood reduction activities to reduce flood effects have been completed or are currently well underway. These works and activities were all considered in the study.

- Results from this study will be used to plan for and carry out future activities.

- More studies are needed to address flooding concerns along tributaries and in other areas that are not listed above.

Manitoba

RELATED PROVINCIAL INITIATIVES

- **The 2011 Manitoba Flood Review Task Force**
  - The review was completed in the spring of 2013 and provided many recommendations as a follow up to the 2011 flood.
  - The province accepted the 126 recommendations in April 2013.
- **Lake Manitoba and Lake St. Martin Regulation Review**
  - The review was completed in the spring of 2013 and considered issues with respect to Lake Manitoba and Lake St. Martin.
  - The province accepted the 17 recommendations in April 2013.
- **Surface Water Management Strategy**
  - Work is ongoing on this initiative and will support decision making for the future management of surface water in the province.
  - Proposes 50 actions to be implemented by 2020.
- **Review of operating guidelines for key provincial flood control infrastructure**
  - This review is ongoing and considers the Red River Floodway, Portage Diversion and Fairford River Water Control Structure.
  - The review will be completed in the spring of 2015.
- **Significant investments in flood mitigation**
  - Individual and Community flood protection province-wide including Lake Manitoba, Brandon, Souris, Melita, Reston, Wawanesa and other communities.
  - In the Province's Individual Flood Proofing Initiative, over 1,200 applicants have approved in the two basins, including 885 around Lake Manitoba.
  - Upgrades and rehabilitation to existing flood control infrastructure, including emergency investments in the provincial Assiniboine River dikes and the Portage Diversion.
- **Evaluation of many additional flood mitigation demands**
  - Recent flooding events across the province has increased the demand for additional flood mitigation measures.

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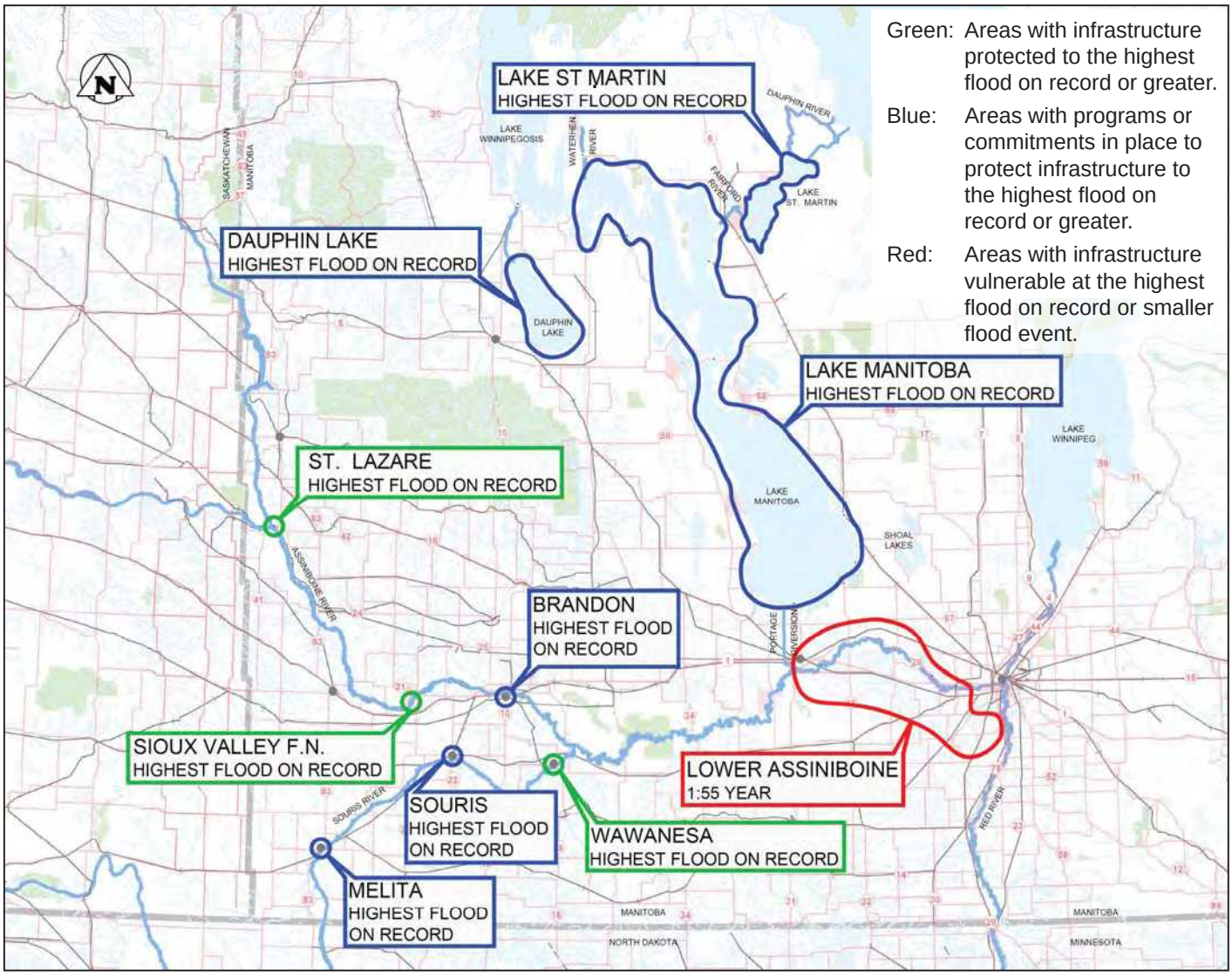
LEVEL OF PROTECTION

Assiniboine River and Lake Manitoba Basins

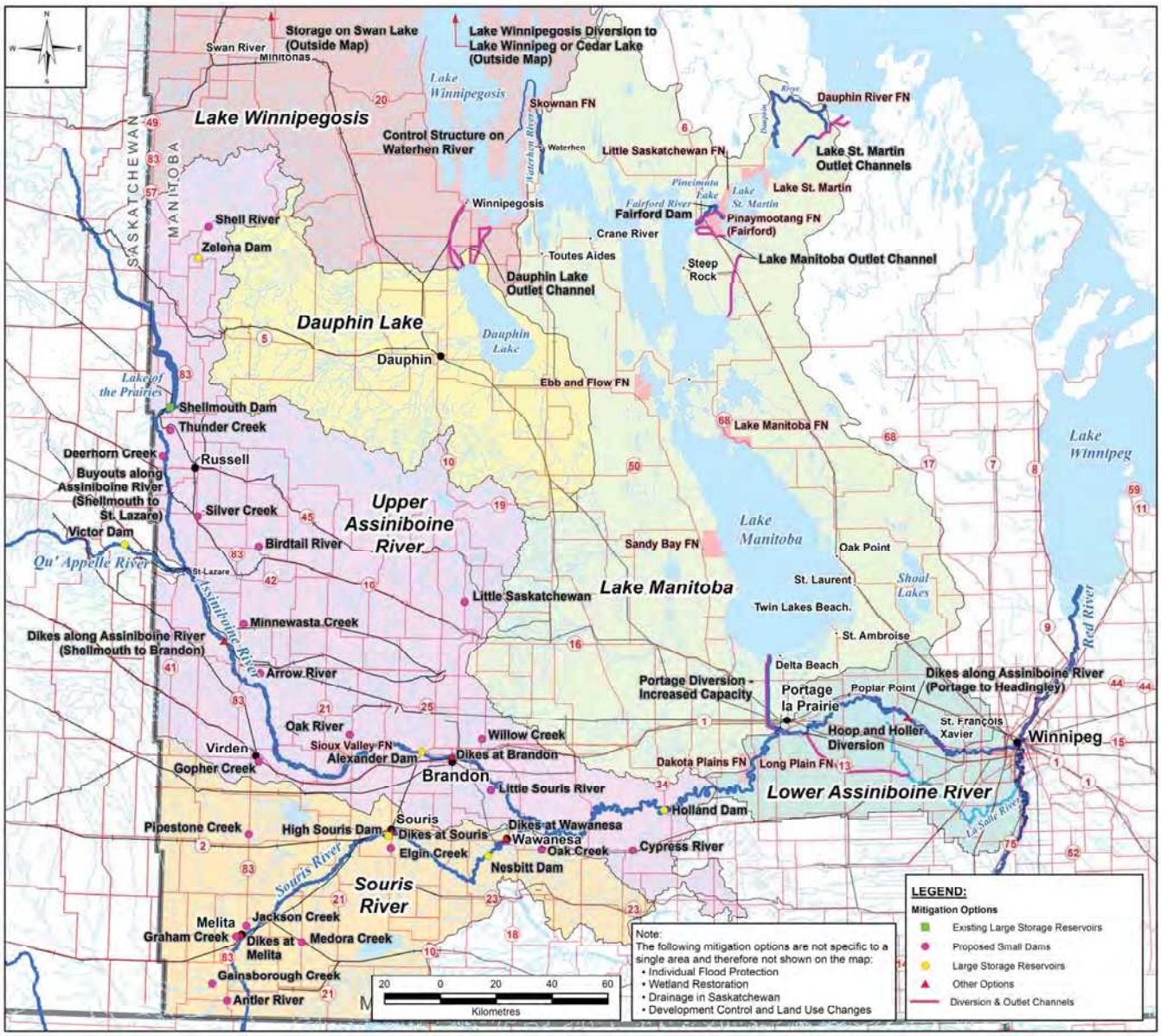


Note: In addition to the vulnerabilities shown on map, there are other vulnerabilities to individual properties and communities that may require further review.

Manitoba

OPTIONS FOR FLOOD MITIGATION

Study Areas and Flood Mitigation Alternatives



Manitoba

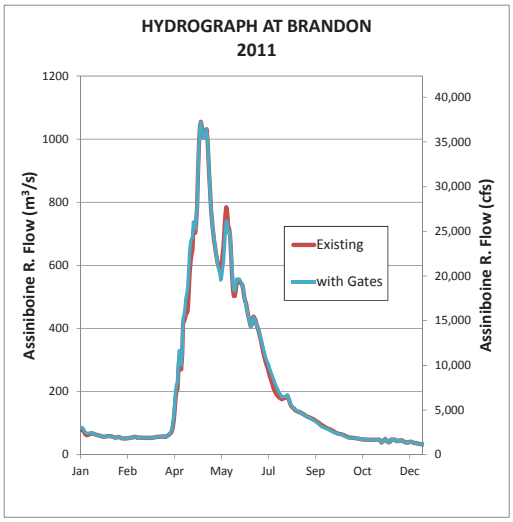
UPPER ASSINIBOINE RIVER

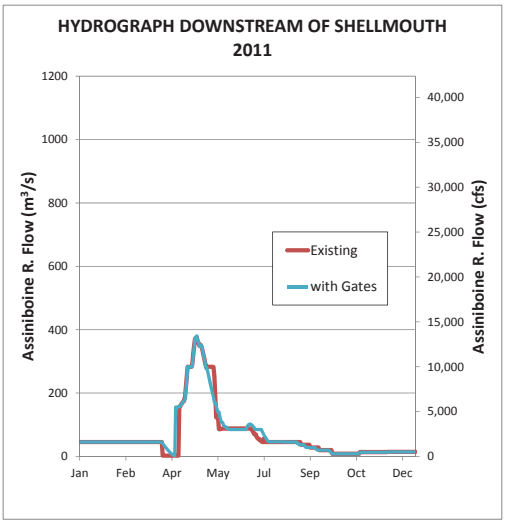
Impacts of Shellmouth Dam Gates on Floods

- The 2011 Flood Review Task Force recommended investigating alternative means to prevent or reduce flood damages on the Assiniboine River below Shellmouth Dam including:
  - Adding leaf gates on the Shellmouth Dam.
  - The purchase of flood prone lands.
  - Constructing dikes along the Assiniboine River.

Shellmouth Dam Leaf Gates

- The current study assessed the impacts of adding the proposed leaf gates on the Shellmouth Dam to water levels and flows during flood events.
- The assessment was based on the current operation guidelines.
- The results show a reduction in the frequency of flooding immediately downstream of the dam and some peak reductions for moderate flood events. However, the gates would have no impact on major flood events as shown on the examples provided below.
- Further downstream, from Brandon to Portage la Prairie, the flood control effects diminish and nearly disappear.





Manitoba

UPPER ASSINIBOINE RIVER

Diking and/or Purchase of Flood Prone Land Affected by Operation of Shellmouth Dam

Diking Along the Assiniboine River

- The study evaluated diking the agricultural land that is affected by operation of the Shellmouth Dam along the upper Assiniboine River.
- Diking options from Shellmouth to St. Lazare as well as to Brandon were considered.
- In addition to the construction of new dikes, permanent pumping stations would be required at regular intervals to pump runoff from the agricultural fields.
- Elimination of flood plain storage would aggravate flooding downstream.

	Shellmouth to St. Lazare	Shellmouth to Brandon
Total Dike Length	180 km	570 km
Number of Pump Stations	40	140
Estimated Cost for 1:5 year protection (approximately 2,000 cfs to 10,000 cfs)	$25 Million	$100 Million
Estimated Cost for 1:50 year protection (approximately 6,000 cfs to 25,000 cfs)	$75 Million	$300 Million

Purchase of Flood Prone Land

- The land purchase option considered the purchase of flood prone lands at fair market value from Shellmouth to St. Lazare.

	Shellmouth to St. Lazare
Total Area of Land Purchased	1300 ha (3300 ac)
Total Area of Land Flooded (Agricultural Component Only)	700 ha (1700 ac)
Estimated Cost of Land Purchase	$20 Million

- The land purchase option is the preferred alternative over linear diking.

Manitoba

UPPER ASSINIBOINE RIVER

Increasing Flood Protection Level in Brandon

- Linear diking has been found to be the most feasible option to increase the level of protection in the City of Brandon.

- A commitment has been made to upgrade the city's flood protection level to the highest flood on record (2011 and 2014) at a cost of approximately $27 Million.

- The project will include:
  - Enhancements to the linear diking system within the city, including both improvements to the dikes and the dike drainage systems.
  - Upgrades to the lift stations along the Assiniboine River corridor.
  - Raising Provincial Trunk Highway 110.
  - New dike construction has already been completed along 18th Street North in the City of Brandon.

Supper Sandbag Dikes in Brandon (May & June 2011)

Manitoba

SOURIS RIVER

Increasing Community Flood Protection Level

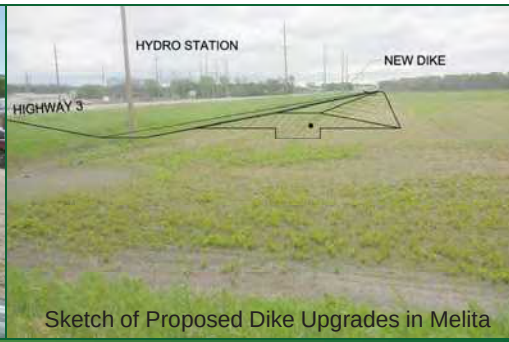
- Linear diking has been found to be the most feasible option to increase the level of protection in the communities of Melita and Souris.

- A commitment has been made to upgrade the flood protection level in the communities of Melita and Souris to the highest flood on record (2011 flood) at a cost of approximately $8 Million to $10 Million.
  - The 2011 flood was roughly a 1:150 year event in the two communities.
  - The projects are currently ongoing with an expected completion in 2015.

- In Wawanesa, the dikes are higher and the community is already protected to levels of at least the highest flood on record.

Sketch of Proposed Dike Upgrades in Souris

Super Sandbag Dike in Souris
(July 2011)



Sketch of Proposed Dike Upgrades in Melita

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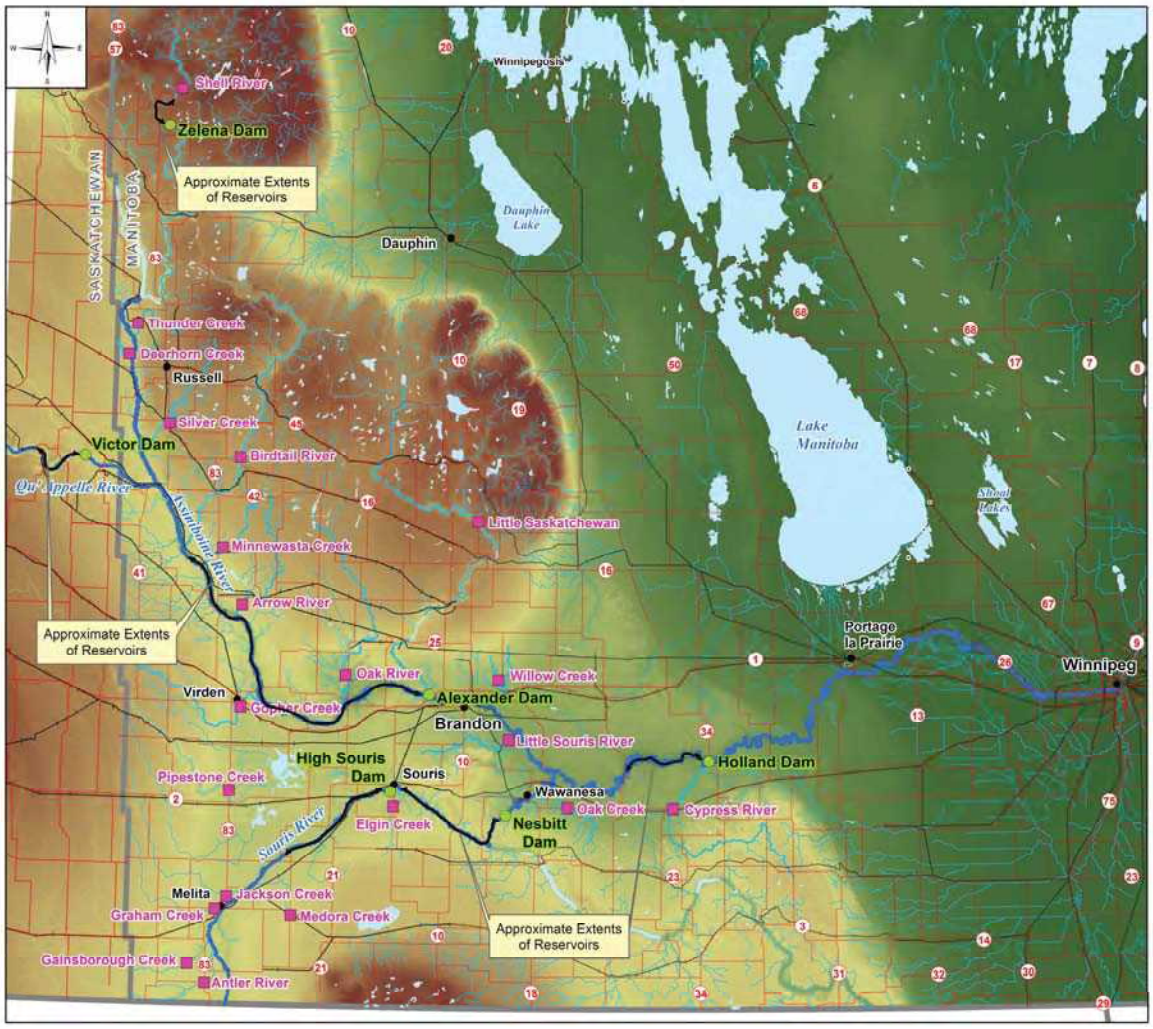
UPPER ASSINIBOINE RIVER AND SOURIS RIVER

Flood Mitigation with Small and Large Dams

- The study analysed six independent large dams as potential flood mitigation options as well as a combination of 21 small dams.



Manitoba

UPPER ASSINIBOINE RIVER AND SOURIS RIVER

Flood Mitigation with Large Dams

- The study analysed each large dam individually and assumed that they would be operated for flood control only and for peak flow reduction on the Assiniboine River.
- To achieve maximum benefits, the reservoirs must be emptied prior to the start of the flood. Also, the reservoir benefits increase with more accurate long range forecasts of runoff into the river systems.

	Holland Dam	Alexander Dam	Victor Dam	Zelena Dam	High Souris Dam	Nesbitt Dam
River	Assiniboine	Assiniboine	Qu'Appelle	Shell	Souris	Souris
Storage Capacity (dam^3)	880,000	1,600,000	170,000	250,000	39,000	408,000
Maximum Area Inundated by Reservoir (ha)	7,500	26,000	2,000	1,800	1,400	2,900
Average Flow over 3 Months to Empty Reservoir (cfs)	5,300	10,600	< 2,000	< 2,000	< 2,000	2,500
Approx. Peak Flow Reduction at Portage for a 1:200 Year Flood	10%	32%	3%	2%	5 %	8 %
Estimated Construction Cost	$270 Million	$525 Million	$145 Million	$90 Million	$116 Million	$211 Million

Note: Shellmouth Reservoir storage capacity is 480,000 dam^3

- The reservoirs would have significant environmental and social-economic impacts:
  - Post-flood discharge
  - Loss of land
  - Terrestrial habitat
  - Agriculture
  - First Nation
  - Impede navigation
  - Quality of water
  - Groundwater levels
  - Alteration of aquatic life
- Due to the significant environmental impacts the reservoir concepts should not be considered further.

Manitoba

UPPER ASSINIBOINE RIVER AND SOURIS RIVER

Flood Mitigation with a Combination of Small Dams

- The study analysed a combination of 21 small dams that would be operated strategically to reduce peak flows on the Assiniboine and Souris Rivers.
- A small dam was defined as a structure that would have a storage capacity of 35,000 dam^3 or less. In comparison, the existing Oak Lake Dam has a total storage capacity of 39,000 dam^3, and the existing Rivers Dam 30,000 dam^3.
- 21 sites were considered on tributaries of the Assiniboine and Souris Rivers, for a total storage capacity of 200,000 dam^3. In comparison, the Shellmouth Dam has a total storage capacity of 480,000 dam^3.

- The total cost of this option was estimated at $480 Million.

- Operating the dams effectively to achieve the maximum benefits would be very difficult:
  - Forecast dependent (difficult to accurately predict weather and runoff).
  - Must be timed perfectly such that water storage coincides with the flood wave on the Souris and Assiniboine rivers.
  - An operator or remote operation is necessary for each structure.

	Souris River at Wawanesa	Assiniboine River at Portage
Peak Flow Reduction for a 1:200 Year Flood (cfs)	1,800	3,200 to 4,600
Peak Flow Reduction for a 1:200 Year Flood (%)	5 to 9	8

- Some of the dams could have local benefits that may require further review.

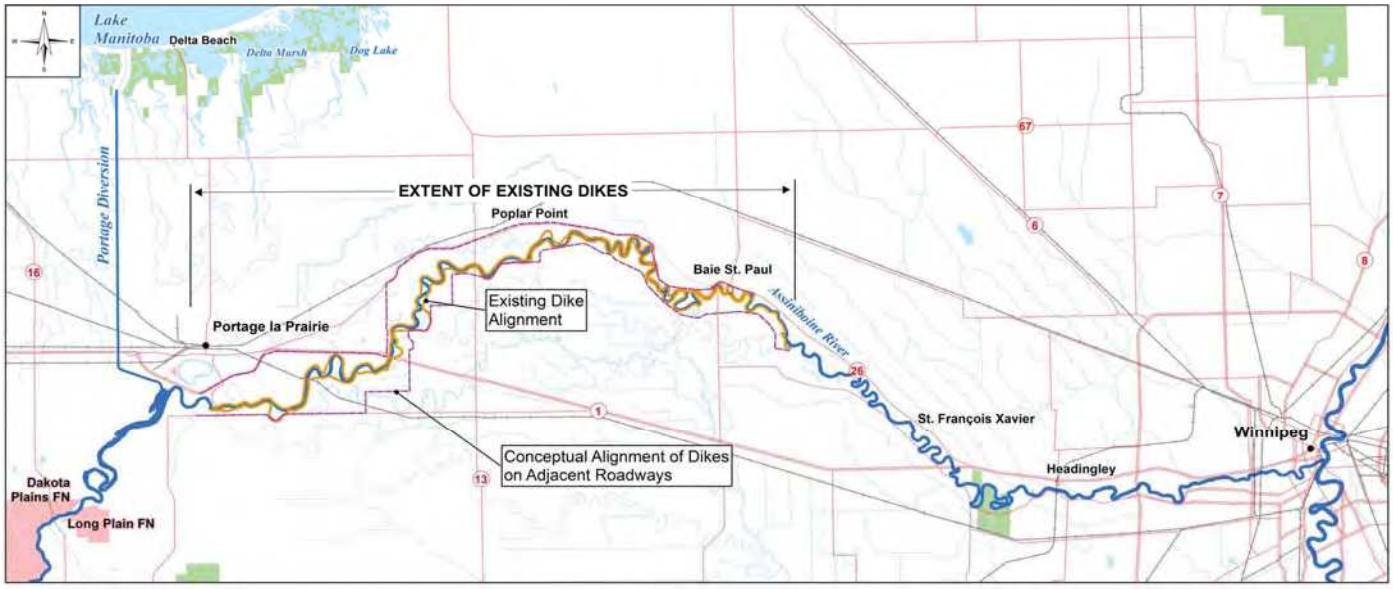
- It was concluded that flow reduction benefits on the Souris and Assiniboine rivers are relatively small and that this is not a feasible flood mitigation solution.

Manitoba

LOWER ASSINIBOINE RIVER

Increasing Capacity of Provincial Assiniboine Dikes

- The existing provincial dikes extend from Portage la Prairie to Baie St. Paul.
- The dikes were successful in providing protection during the floods of 2011 and 2014 at a flow of about 18,000 cfs, but required substantial emergency efforts.
- The capacity in 2011 and 2014 was less than in 1976 due to the difference in duration of the floods. As well, the late occurrence of the 2011 and 2014 floods in the season allowed the dikes to thaw, where in 1976 the dikes were frozen.
- The study evaluated three alternatives to increase the capacity of the existing provincial dikes for a range of flows:
    1. Upgrade existing dikes in current location.
    2. Move dikes to adjacent roadways.
    3. Combination of existing dike upgrades in some areas and moving dikes to adjacent roadways in other areas.



Manitoba

LOWER ASSINIBOINE RIVER

Increasing Capacity of Provincial Assiniboine Dikes

- The upgraded dikes would meet modern design standards:
  - Minimum two-feet freeboard above design water level.
  - Rip-rap erosion protection at critical areas.
  - Purchase land which dikes are located on to ensure maintenance access.
  - Minimum 20 feet top width for construction equipment.
  - Gravel topping for all season maintenance access.
  - All season road access at minimum three kilometres spacing.
- The option of moving the dikes to adjacent roadways includes the additional cost of purchasing the agricultural land and of individually protecting infrastructure located between the new dikes and the existing dikes.

Capacity	Portage la Prairie to Baie St. Paul		
	Upgrade Existing Dikes in Current Location	Move Dikes to Adjacent Roadways	Combination of Existing Dike Upgrades and Moving Dikes to Adjacent Roadways
18,000 cfs	$140 Million	$675 Million	$ 425 Million
23,100 cfs	$245 Million	$780 Million	$ 520 Million
28,000 cfs	$350 Million	$900 Million	$ 630 Million

- The most feasible option is to upgrade the existing dikes in their current location with some local upgrades to the alignment to address riverbank instabilities and improve access.

Manitoba

LOWER ASSINIBOINE RIVER

Increasing Flood Protection Level Downstream of Provincial Assiniboine Dikes

- Three options were considered to increase the level of protection downstream of the existing provincial dikes for a range of flows, from approximately Baie St. Paul to Headingley:
    1. Extend the provincial dikes.
    2. Individual flood proofing (ring dikes or raising buildings).
    3. Purchase of vulnerable properties.

- Extending the provincial dikes requires pump stations at about five locations for local drainage.
- The individual flood proofing and purchase options do not address the potential impacts to agricultural land.

Capacity	Baie St. Paul to Headingley		
	Extend Provincial Dikes	Individual Flood Proofing	Purchase of Vulnerable Properties
18,000 cfs	$80 Million	$ 3 Million	$ 37 Million
23,100 cfs	$115 Million	$ 12 Million	$ 85 Million
28,000 cfs	$154 Million	$ 24 Million	$ 121 Million

- The most feasible option is the individual flood proofing of vulnerable properties.

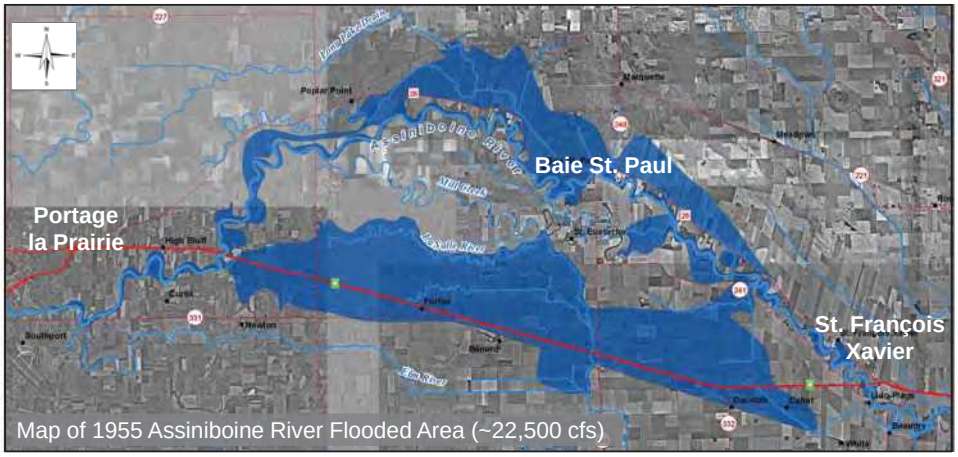
Manitoba

LOWER ASSINIBOINE RIVER

Individual and Community Flood Protection

- The study examined the option of protecting the homes and communities individually (similar to Red River Basin) instead of linear dikes along the river.

- Without the linear dikes, there would widespread flooding along the lower Assiniboine River area. Flows would spill into the La Salle River system during large floods which was not considered acceptable.



Map of 1955 Assiniboine River Flooded Area (~22,500 cfs)

- The estimated cost for individual and community flood protection at 23,100 cfs is estimated to be as high as $240 Million. This does not consider flood damages to existing infrastructure, which is expected to be significant in major floods.

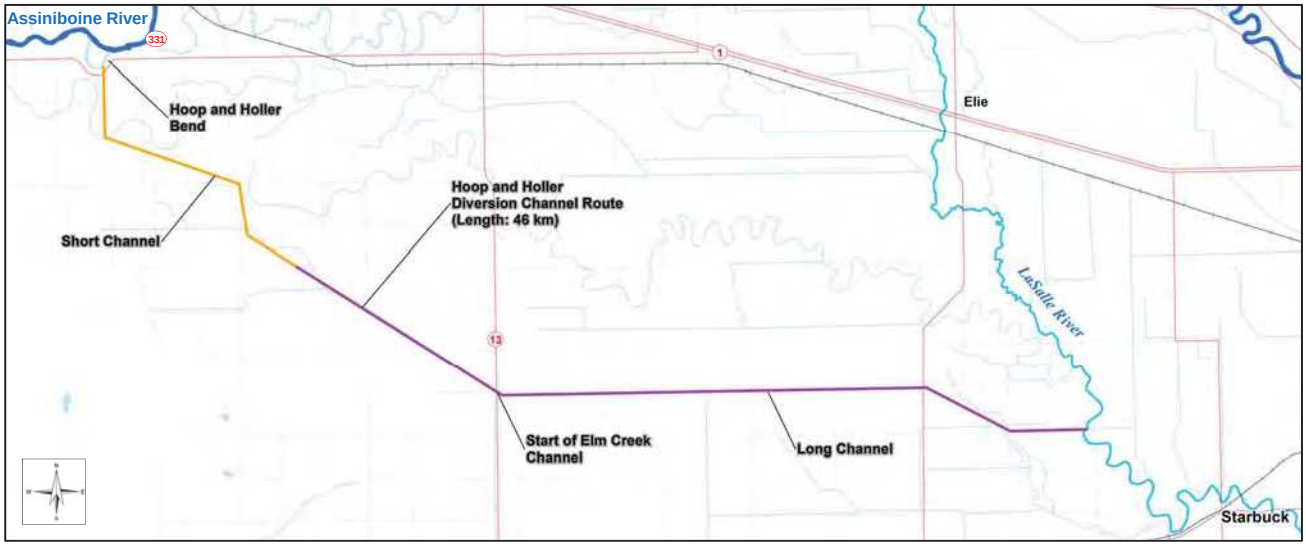
- The study has concluded that this option is not a feasible solution due to the negative impacts and costs of widespread flooding of the lower Assiniboine River area;
  - The transportation network would be severely affected with closures of many municipal and provincial roads.
  - Communities and individuals would need to be evacuated due to the road closures and limited access.

Manitoba

LOWER ASSINIBOINE RIVER

Hoop and Holler Release Diversion

- The 2011 Flood Review Task Force recommended to construct a permanent controlled wasteway to pass Assiniboine River flows in excess of the combined capacity of the Portage Diversion and Assiniboine River channel and dikes.
- The Hoop and Holler Diversion Channel would divert 3,900 cfs towards the La Salle River:
  - The capacity is limited by the La Salle River downstream and the diversion channel could not be used if the La Salle River is full due to local runoff.
  - Estimated cost ranges between $80 to $310 Million depending on where the excavated portion of the channel is terminated.
- It was concluded that Hoop and Holler is not feasible as a permanent option because it:
  - Increases risk of flooding on Elm Creek Drain and the La Salle River.
  - Has significant opposition from local stakeholders.
  - Is not an economical long term strategy compared to other flood mitigation options.

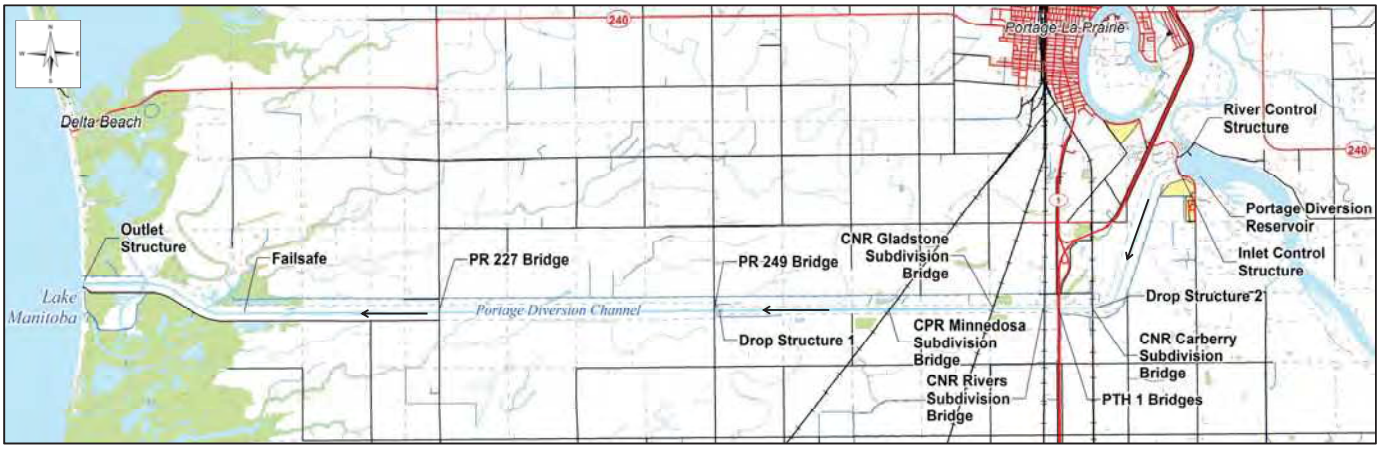


Manitoba

LOWER ASSINIBOINE RIVER

Options to Increase Portage Diversion Capacity

- The Portage Diversion was put into service in 1970 and consists of:
  - Approximately 29 kilometres long channel.
  - Three roadway bridges and four railway bridges.
  - Two drop structures.
  - River control structure and reservoir.
  - Diversion control structure and Outlet structure.
- The original design capacity of the Portage Diversion was 25,000 cfs.
- The capacity was increased to approximately 34,000 cfs on an emergency basis during the floods of 2011 and 2014.
- The Portage Diversion provides significant water level benefits to:
  - City of Winnipeg
  - City of Portage la Prairie
  - Communities, residents, agricultural producers and other stakeholders along the Assiniboine River and La Salle River watershed.
- Operation guidelines are currently under review by the province.



Manitoba

LOWER ASSINIBOINE RIVER

Options to Increase Portage Diversion Capacity

- Three options were evaluated to permanently increase the Portage Diversion capacity to 34,000 cfs.
  - Combined with 23,100 cfs provincial dikes and individual protection on the lower Assiniboine River, 34,000 cfs capacity provides protection for a 1:200 year flood.
- All options include elimination of the existing Failsafe by raising the low portion of the west dike near the outlet.

1. Widening of Existing Diversion Channel – Estimated Costs: $543 Million
   - Channel excavation and moving back of the Portage Diversion dikes on both sides of the channel.
   - Bridges, control structures and drop structures would have to be replaced or upgraded to be wider.

2. Construction of a New Parallel Channel – Estimated Costs: $333 Million
   - Excavation of a new channel and construction of new dikes parallel to the existing Portage Diversion.
   - West side was studied, however the east side could also be considered.
   - New bridges, control structures and drop structures would be required.
   - Has a dis-benefit of “doubling” the infrastructure (ex: increased environmental impact, maintenance activities, operating efforts and monitoring, etc.).
   - Includes the cost of maintaining the existing Portage Diversion infrastructure.

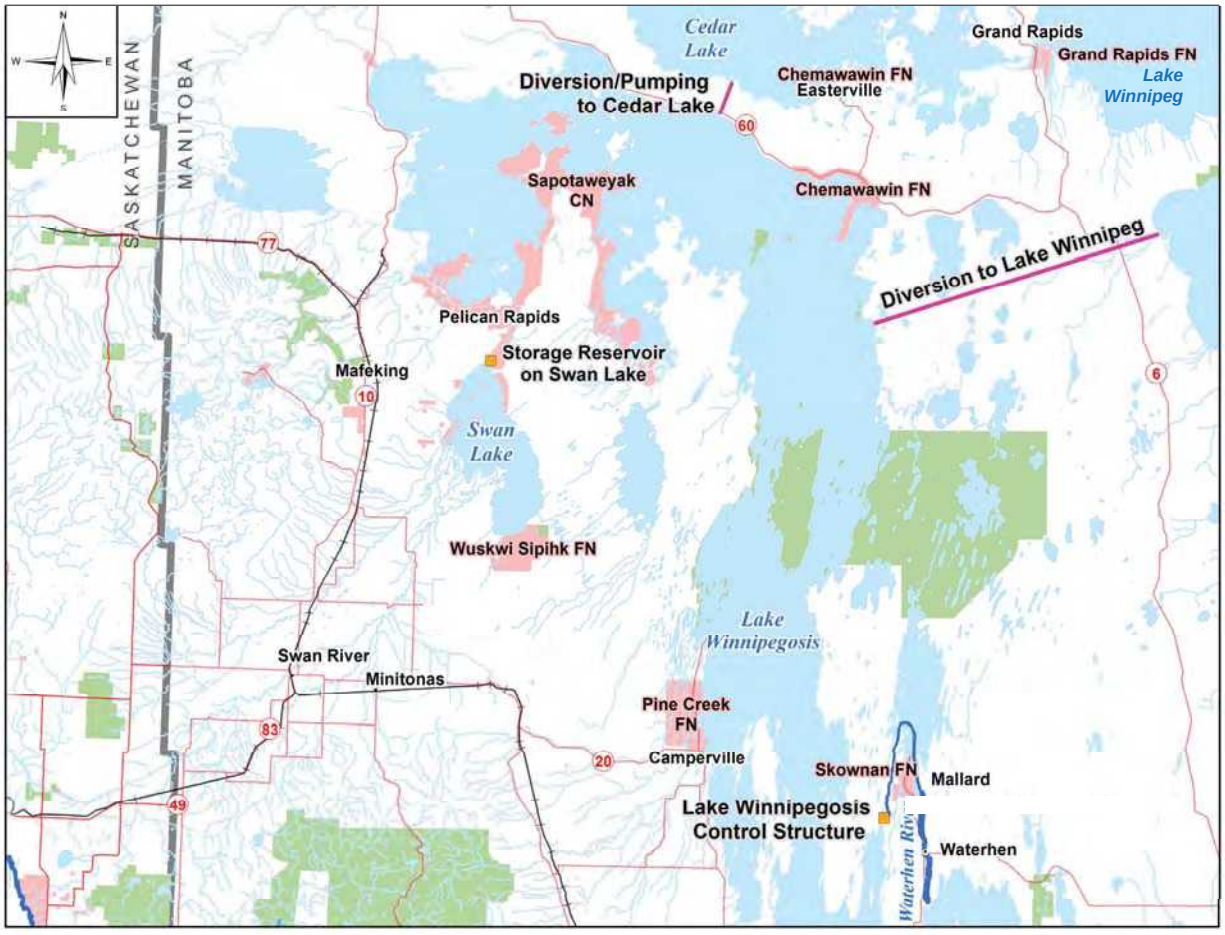
3. “Retrofit” the Existing Portage Diversion – Estimated Costs: $314 Million
   - Raising and upgrading of the dikes in the channel and in the reservoir.
   - Upgrading the control and drop structures, including the river control structure.
   - Several bridges would have to be replaced or upgraded due to the increased water levels in the channel.

Manitoba

LAKE WINNIPEGOSIS

Flood Mitigation Options

- Four flood mitigation options were considered on Lake Winnipegosis:
    1. Diversion Channel from Lake Winnipegosis to Cedar Lake.
    2. Diversion Channel from Lake Winnipegosis to Lake Winnipeg.
    3. Control Structure on Waterhen River.
    4. Storage Reservoir on Swan Lake.



Manitoba

LAKE WINNIPEGOSIS

Flood Mitigation Options on Lake Winnipegosis

- The Diversion Channel option to Cedar Lake is about six kilometres long and requires pumping as the average water levels in Cedar Lake are six feet higher.
- The Diversion Channel to Lake Winnipeg is over 60 kilometres long, which makes the option very costly.
- A control structure on the Waterhen River is not considered a feasible flood mitigation strategy as it would increase the water levels on Lake Winnipegosis.

Option	Design Capacity	Estimated Construction Cost	Effects on Lake Winnipegosis Water Levels in 2011 & 2012	Effects on Lake Manitoba Water Levels in 2011
Diversion to Cedar Lake	2,500 cfs to 10,000 cfs	$ 100 to $ 250 Million	Reduced by 1.0 to 3 ft	Reduced by 0.5 to 1.5 ft
Diversion to Lake Winnipeg	10,000 cfs	> $ 1.3 Billion	Reduced by 1.0 to 1.5 ft	Reduced by 0.5 to 1.5 ft
Control Structure on Waterhen River	n/a	$ 33 Million *	Increased by 0.5 to 4.0 ft	Reduced by 0.5 to 1.5 ft

* Note: This option has additional costs due to water level rises on Lake Winnipegosis.

- Developing Swan Lake into a storage reservoir could reduce water levels by 0.3 feet on Lake Winnipegosis and 0.1 feet on Lake Manitoba during a flood.
- An increase in water levels on Swan Lake would have environmental impacts.
- Field investigations and further studies would be necessary to estimate the cost.
- This study has concluded that Lake Winnipegosis options are not viable flood mitigation solutions due to the comparative costs of other alternatives and environmental issues.

Manitoba

LAKE MANITOBA & LAKE ST. MARTIN

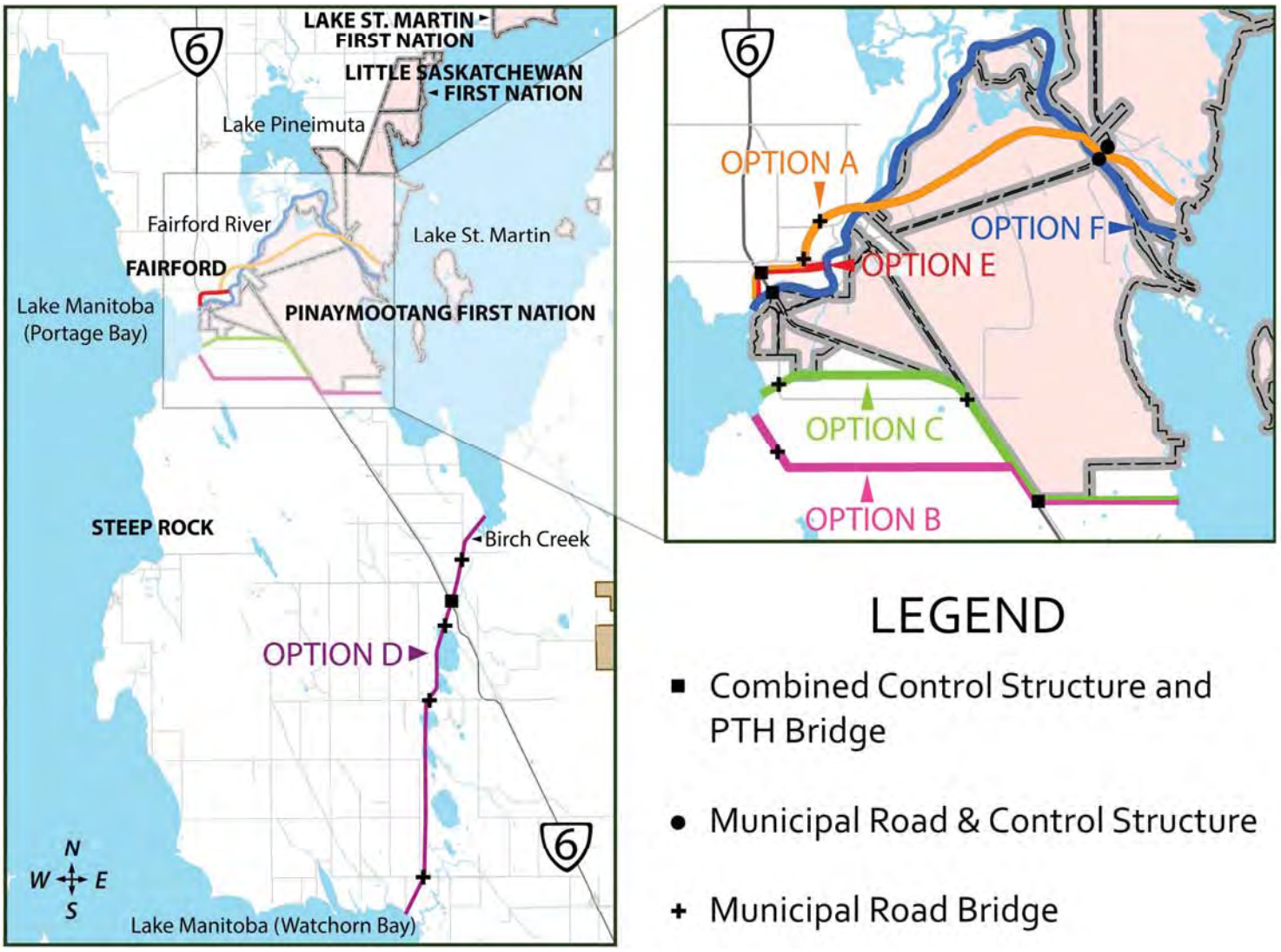
Conceptual Design of Outlet Channels

- The Province is committed to enhancing Lake Manitoba and Lake St. Martin outlet capacities to better regulate water levels on these lakes.
- An open house to solicit public opinion on the conceptual design options was held in September 2014 in Ashern.
- The preliminary recommendation was that the design should consider a 5,000 to 7,500 cfs channel from Lake Manitoba.
- The next stages of the outlet channel project include:
  - Preliminary engineering.
  - Aboriginal consultations.
  - Environmental and regulatory reviews.
  - Further public engagement.
  - Completion of the design.
  - Land acquisition.
  - Construction.
- The Province is currently proceeding with preliminary engineering for the preferred alternatives with a Lake Manitoba design capacity of 7,500 cfs, as announced in the November 20, 2014 Throne Speech.

Manitoba

LAKE MANITOBA

Preliminary Outlet Channel Options



Map showing Lake Manitoba Outlet Channel options

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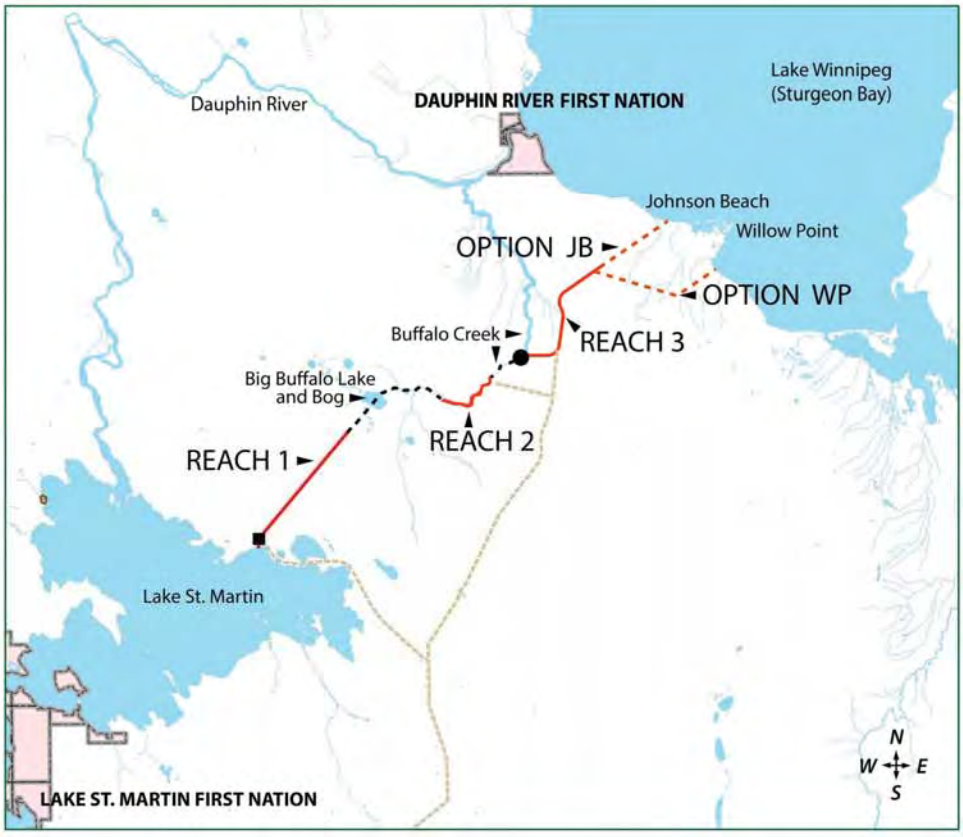
LAKE ST. MARTIN

Preliminary Outlet Channel Options



Map showing Lake St. Martin Outlet Channel options

LEGEND

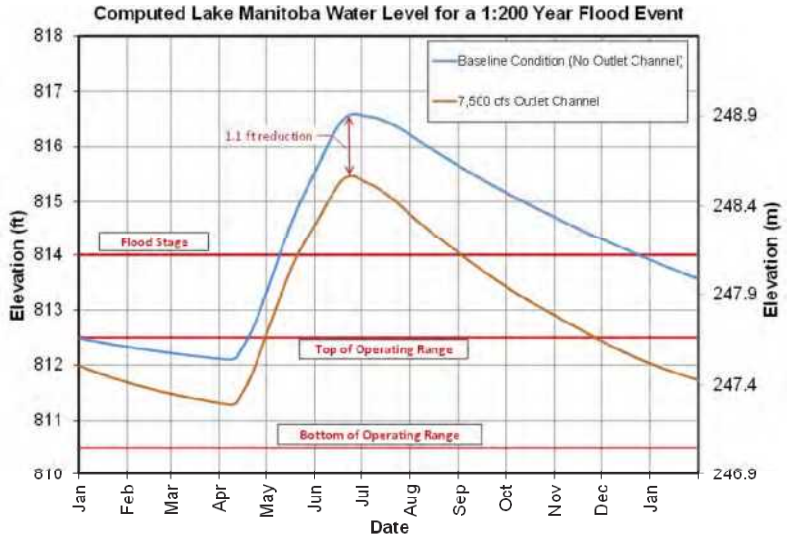
- Inlet Control Structure
  - Regulates flow from Lake St. Martin to Reach 1
- Proposed Access Road
- Control Structure
  - Diverts flow from Buffalo Creek to Reach 3 during flood operation

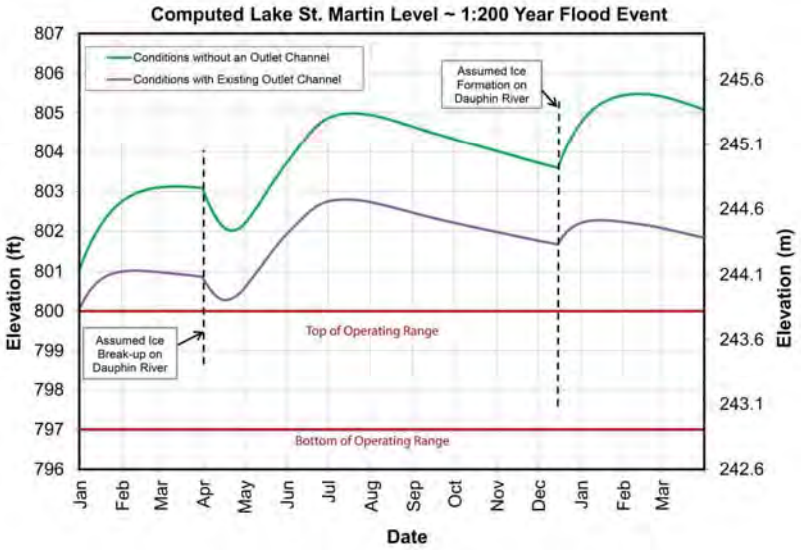
Manitoba

LAKE MANITOBA & LAKE ST. MARTIN

Benefit to Water Levels

- For a 200-year event, without outlet channels, the peak water level without wind would be about 816.6 feet on Lake Manitoba and 805.4 feet on Lake St. Martin.





- A new outlet channel would reduce the peak Lake Manitoba water level by 1.1 feet for a 7,500 cfs channel.
- Existing emergency outlet channel will reduce the 200-year peak Lake St. Martin level by 2.7 feet.
- These peak level reductions would be very similar for all options since a Lake Manitoba outlet channel would be combined with an expanded Lake St. Martin outlet channel.
- The magnitude of these peak level reductions would be very similar for floods ranging from a 100 to a 500-year event.
- Improvements to the lower Assiniboine River capacity will help further reduce water levels on Lake Manitoba.

Manitoba

LAKE MANITOBA & LAKE ST. MARTIN

Preliminary Conclusions and Recommendations

Lake Manitoba Outlet Channel

- The larger capacity of 7,500 cfs was adopted by the province (Throne Speech, November 20, 2014).
- Alignments C and D are preferred based on their cost effectiveness and high environmental ratings.
- Preliminary cost estimated at $240 Million.

Lake St. Martin Outlet Channel

- Should be made permanent with capacity of 11,500 cfs to accommodate the increased inflows from a new Lake Manitoba outlet channel.
- An inlet control structure and permanent access road should be constructed.
- Preliminary cost estimated at $210 Million.
- At this stage, the Willow Point option is preferred.
- The effects of the outlet channels on Lake Winnipeg would be negligible.

Option C Overview Towards Lake Manitoba (June 2011)

Option D Overview Looking South (June 2011)

Option D at Watchorn Bay (June 2011)

Manitoba

ADDRESSING THE LOWER ASSINIBOINE RIVER VULNERABILITIES

- The lower Assiniboine River was identified as an area which has infrastructure that is vulnerable at floods much smaller than the highest flood on record.

- Combinations of options were considered to increase the flood protection level (FPL) in the area.

- The 1:200 year flood was selected as a target mitigation FPL as per the 2011 Flood Review Task Force recommendation.

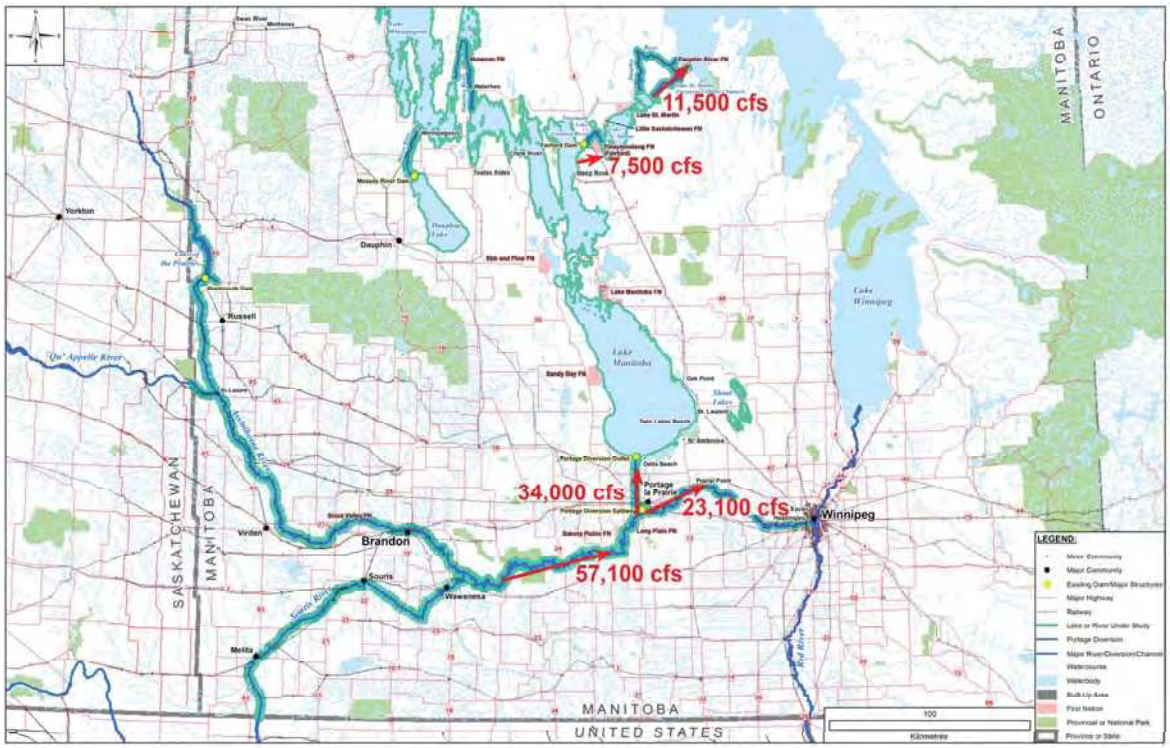
- The alternative recommended in this study consists of:
  - Upgrading the Portage Diversion to 34,000 cfs,
  - Upgrading the Assiniboine River capacity to 23,100 cfs,
  - Constructing a new Lake Manitoba outlet with a capacity of 7,500 cfs, and
  - Making the Lake St. Martin outlet channel permanent with a capacity of 11,500 cfs.

- This alternative is shown schematically on the next board.

Manitoba

ADDRESSING THE LOWER ASSINIBOINE RIVER VULNERABILITIES

Regulated Flood Protection Level – 1:200 year

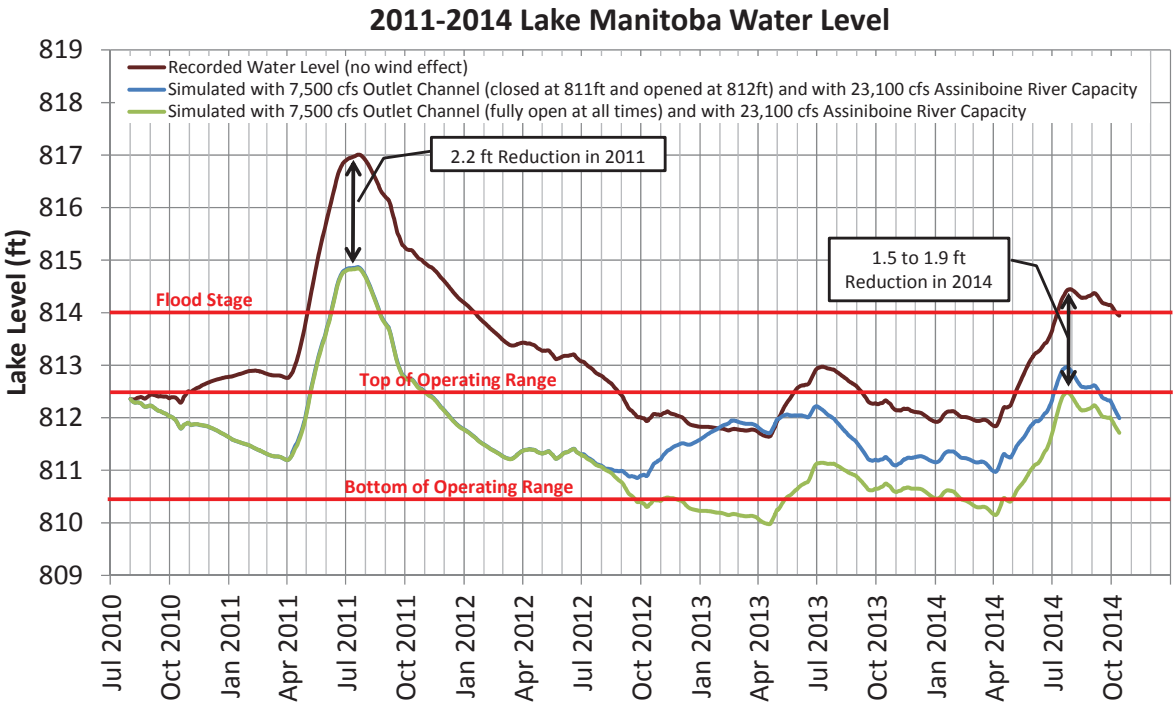


Flood Protection Infrastructure	Existing Capacity	Proposed Capacity	Preliminary Estimated Cost of Upgrade
Provincial Assiniboine Dikes	15,000 cfs	23,100 cfs	$245 Million
Assiniboine River Baie St. Paul to Headingley	17,000 cfs	23,100 cfs	$10 Million
Portage Diversion	25,000 cfs	34,000 cfs	$310 Million
Lake Manitoba Outlet	n/a	7,500 cfs	$240 Million
Lake St. Martin Outlet	4,000 cfs	11,500 cfs	$210 Million
		Total Cost	$ 1015 Million

Note: Estimated Costs do not include Individual Flood Protection on Lake Manitoba.

WATER LEVEL COMPARISON ON LAKE MANITOBA (2011 to 2014)

- Recorded water levels from 2011 to 2014 were compared to simulated water levels assuming:
  - A 7,500 cfs Lake Manitoba outlet channel .
  - Upgraded capacity of 23,100 cfs on the lower Assiniboine River.
- Results show decreased lake water levels due to the increased capacity of the lower Assiniboine River.



Notes:

- Assiniboine River flows were assumed to equal recorded flows after June 1 of each year.
- There would be additional benefits if flows on the Red River are below normal.
- Simulations assumes a 34,000 cfs Portage Diversion capacity.

Manitoba

DAUPHIN LAKE

Auxiliary Outlet Channel Options

- The peak water level of 861.1 ft on Dauphin Lake during the 2011 flood was the highest on record.
- The 2011 flood event was estimated to have a return period of 1:110 year.
- The second highest peak on record occurred in 2014 at approximately 859.7 ft.
- The study evaluated four auxiliary outlet channel options to lower Dauphin Lake levels in times of flood.

Option	Approximate Length	Approximate Width	Estimated Cost	Estimated Water Level Reduction in 2011
West Channel	27.2 km	30 m	$ 260 Million	2 ft to 2.5 ft
Tributary Channel	7.1 km	30 m	$ 80 Million	2 ft to 2.5 ft
East Channel	19.6 km	30 m	$ 190 Million	2 ft to 2.5 ft
Short Channel	14.7 km	30 m	$ 120 Million	2 ft to 2.5 ft
Short Channel	14.7 km	60 m	$ 225 Million	6 ft to 6.5 ft

Manitoba

DAUPHIN LAKE

Other Options to Lower Lake Levels

The study also considered:

- River channel improvements:
  - Requires the excavation of one foot to three and a half feet in the upper 12 to 15 kilometres reach of the river.
  - Preliminary cost estimates ranged between $14 Million to $50 Million.
  - Water level reduction in a repeat 2011 flood event would be one to two feet.
  - This option has significant environmental impacts such as the destruction and alteration to fish habitat.

- Storage development opportunities in watershed:
  - Over 33 dams on the scale of the Pleasant Valley Dam or 156 dams on the scale of the Vermillion Dam are necessary to reduce water levels by about two feet during a 2011 type flood.
  - This total number of structures is impractical.
  - The total volume of water stored with a lesser number of structures would be insignificant.

- Sediment deposition in the lake:
  - Sediment measurements in the 1990's showed a very slow rate of infill.
  - It would take centuries before the quantity of sediments deposited in the lake would affect the passage of floods.

This study has concluded :

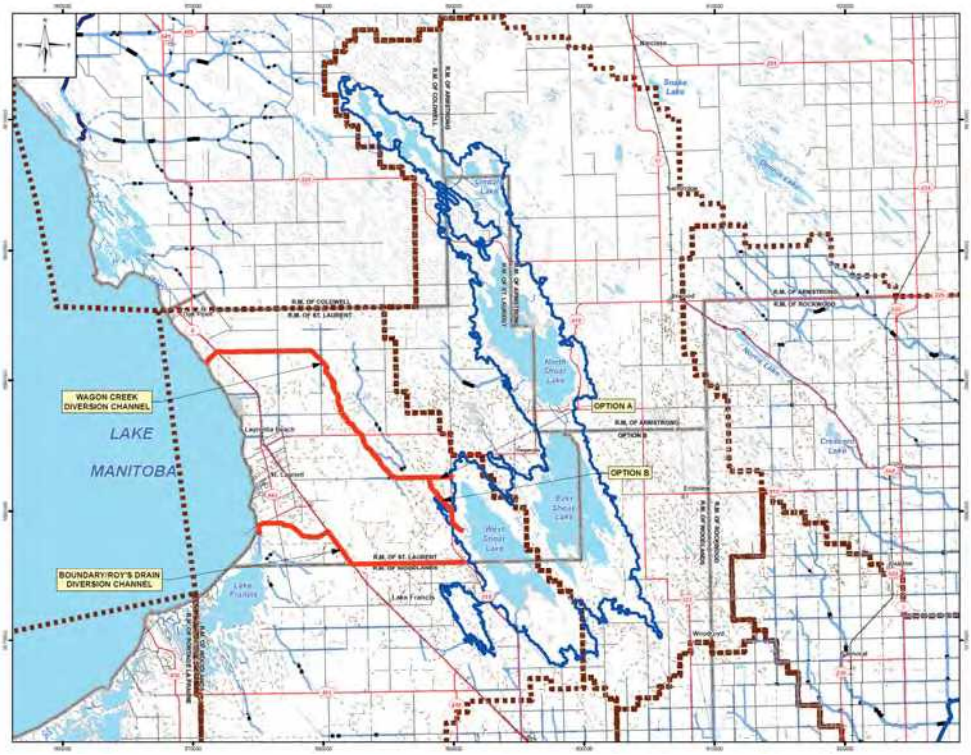
- Options investigated are not cost effective and have negative environmental impacts.
- Protection to the highest flood on record through Individual Flood Proofing and Development Controls is the best solution.

Manitoba

SHOAL LAKES

Flood Mitigation in the Shoal Lakes

- Flood mitigation alternatives to address flooding issues within the Shoal Lakes Watershed were previously studied in 2008 to 2010.
- Three options were considered at the time (2009 dollars):
  - Diversion / outlet channel ~ $25 Million – $30 Million.
  - Upland storage ~ $5 Million.
  - Purchase of flood prone lands ~ $11 Million.
- The most attractive option from an economic perspective was the purchase of flood prone lands, which is being pursued by the province. The original study identified approximately 16,900 hectares of land to be purchased, from 117 different property owners.



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DEVELOPMENT CONTROLS

- Development controls were considered as a means to restrict future development in a Designated Flood Area (DFA).

- With proper planning, a DFA can effectively limit flood damages that could occur in new developments in flood prone areas. However, it does not address vulnerabilities of pre-existing infrastructure and can also affect growth in existing communities. The impacts of a DFA on future development will vary depending on:
  - Location.
  - Current level of flood protection.
  - Extents of the DFA .

- Currently, only two areas of the province have a legislated DFA:
  1. Red River Valley – all development must be protected to 1997 flood plus two feet.
  2. Red River north of Winnipeg – all development must be at or above the recorded high water level plus two feet (varies between 1997 flood and ice-affected floods).

- The target Flood Protection Level varies by location across the province and is based on the following criteria:
  - The 1:200 year event, recommended by the 2011 Manitoba Flood Review Task Force, which is greater than the previous 1:100 year standard.
  - The highest flood on record, or
  - A higher flood event that can be justified in an economic analysis that considers costs and benefits.

- Where there is no DFA, Planning Districts and Municipalities have the authority to require that developments are flood protected. This authority should be more rigorously enforced.

- Lower Assiniboine River and Lake Manitoba are areas where DFA's should be considered.

Manitoba

WETLAND RESTORATION

- Analysis and literature review of over 40 documents was completed.

- Report by Dr. John Pomeroy: *Enhancements and Testing of the Prairie Hydrological Model* was released in the spring of 2014.
  - The study was based on the Smith Creek watershed, which is about 460 km^2 (the Assiniboine River Basin at Portage is about 160,000 km^2).
  - Indicates the impact of wetlands on peaks and volumes is also significant for major, large floods.
  - This finding for major, large floods is contrary to the conclusion of the majority of previous studies.

- General findings on wetland loss (pre Pomeroy):
  - No significant increase in peaks of major floods.
  - Increased peaks and total runoff volumes for moderate floods.

- A basin assessment was conducted to determine effects of wetland restoration using information from the Pomeroy Study:
  - For a repeat 2011 flood, a 15% restoration of wetlands could result in a peak flow reduction of approximately 30% at Portage la Prairie.
  - The estimated cost to restore 15% of wetlands is $1.5 Billion.

- Large scale wetland restoration is therefore not cost effective as a mitigation option. However smaller scale projects could be pursued as they arise.

- Manitoba's Surface Water Management Strategy endorses "no net loss" in wetlands and encourages water retention.

Manitoba

SHARING YOUR PERSPECTIVES

The project team would like to know your thoughts based on the information that has been shared here today.

Your input/feedback will help us as we proceed with finalization of the report by the spring of 2015.

- Comment forms are available for you to fill out.
- Forms can also be submitted electronically or by mail, so long as they are received before January 10, 2015.
- Details are provided on the comment form. Please ask a project team member to help you if you have any questions.

Contact information: feedback@floodstudy.ca

Mailing address: Colin Siepman, P.Eng., KGS Group
KGS Place
3rd Floor - 865 Waverley St.
Winnipeg, MB R3T 5P4

THANK YOU!

Manitoba

Assiniboine River and Lake Manitoba Basins

Flood Mitigation Study

Open House Comment Form

Please take this opportunity to fill in this brief Comment Form based on the information provided at this event. Please drop the completed comment form in the box provided. Comments may also be mailed or emailed to the addresses provided on the back of this form.

Your Name (optional): _____

Which of the following best describes your personal connection to this process (select all that apply)?

- ☐ Homeowner
- ☐ Cottage Owner
- ☐ Farmer/Rancher
- ☐ Business Owner
- ☐ First Nations Member
- ☐ First Nations Leader
- ☐ Municipal Elected Official
- ☐ Municipal Employee
- ☐ Provincial Employee
- ☐ Other: _____

How did you hear about the event?

- ☐ Newspaper
- ☐ Radio/public service announcement
- ☐ Letter invitation
- ☐ Email
- ☐ Word of mouth
- ☐ Other: _____

Was the information presented on the boards and/or your discussions with Open House staff helpful?

- ☐ Yes
- ☐ No

How could this Open House event have been more helpful to you?

_____

_____

_____

_____

What perspectives do you have about the options that have been presented today?

_____

_____

_____

_____

_____

_____

_____

_____

Are there any options that are of particular interest or concern? Please explain.

What other comments or questions would you like to provide today?

If you would like us to email a copy of the Open House boards to you, please include your email address (available online athttp://www.gov.mb.ca/mit/floodinfo/floodproofing/reports/index.html):

Email Address: _____

THANK YOU FOR TAKING THE TIME TO PROVIDE YOUR COMMENTS.
YOUR PARTICIPATION IS VERY IMPORTANT TO THE STUDY PROCESS.

If you would prefer, comments may also be submitted via mail or email no later than January 10, 2015 to:

Email: feedback@floodstudy.ca

Mailing address:
Colin Siepman, P. Eng KGS Group
KGS Place
3rd Floor-865 Waverley St.
Winnipeg, MB. R3T 5P4

APPENDIX A3

ROUND 2 OPEN HOUSE SIGN-IN SHEETS

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OPEN HOUSE SIGN IN SHEET

DECEMBER 8, 2014 - DAUPHIN, MANITOBA Leadership 1-3

PRINT NAME	SIGNATURE	AFFILIATION
Arthur Jonasse.	<original signed by>	Pk. of West Interlake
John BEZENER	<original signed by>	R.M. of Lost Interlake.
John Wainwright	<original signed by>	"
Ron Ryz	<original signed by>	Rm of Dauphin
Wayne Hildebrand	<original signed by>	RM of Langford.
CLINTON CLEAVE	<original signed by>	RM OF OCHRE RIVER
WALTER Tymchuk	<original signed by>	
John DeWarle	<original signed by>	
DON DUNFIELD	<original signed by>	RM of Dauphin
DENNIS FORBE	<original signed by>	RM Dauphin
11 Maurice Maguet	<original signed by>	RM Ste Rose

OPEN HOUSE SIGN IN SHEET

D OH (1)

1 of 2

DECEMBER 8, 2014 - DAUPHIN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
K. PIASTA	<original signed by>	Land Owner
HANA / ANDERSON	<original signed by>	Land owner
Jim Heshka	<original signed by>	MASRD
Richard Gattie	<original signed by>	Land owner
John Pshebinski		Benyk's Point Lake Man.
Clayton Watts	<original signed by>	Dauphin Beach
Jessica Pawluk	<original signed by>	Dauphin Herald
Troy Vendramin	<original signed by>	Cottage Owner
G. Bretecher	<original signed by>	manipogo Cottages
D. DEANS	<original signed by>	DAUPHIN LAKE
Ruth Zahorodny	<original signed by>	Land owner
Rosalie Bonny	<original signed by>	Cabin Owner Saltiston Lake Subdivison
Lyle Finney	<original signed by>	R.M. of Alonsa
Malcolm STRANG	<original signed by>	Ochre Beach -
Rod McFadyen	<original signed by>	City
Eleanor McLean		Resident - Benyk's Point
Lourens Arends	<original signed by>	Land owner
Kate Storey	<original signed by>	Landowner - Flooded
Wesley Saari	<original signed by>	CITIZEN
David Swanton	<original signed by>	Farmer

OPEN HOUSE SIGN IN SHEET

Dott (2) 2 of 2

DECEMBER 8, 2014 - DAUPHIN, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Elsie M Hill	<original signed by>	Hill's Resort
Ro VanDeVanDe Kerckhove	<original signed by>	Rancher
Derm England	<original signed by>	CONSULTANT PLANNER.
Derrick Venoman	<original signed by>	COTTAGE OWNER
Tom Gibbs	<original signed by>	Parks MB Con.
Wilc Borrin	<original signed by>	COTTAGE
Mike Cuthbert	<original signed by>	cottage owner.
Brian Byford	<original signed by>	RANCHER
Joseph Mauld	<original signed by>	Sioux Valley First Nation
Bert Rutbeek	<original signed by>	Land owner.
Rick Parcodi.	<original signed by>	Cottage Lake MB
Chris Dziadyk	<original signed by>	Landowner / farmer
Mark McAvoy	<original signed by>	CKDM radio
Marcel Keeling	<original signed by>	Town of Dauphin

4

OPEN HOUSE SIGN IN SHEET

Brandon Leadership

DECEMBER 9, 2014 - BRANDON, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
DENIS FOUILLARD	<original signed by>	RM Archie - Ellice
BLAIR WOODS	<original signed by>	Mayor Grassland
WIL REX	<original signed by>	COUNCILLOR VICTORIA RM
Tom Molland	<original signed by>	Hamiota/Assiniboine Basin Initiative
HAROLD BURKESS	<original signed by>	REEVE RM VICTORIA
Reg MARGINET	<original signed by>	R.M. Victoria council Member
Bob Adriaanse		Area RM North Cypress Pelton
TINA Williams	<original signed by>	Town of Virden
Joe Goodwill	<original signed by>	RM Glenwood
Stuart Olmstead	<original signed by>	Town of Carberry

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OPEN HOUSE SIGN IN SHEET

Bolt

① of 4

DECEMBER 9, 2014 - BRANDON, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Fred Taylor	<original signed by>	Farmer
DENIS CARTER	<original signed by>	REEVE RM WOODWORTH
Dorothy Howard	<original signed by>	LAND OWNER.
Mary & Rhonda Thiessen	<original signed by>	Land owner
Cram Bru	<original signed by>	Media (Bdn Sun)
James Hood	<original signed by>	Assiniboine Community College
Morris Drummond	<original signed by>	Land owner.
Rick Chrest	<original signed by>	Mayor of Brandon
BILL JEFFREY	<original signed by>	JEFFREY FARMS
Leigh Smith	<original signed by>	Landowner
RODNEY PEARN	<original signed by>	Landowner.
DENNIS McMILLAN	<original signed by>	Land + Cottage owner
Robert Howell	<original signed by>	Landowner
Ryan Johnston	<original signed by>	Landowner
MARCEL FOUILLARD	<original signed by>	Landowner
A.C(ALF) KENNEDY	<original signed by>	Longtime Brandon resident
Gerry Joynt	<original signed by>	Landowner
Richie Joynt	<original signed by>	"
KAREN FULHAM	<original signed by>	Landowner
Wendel Baelbeck	<original signed by>	Landowner - Brandon.
ROLAND ELLIOTT	<original signed by>	BRANDON - LAND OWNER
Anne Allen	<original signed by>	"
Lonnie Patterson	<original signed by>	Bdn. City Councillor

66 Signed in 1 did not and
3 media people did not sign in.

2 of 4

OPEN HOUSE SIGN IN SHEET

Both

DECEMBER 9, 2014 - BRANDON, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
STU BRIESE	<original signed by>	MLA Agassiz
BRIAN KAYES	<original signed by>	City of Brandon
Darren Harrison	<original signed by>	Prairie Mountain Health
Constant Fouillard	<original signed by>	St Lazare MB
Marc Fouillard	<original signed by>	St Lazare MB
David Howard	<original signed by>	land owner
Graham Jaur	<original signed by>	Brandon
ED LEZOND	<original signed by>	land owner
STAN COCHRANE	<original signed by>	ASS VALLEY ASS LAND OWNER RM SIFTON
David Henry	<original signed by>	landowner. RM Victoria
Larry Stewart	<original signed by>	Westman Journal
Heather Dalgleish	<original signed by>	RM of Cornwallis
DAVE BAXTER	<original signed by>	LAND OWNER (COTTAGE)
MARILYN Mc[illegible]	<original signed by>	PORTAGE LAND OWNER (COTTAGE)
Gerald Smith	<original signed by>	Land owner
Dennis Kowall	<original signed by>	Land owner
HENRY CAREY	<original signed by>	LAND OWNER
LINDSAY COWLING	<original signed by>	Brandon Youth Soccer Assoc.
Mark Kovatch	<original signed by>	Land Owner (Business)
Mel Hofer	<original signed by>	Land owner
REG HEMER	<original signed by>	MLA Brandon West
Jack HOFER	<original signed by>	Land Owners
RICHARD FULHAM	<original signed by>	LAND OWNER

3 of 4

OPEN HOUSE SIGN IN SHEET

Bolt

DECEMBER 9, 2014 - BRANDON, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Chris Adam	<original signed by>	RM Cornwallis
Ray Poole	<original signed by>	Oak Island Resort.
Kevan Sumner	<original signed by>	Manitoba Municipal Gov't
Darryl Gerrard	<original signed by>	Assiniboine Negawy
Uwe Johnas	<original signed by>	Land owner -
Paul MOORE	<original signed by>	Resident BRANDON.
ROD McKAY	<original signed by>	LANDOWNER, R.M. WHITEHEAD COUNCILLOR
Ted Snure	<original signed by>	Concerned citizen.
CLIFF MILLER	<original signed by>	MCA SPRUCE WOODS
Barrie Cochrane	<original signed by>	landowner RM Daly
Bill P	<original signed by>	Brandon University
12 Lois MacDonald	<original signed by>	Brandon Riverbank

OPEN HOUSE SIGN IN SHEET

6 4 of 4

Both

DECEMBER 9, 2014 - BRANDON, MANITOBA

	PRINT NAME	SIGNATURE	AFFILIATION
	MIKE WADDELL	<original signed by>	LANDOWNER / RM OF CORNWALLIS
	Jacqueline Moore	<original signed by>	Land Owner.
	STAN TAYLOR	<original signed by>	land owner
	Phil Weiss	<original signed by>	Brandon resident.
	DAVID W. FAURSCHOU	<original signed by>	POURTER LA PRAIRIE, MB.
	RON FORTIN	<original signed by>	SOURIS, MB.
	BRUNO SCHMALTZ	<original signed by>	PROV OF MB
8	Glen Newton	<original signed by>	Brandon, mb.

OPEN HOUSE SIGN IN SHEET

1 of 12
POH

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Amy Sissons	<original signed by>	Landowner.
Doug Simpson	<original signed by>	"
Brian Wilson	<original signed by>	Manitoba Ag.
Sandi Knight	<original signed by>	Landowner
Laurie Leslie	<original signed by>	Land Owner
Dan Brow	<original signed by>	Landowner
Shirley Tyrrell	<original signed by>	MR
Kelly Collett	<original signed by>	Portage Country Club
AL Myska	<original signed by>	TWIN LAKES Cottage
Joyce Myska	<original signed by>	"
Bryan Dion	<original signed by>	Business Owner/Land.
BRIAN SIGFUSSON	<original signed by>	R.M. of Coldwell Lundar MB
DOUG OLIVER	<original signed by>	WOODLANDS
<original signed by>	<original signed by>	Portage Landowner
Paul Trimble		landowner
Rick Pryer	<original signed by>	Consultant
Robin Rogers	<original signed by>	landowner
Keith Sparling	<original signed by>	Treherne farmer
Kelvin Van Camp	<original signed by>	Landowner
Todd Thurston	<original signed by>	Landowner
Rae Trimble-Olson	<original signed by>	Landrenter.
Fred de Leeuw	<original signed by>	LANDOWNER
HOPE SIGFUSSON	<original signed by>	Lundar MB

Open House 282 signed in about 5 did not
Leadership 17 signed in.

OPEN HOUSE SIGN IN SHEET

POH 2 of 12

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Gary Tully	<original signed by>	Landowner.
Cam Calvert	<original signed by>	
KEN HOLLAND	<original signed by>	Cottage
Bob McCallister	<original signed by>	Farm - Portage.
JASON BACCIARA	<original signed by>	LiDo Attp / CARTIER
Uschi Wright	<original signed by>	Delta Beach
Sheila Henderson	<original signed by>	Delta Beach homeowner
Don Drain		
Dan Wishart	<original signed by>	- MLA - Landowner.
Don Clarkson	<original signed by>	ALMS DELTA BEACH ASSOC INC.
Ken Mitchell	<original signed by>	
Lou Collett	<original signed by>	Delta Beach landowner
Roger Gillis	<original signed by>	St. Lawrence / Exw
Steve Hofer	<original signed by>	Barrickman Cell -
KERRY YANKE	<original signed by>	St. Francis Landowner.
ROLAND PATENAUDE	<original signed by>	LAND OWNER
Merv Pauch	<original signed by>	Delta Beach - Owner.
GWEN FERGUSON	<original signed by>	DELTA BEACH COTTAGE OWNER
DARYL CHECK	<original signed by>	" " " "
Doug Gall	<original signed by>	
Yolanda Yobb	<original signed by>	Oakville, Langenburg, Sk
Tyran. Nutbean	<original signed by>	" " " "
DAVID NUTBEAN	<original signed by>	Oakville, MB

OPEN HOUSE SIGN IN SHEET

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P04

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Gilles Chabot	<original signed by>	Cartier
Chad Leitold	<original signed by>	MIT
Dave Jeffries	<original signed by>	Jeffries Bros. Veg. Growers
K. Turko	<original signed by>	
Todd Eckert	<original signed by>	
Laurie & Brian Wolfe	<original signed by>	SFX
John Caiste	<original signed by>	Portage
Craig Jeffries	<original signed by>	Portage
Heather Gillis	<original signed by>	St. Laurent
Jonnette Kitson	<original signed by>	Delta
Sam Connery	<original signed by>	Connery Farms
Steve Hoffe	<original signed by>	Cartier
Shannon Mar	<original signed by>	MLA Morris
Chris Bures	<original signed by>	Portage la Prairie Chamber
Penny Delf	<original signed by>	Delf Farms
Dave Taylor	<original signed by>	Landowner
Barb Taylor	<original signed by>	Landowner.
John Stewart	<original signed by>	Landowner
Cam Stewart	<original signed by>	Landowner
ELEANOR CIFFIN	<original signed by>	Cottage owner 3X
Clorissa Egilson	<original signed by>	Langruth land owner
Bob Chubb	<original signed by>	Delta Home Owner.
Colleen Tomalin	<original signed by>	Delta Beach (Village).

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PoH

OPEN HOUSE SIGN IN SHEET

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
R Neufeld	<original signed by>	Plap MB
LAWRENCE PATON	<original signed by>	Bae St Paul
Tracey + Andy Taylor	<original signed by>	PlaP - Delta
Garry Clarke	<original signed by>	PlaP
Justyn McCabe	<original signed by>	PLAP.
Jim Leslie	<original signed by>	PlaP
LESLIE TBAI	<original signed by>	Bae St. Paul.
JOHN ROBINSON	<original signed by>	PLaP
Garry Wright	<original signed by>	Delta Beach
Wes Hamm	<original signed by>	PLAP
JON GERRARD	<original signed by>	Woodlands
Kyle Nichol	<original signed by>	PlaP
BRIAN WOLFE	<original signed by>	ST FRANCOIS
	<original signed by>	
Derrick Berthelet	<original signed by>	PLAP
MARSHALL PIPER	<original signed by>	LASALLE - RED RIVER C.D.
LARRY BAKER	<original signed by>	OWNER MB. ASSOCIATION OF COTTAGE
Rob Schinkel	<original signed by>	Bedville Beach
CRAIG CAMPBELL	<original signed by>	DELTA BEACH
Terry Skinner	<original signed by>	DeltaBeach.
Kevin Yuill	<original signed by>	Diversion / Lake Manitoba
Garett Gall	<original signed by>	Dog gall
Kris Eglson	<original signed by>	Langruth - Landowner

OPEN HOUSE SIGN IN SHEET

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POH

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Bob Saraamp	<original signed by>	Poplar Point
Todd Campbell	<original signed by>	Delta
E. Trimble &	<original signed by>	Delta
MARSHALL MCLEOD	<original signed by>	Macdonald, MB
Dianna Meseyton Neufeld	<original signed by>	Portage
Gilbert JACQUEMAIN	<original signed by>	Baie St Paul
Chris Connery	<original signed by>	Connery farms
Andrea McCabe	<original signed by>	Portage
Aaron Doherty	<original signed by>	Portage
Robert Pragnell	<original signed by>	Portage
Joel Martens	<original signed by>	St. Francois Xavier
Trevor Wuerch	<original signed by>	St. Laurent MB
Ken Hoffman	<original signed by>	St Francois Xavier
Shauna-Lee Leslie		Portage la Prairie H+H
Robert Stanger	<original signed by>	PlaP H+H
KIM WATSON	<original signed by>	BAIE ST PAUL - Aug 26
Todd Andrich	<original signed by>	DELTA BEACH - Portage
ROGER MORACHNIE	<original signed by>	MIT WATER CONTROL
Roland Bouchard	<original signed by>	Elie, Gretna
GORDON STEWART	<original signed by>	DELTA
HAZEL STEWART	<original signed by>	DELTA
GEORGE DAYTON	<original signed by>	ST FRANCOIS XAVIER
WARREN NEUFELD	<original signed by>	BERKVILLE BEACH

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POH

OPEN HOUSE SIGN IN SHEET

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
BOB KITCHEN	<original signed by>	landowner
Allan Adams	<original signed by>	landowner / Security
S. Ducks	<original signed by>	Landowner
L. Ducks	<original signed by>	Landowner
Kevin Mason	<original signed by>	Farmer
Mike Stanton	<original signed by>	landowner
Jean & Lennie Lesieur	<original signed by>	LANDOWNER..
Laurent Ponsin	<original signed by>	Landowner
RANDY PAINTER	<original signed by>	LANDOWNER.
MARhe I Mynott	<original signed by>	Landowner.
Harvey D Murray	<original signed by>	Assiniboine
Ann Lacroix	<original signed by>	landowner
Bobby M Dal	<original signed by>	Land Owner
Chris McCallister	<original signed by>	landowner
Jan Unrau	<original signed by>	landowners
Harold Unrau	<original signed by>	LANDOWNER
Harry + Holly DEANDER	<original signed by>	
Vienneo Egan	<original signed by>	landowner Dell in Beef
Joseph wife	<original signed by>	Land owner
ANDRE ALLARD	<original signed by>	Land owner
Gary Smith	<original signed by>	Landowner
Jonathan Gross	<original signed by>	Land owner
AL [illegible]	<original signed by>	INTEREST

OPEN HOUSE SIGN IN SHEET

7 of 12
Port

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Shea Doherty	<original signed by>	Hoop and Holler
Shannon Tessier	<original signed by>	Holstein Hoop Holler
Chris Emerson	<original signed by>	Ortros Owner Mng.
STEVE DENKEWICH	<original signed by>	GREENKEEPER NURSERY
JACK McGOWAN	<original signed by>	Oakland-Delta Marsh
Doug White	<original signed by>	Farmer
Larry Rushton	<original signed by>	Land Owner
WAYNE Ostapowich	<original signed by>	Farm Manager
Judi Linden	<original signed by>	DBA
Alex STEWART	<original signed by>	Home Owner/Landowner
GERRY MEILLEUR	<original signed by>	ITZKE RIVER FARM LTD.
Blair Geisel	<original signed by>	
Terry Knight	<original signed by>	ASSI. RIVER
Maureen Cousins	<original signed by>	MB Beef Producers
LYLE GILBERT	<original signed by>	Portage Country Club.
Trine Ferris	<original signed by>	City of Portage
TERRY [illegible]	<original signed by>	P/S?
Don McRitchie	<original signed by>	Manitoba
Darlene Rice Doty	<original signed by>	M.S.
Toby Trimble	<original signed by>	RM of Portage
John & Tammy [illegible]	<original signed by>	High Bluff
John Hoy		
Roland Jeffries	<original signed by>	Jeffries Bros. Vegetables

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POH

OPEN HOUSE SIGN IN SHEET

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Todd Pelletier	<original signed by>	Farmer+landowner
Dale Hinton	<original signed by>	" " "
DAVE ZAROSKI	<original signed by>	Farm + Hog + Hobby
CHERRY GRETSINGER	<original signed by>	LO OWNER
JUSTIN REID	<original signed by>	LA SALLE RESIDENT CD
Ray BOND	<original signed by>	Landowner
Jame Hofer	<original signed by>	Southern Breeze
Dan Ste		North Rim
Nelson Hulme	<original signed by>	Farmer.
LORNE HENRY	<original signed by>	FARM -
Jackie North	<original signed by>	Farm
Mark Peters	<original signed by>	Farm
Gunter Fachmann	<original signed by>	Farm
Bob Thiessen	<original signed by>	LAnd Owner
Steve Ilk	<original signed by>	Land Owner
Ted Arent	<original signed by>	" "
DARIL GOBERT	<original signed by>	LAND OWNER
Jim Moothouse	<original signed by>	Landowner
PATTI LOBEL	<original signed by>	Business + Landowner
JEFF BIRD	<original signed by>	RESIDENT
Jonathon Wipf	<original signed by>	Farmer
Sylvie Allard	<original signed by>	Ret. teacher
Corey Murray	<original signed by>	Farmer

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POH

OPEN HOUSE SIGN IN SHEET

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
LARRY MUIRHEAD	<original signed by>	DELTA BEACH ASSOCIATION
Harvey Dahlen		Delta
R.L. Bicknell	<original signed by>	Hoop & Holler
James J. Leslie		Poplar Point
L. Roberts	<original signed by>	Hoop & Holler
Gaye Boehme	<original signed by>	Delta Beach
HEATHER EVERSON	<original signed by>	Hoop & Holler
WALTER KUZYK	<original signed by>	Poplar Point
Kevin & Wendy Bartindale	<original signed by>	Hoop & Holler
Hope Hamilton	<original signed by>	
PRESTON MEYER	<original signed by>	PORTAGE la PRAIRIE CHAMBER OF COMMERCE
TIM CHEVRIER	<original signed by>	LIDO PLAGE NORTH
Bryan Ezako	<original signed by>	Lido Plage North
Debbie Burnett	<original signed by>	Hackberry Delta
LORNE JORDAN	<original signed by>	Land owner 4-17-7 W1
Kindra Brookes	<original signed by>	Portage Planning District
Lyle McDonald	<original signed by>	PR Diversion
GLEN SAWATZKY	<original signed by>	242 Highway
KAY & LYNDA MOAR	<original signed by>	DELTA BEACH
Gary + Jodi Friesen	<original signed by>	Hwy #26, 41 46.
KEITH NICHOL	<original signed by>	Delta
JOE JOHNSON	<original signed by>	Langruth
JIM MUIR	<original signed by>	Inland Group

OPEN HOUSE SIGN IN SHEET

OH- PoH

10 of 12

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
PETER W SINCLAIR	<original signed by>	LAND OWNER
Richard Leclerc	<original signed by>	Land Owner
DOUG LOUK	<original signed by>	LAND OWNER
Malcolm Wild	<original signed by>	Westbourne
Ed Patzer	<original signed by>	Land owner
BRAD KNIGHT	<original signed by>	Land owner
Jim Knight	<original signed by>	Lake -
Bert Rutbeek	<original signed by>	Land owner
Garry Sanderson	<original signed by>	Land owner
Blye Sissons	<original signed by>	Land owner
PHILIP THORDARSON	<original signed by>	Landowner
ROSS TUFFORD	<original signed by>	LANDOWNER
JIM FRIESEN	<original signed by>	WINNIPEG
Ryan Hunter	<original signed by>	Land owner
Michele Tumber	<original signed by>	landowner SFX.
MIKE TUMBER	<original signed by>	✓ S.F.X
Shawn Cabak	<original signed by>	
Ray Verney	<original signed by>	Landowner Lake MB
Jill Verney	<original signed by>	Landowner Lake MB
Cynthia Dietz	<original signed by>	U Manitoba
BRUCE BROWN	<original signed by>	LANDOWNER
LINDSAY HANNESON	<original signed by>	LAND OWNER
TRICIA HOARE	<original signed by>	Landowner

OPEN HOUSE SIGN IN SHEET

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Polt

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Leif Nutbean	<original signed by>	Oakville
Bob Harrison	<original signed by>	Private
DWAYNE CLARK	<original signed by>	SFX REMS
JAMES DEGROOT	<original signed by>	SFX.
Kelly Tomlin	<original signed by>	RBA DELTA RESIDENT
Pat Hughes	<original signed by>	Delta Beach Homeowners
Stefan Heller	<original signed by>	DBWA + DIVERSEA
Rudy Schmidt	<original signed by>	Oakville
Helen Schmidt	<original signed by>	Oakville
DWAYNE LESLIE	<original signed by>	POPLAR POINT
Jane Froese	<original signed by>	Hoop + Holler
ALAN OWENS	<original signed by>	DELTA BEACH
Jenna Rocco	<original signed by>	Portage Graphic
Barry MacBride	<original signed by>	BDM Project
Walter Fast	<original signed by>	
Sharon Kohut	<original signed by>	Hall's Beach
PAUL RIEDLE	<original signed by>	RESIDENT.
Stephanie Page	<original signed by>	Beckville Beach.
FRED WATTS	<original signed by>	Hillcrest Bluffs.
Dan Muzia	<original signed by>	Keystone Ag Producers
Doug Faurschou	<original signed by>	Land / Cottage owner
Gerald Faurschou	<original signed by>	" "
Allen Dell	<original signed by>	Land Owner

OPEN HOUSE SIGN IN SHEET

Port

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

12 of 12

PRINT NAME	SIGNATURE	AFFILIATION
Aaron Wilgosh	<original signed by>	CFRY Radio
PAT SPARLING	<original signed by>	Boyne Valley Water Initiative
GEORGE KOTELO	<original signed by>	COTTAGE OWNER DELTA
Janet Painter	<original signed by>	cottage owner house/land by diversion
Leonard Dziedzic	<original signed by>	Flooded landowner
Cullen Pereira	<original signed by>	R.M. of CARTIER - Flooded home owner.
Claude Regnier	<original signed by>	RM of SFX Home owner
Estelle Regnier	<original signed by>	RM of St FX Home owner
Monica Griffiths	<original signed by>	Flooded Assiniboine farmer St E ana.
JOHN DAVID GRIFFITHS	<original signed by>	FLOODED 2011 ASSINIBOINE FARMER ST Eustache, 2013 & Nov 2014
Michael Williamson	<original signed by>	Flooded @ Beaconville Beach 2011+2014
BILL BIEW	<original signed by>	LAURENTIA BEACH
Tom Teichroeb	<original signed by>	Landowner MB.
RICHARD GRAY	<original signed by>	MB MUN. GOV'T
Lorne Rossnagel	<original signed by>	Pleasant mb.
Deb Rossnagel	<original signed by>	Pleasant mb.
Beth Connery	<original signed by>	P la P., MB. Rural landowner
MITCH MILLER	<original signed by>	Oakville MB

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OPEN HOUSE SIGN IN SHEET

Leadership and Poll

DECEMBER 10, 2014 – PORTAGE LA PRAIRIE, MANITOBA

PRINT NAME	SIGNATURE	AFFILIATION
Estelle Thornson	<original signed by>	RM of Cartier
Dale Fossay		RM of Cartier
Rick Plaisier	<original signed by>	AMM & RAM Station
Kevin Blight	<original signed by>	RM OF PLAP
Larry Gibbs	<original signed by>	RM of Portage
Dale Wood	<original signed by>	RM of Portage
Trevor Kirby	<original signed by>	RM of Woodlands
Kam Blight	<original signed by>	RM Portage
Ken Beaudin	<original signed by>	RM Cartier
Virginia Beckwith	<original signed by>	" "
Jason Tanguay	<original signed by>	RM of Cartier
Andrew Peters	<original signed by>	R.M. Cartier.
Tim deDelley	<original signed by>	RM Cartier
Darrel Hrechkosiuk	<original signed by>	RM of Portage.
Irvine Felcis	<original signed by>	City of Portage
Roy Tuffaro	<original signed by>	RM of Portage
Laura Rogers	<original signed by>	RM of Cartier
[illegible]	<original signed by>	Farmer.
[illegible]	<original signed by>	Farmer.
Karen Timchuk	<original signed by>	RM of Cartier
Bob Bilous		" " "
Susan Fulton	<original signed by>	" " "
[illegible]	<original signed by>	City of Portage

6 Open House 17 Leadership

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APPENDIX A4

ROUND 2 STAKEHOLDER SUBMISSIONS (CONFIDENTIAL)