37 REQUIREMENTS FOR THE FEDERAL ENVIRONMENTAL ASSESSMENT

Federal requirements for the environmental assessment of the Project are addressed, as required by Section 23 of the EIS Guidelines, in various sections of the EIS as presented in Table 37.1.

Table 37.1 Federal Requirements for the Environmental Assessment of the Project

Federal Requirement	Relevant Section of the EIS
Indirect Effects: Section 5 of the CEAA requires an assessment of the environmental effects of the Project, including certain indirect effects	The requirements to assess the following indirect effects are found in the sections of the EIS referred to below: • Volume 3 Economic and Land and Resource Use Effects Assessment Sections 16–27 • Volume 3 Section 19 Current Use of Lands and Resources for Traditional Purposes • Volume 4 Social Effects Assessment Sections 28–31 • Volume 4 Section 33 Human Health Effects Assessment • Volume 4 Section 32 Heritage Resources (Any structure, site, or thing that is of historical, archaeological, paleontological, or architectural significance, and physical and cultural heritage)
Alternatives to and Alternative Means	Volume 1 Section 5.3 Alternatives to the Project Volume 1 Section 6~~4~~ Alternative Means of Carrying Out the Project
Need for and Purpose of the Project	Volume 1 Sections 5.1 and 5.2 Need for and Purpose of the Project
Species at Risk Act	Volume 2 Section 12 Fish and Fish Habitat, Section 13 Vegetation and Ecological Communities, and Section 14 Wildlife Resources
Comments From the Public and Aboriginal Persons	Volume 1 Section 9 Information Distribution and Consultation
Current Use of Lands and Resources for Traditional Purposes	Volume 3 Section 19 Current Use of Lands and Resources for Traditional Purposes
Effects of the Environment on the Project	Volume 5 Section 37.1 Effects of the Environment on the Project
Potential Accidents and Malfunctions	Volume 5 Section 37.2 Potential Accidents and Malfunctions
Cumulative Effects	Volume 2 Sections 12–15, Volume 3 Sections 16–27, Volume 4 Sections 28–33, and Volume 5 Section 37.3 Summary of Cumulative Effects
Capacity of Renewable Resources	Volume 5 Section 37.4 Capacity of Renewable Resources
Requirements of any Follow-Up Program	Volume 2 Sections 12–15, Volume 3 Sections 16–27, Volume 4 Sections 28–33, Volume 5 Section 37.5 Requirements of Any Follow-Up Program, and Volume 5 Section 39 Complete List of Mitigation and Follow-Up Measures

The federal considerations required by Section 23 of the EIS Guidelines are the following:

- Changes to the environment

- Changes to components of the environment within federal jurisdiction
- Changes to the environment that would occur on federal or transboundary lands
- Changes to the environment that are directly linked or necessarily incidental to federal decisions
- Effects of changes to the environment on Aboriginal peoples
- Effects of changes to the environment that are directly linked or necessarily incidental to federal decisions

Potential effects, mitigation measures, residual effects, and a determination of significance for federal considerations are summarized in Volume 5 Section 38 Table 38.2.

37.1 Effects of the Environment on the Project

37.1.1 Introduction

Local conditions and natural hazards, such as severe or extreme weather conditions and external events, could adversely affect the Project. This section identifies the potential effects of the environment on the Project and describes the planning, design, and construction strategies to avoid or minimize those potential effects.

The following environmental factors that would potentially affect the Project are described:

- Extreme weather events
- Sedimentation of the reservoir
- Seismic activity
- Wildfire
- Flooding
- Low flow or drought conditions
- Slope stability and mass wasting events
- Climate change

The environmental factors listed above could adversely affect the Project by:

- Causing an accident or malfunction (the potential effects of accidents and malfunctions on the environment are described in Section 37.2)
- Delaying the Project in-service date by more than one year
- Interrupting service during operation
- Causing damage to infrastructure that compromises the safety of employees or the public
- Causing damage to infrastructure that is not economically or technically feasible to repair

To mitigate these potential effects, risk assessments are undertaken to select the appropriate return period or recurrence interval for the natural hazards that are used in the design. In assessing risk, the return period for a natural hazard is typically selected based on the potential financial, social, and environmental consequences of a malfunction or accident that could result from the hazard. A return period, also known as a recurrence interval, is an estimate of the interval of time between events such as an earthquake or flood of a certain intensity or size. A return period is a statistical measurement denoting the average recurrence interval over an extended period of time.

The selection of the return period also considers the longevity of a structure. For example, a lower return period may be selected for the design of a temporary construction infrastructure that would be in service for fewer seasons than a permanent infrastructure. In addition, lower return period events may be used for temporary works, as construction equipment is at hand to undertake any additional mitigation works and to make any necessary repairs.

In describing the probability of natural hazards, the term annual exceedance frequency is sometimes used. An event with an annual exceedance frequency of 1/10,000 has a return period of 10,000 years and has a 1% probability of occurring in a 100-year period.

37.1.2 Environmental Data

This section identifies the sources of data that were used to characterize baseline conditions at the Project, used to select the return periods for natural hazards, and used in the design to mitigate the potential effects of the environment on the Project.

37.1.2.1 Climate Data

Local and regional climate data were collected from Environment Canada, the Alberta Government, BC Hydro, the Western Regional Climate Center, the National Climatic Data Center, U.S. National Oceanic and Atmospheric Administrative Forecast Systems, and radiosonde facilities in Edmonton and Montana to understand and assess local and regional data. The locations of the local and regional meteorological stations are shown in Figure 37.1.

Local climate data were used to determine design parameters such as wind intensity and direction, precipitation as both rain and snow, and temperature.

Regional climate data were required to determine probable maximum events, which include both flooding and precipitation. As described in Section 37.1.9, regional climate data were used to identify probable maximum precipitation and the probable maximum flood.

37.1.2.2 Flow Data

Refer to Volume 2 Section 11.4 Surface Water Regime for a description of the surface water hydrology of the Peace River.

Floods on the Peace River at the dam site would consist of regulated discharges from Peace Canyon Dam combined with unregulated inflows from the catchment between Peace Canyon Dam and the proposed Site C dam site. The catchment between Peace Canyon Dam and the proposed Site C dam site is referred to as the Site C local catchment.

Flow data were collected from the Water Survey of Canada (WSC) gauges 07FA004 (Peace River above Pine River, referred to in this section as the Site C gauge) and 07EF001 (Peace River at Hudson's Hope, referred to in this section as the Hudson's Hope gauge).

Regional flow data were collected for the prediction of the probable maximum flood. This required the analysis of flow data in both the Williston and Site C local catchments.

37.1.2.3 Seismic Data

Refer to Volume 2 Section 11.2 Geology, Terrain, and Soils for descriptions of the regional and site-specific seismic data, and the regional and site-specific seismic hazard assessments.

37.1.3 Extreme Weather Events

Extreme weather conditions that were considered in the design and planning for the Project include:

- Wind
- Rain
- Snow
- Ice
- Lightning
- Low temperatures

These extreme weather events would be mitigated by the selection of appropriate return periods based on hazard assessments described in Section 37.1.1. The following subsections describe the potential effects of the extreme weather events listed above on the following Project components:

- Dam, generating station, and spillways
- Transmission line and substation
- Highway 29 realignment and other roads
- Hudson's Hope shoreline protection

37.1.3.1 Dam, Generating Station, and Spillways

37.1.3.1.1 Wind

Wind can adversely affect the dam, generating station, and spillways by:

- Generating waves on the reservoir that:
  - Cause erosion
  - Combined with set-up, overtop the reservoir retaining structures
- Damaging buildings that house generating equipment and flow control equipment

Wind-generated waves and set-up can damage the earthfill dam by:

- Eroding the upstream face
- Overtopping and eroding the crest and downstream face

These potential adverse effects on the earthfill dam are mitigated by providing:

- Riprap on the upstream face to prevent erosion
- Sufficient freeboard to prevent the dam from being overtopped

The RCC buttress, power intakes, spillway headworks, and other concrete structures that retain the reservoir are:

- Not susceptible to erosion by waves
- Less susceptible to damage by overtopping

Therefore, the freeboard requirements for the earthfill dam govern.

The Canadian Dam Association Dam Safety Guidelines (CDA 2007a) recommend the following freeboard for earthfill dams to allow for wind effects:

- Normal freeboard:
  - With the reservoir at the maximum normal level, no overtopping by 95% of the waves caused by the wind with a return period of 1,000 years
- Minimum freeboard:
  - With the reservoir at the maximum flood level during passage of the inflow design flood, no overtopping by 95% of the waves caused by the wind ranging from return periods of two years to 100 years, depending on the consequences of overtopping

As described in Volume 2 Section 11.10 Microclimate, the North Peace Regional Airport, EC Climate ID: 1183000, is the only location close to the microclimate study area where long-term climate information is available. Meteorological stations were installed in the Peace River valley to support the evaluation of the microclimate. The first full year of climate data collection from the stations in the Peace River valley was completed in January 2012.

Wind data from the ~~Fort St. John a~~North Peace Regional Airport were used for the analysis of wind-generated waves and wind set-up on the reservoir, and for the assessment of the freeboard required for dikes, cofferdams, and the earthfill dam. This is because wind data have been recorded at the airport station over a much longer period, which is better suited to the frequency analysis required to estimate return periods.

A comparison between the wind data from the meteorological stations in the Peace River valley and the wind data from the ~~Fort St. John a~~North Peace Regional Airport station showed that wind speeds at the ~~Fort St. John a~~North Peace Regional Airport station up on the plateau are higher than in the valley. Therefore, using the wind data from the airport gives conservative results.

The weather station at the airport is located about 12 km northeast of the dam site, and the period of record analyzed was from 1971 through 2010.

The wind velocities recorded at the airport were increased by 20% to allow for the wind over water effect, which is the tendency for higher wind speeds over open water than over land. It was assumed that the wind would blow consistently from the direction that would produce the worst-case wind set-up and wind-wave effects.

Table 37.2 shows the required freeboard and the provided freeboard for the Stage 2 upstream cofferdam and the earthfill dam.

Table 37.2 Freeboard Requirements for Wind Effects

Structure	Normal		Minimum	
	Required [a]	Provided	Required [b]	Provided
Stage 2 upstream cofferdam [c]	1.0 m	13.6 m	0.9 m	5.9 m
Earthfill dam [d]	1.2 m	7.6 m	1.1 m	3.1 m

NOTES:

[a] Normal freeboard calculated with 1,000-year wind

[b] Minimum freeboard calculated with 100-year wind

[c] For the Stage 2 upstream cofferdam:

- Normal freeboard calculated with the upstream water level for passage of 2,100 m^3/s, equal to full discharge from Peace Canyon powerhouse plus 118 m^3/s flow from the local catchment between Peace Canyon Dam and the Project
- Minimum freeboard calculated with the maximum flood level during passage of the June 2001 flood, which has an estimated return period of 60 years; see Section 37.1.8 for more information on floods used for the design of the Stage 2 diversion

[d] For the earthfill dam:

- Normal freeboard calculated for the maximum normal reservoir level
- Minimum freeboard calculated for the maximum flood level during passage of the inflow design flood; see Section 37.1.9 for more information on the inflow design flood

The freeboard provided on the Stage 2 upstream cofferdam and the earthfill dam are greater than required for wind effects because other requirements govern:

- See Section 37.1.10.3 for Stage 2 upstream cofferdam for a description of the landslide-generated wave freeboard requirements
- See Section 37.1.3.1.4 for a description of frost protection of the impervious core of the earthfill dam and Section 37.1.10.3 for a description of the earthfill dam landslide-generated wave freeboard requirements

The riprap on the upstream face of the Stage 2 upstream cofferdam has been designed for the 100-year wind wave. The riprap on the upstream face of the earthfill dam cofferdam will be designed for the waves generated by the 1,000-year wind or a wind speed of 100 km/h, whichever is greater.

Structures such as the powerhouse superstructure and other buildings will be designed for wind in accordance with the National Building Code of Canada, using applicable importance factors. The provisions of the National Building Code of Canada are based on 50-year return period winds.

37.1.3.1.2 Rain

Drainage and erosion control measures such as ditches, culverts, detention ponds, and riprap are required to control runoff and prevent erosion or other damage to the dam, generating station and spillways, and associated roads and infrastructure.

Extreme rainfall as it relates to floods during operations is described in Section 37.1.9.

At the dam, generating station, and spillways, drainage and erosion control measures such as ditches, culverts, and riprap would be designed for:

- The 20-year rainfall for temporary works during construction
- The 200-year rainfall for the permanent works

37.1.3.1.3 Ice

Ice can adversely affect the dam, generating station, and spillways by:

- Raising water levels during construction, due to the ice front reaching the dam site area
- Loading from thermal expansion of ice on the reservoir

The current ice regime in the Peace River and the predicted ice regimes during construction and during operation are described in Volume 2 Section 11.7 Thermal and Ice Regime.

An ice front is predicted to reach the dam site area once every 15 years on average. As described in Volume 2 Section 11.7 Thermal and Ice Regime, as the ice front advances upstream, water levels typically rise by between 1 m and 5 m, due to the increased resistance and thickness of the ice cover.

Except for the Stage 2 upstream cofferdam, all temporary construction infrastructure, including roads and cofferdams, would be constructed to have at least 1 m freeboard above the maximum predicted ice level with the ice front at Site C. The freeboard for the Stage 2 upstream cofferdam with the water levels increased by 5 m due to the ice front reaching Site C would be 8.6 m with an inflow of 2,100 m^3/s.

The ice studies described in Volume 2 Appendix H Reservoir Water Temperature and Ice Regime Technical Data Report indicate that in the year with the thickest ice cover on the reservoir, the approach channel could be ice-free. Nevertheless, the dam, generating station, and spillways would be designed for the ice loads that could occur during operations due to thermal expansion of ice on the reservoir and in the approach channel.

37.1.3.1.4 Low Temperatures

Low temperatures could adversely affect the dam, generating station and spillways by:

- Frost penetration damaging the core of the earthfill dam
- Ice buildup preventing the operation of spillway gates
- Freezing preventing operation of other flow control equipment

Frost penetration can damage impervious cores constructed from frost-sensitive soils due to the formation of ice lenses during the winter and the subsequent thawing in the spring (Solymar and Nunn 1983).

Factors that mitigate such damage are surcharging the core with gravel to prevent frost penetration and to increase the confining pressure on the core material (Solymar and Nunn 1983; Paré et al. 1978).

In addition to the freeboard requirements for earthfill dams to allow for wind effects, the Canadian Dam Association Dam Safety Guidelines recommend that the thickness of the material covering the impervious core should be sufficient to prevent freezing of the core in winter.

Based on the freezing index for North Peace Regional Airport, the maximum frost penetration would be less than 3 m. Therefore, 3 m of gravel would be provided above the top of the core to prevent frost penetration.

Operation of spillway gates would be required in the winter to maintain downstream flows if the generating station went off-line for any reason. Operation of spillway gates in the winter can be prevented by ice buildup on the body of the gates and on the seals around the perimeters of the gates. The downstream face of the spillway gates would be provided with an insulated enclosure, and heaters would be provided to prevent ice buildup.

All other flow control equipment would be housed in heated enclosures or provided with heaters to maintain required operability during the winter.

Heating and insulation design would be based on the extreme minimum temperature measured at North Peace Regional Airport.

37.1.3.2 Transmission Line and Substation

The extreme weather events that could adversely affect the transmission infrastructure are:

- Wind, ice, or combined wind and ice that can overload transmission lines or towers
- Snow pack and snow conditions, including snow creep that can overload towers
- Lightning strikes that can cause a power outage

The transmission line and substation would be designed to withstand, at a minimum, 1 in 200-year wind, ice, and snow loads.

Lightning arresters, overhead ground wires, and low station ground resistance would be used to protect the transmission and substation equipment from potential damage by lightning strikes.

37.1.3.3 Highway 29 Realignment and Other Roads

The extreme weather events that could adversely affect Highway 29 and other road infrastructure are:

- Wind, which would:
  - Load bridges
  - Generate waves on the reservoir, which could erode embankments and causeways
- Rain, which would have to be drained from the road surface
- Snow, which would:
  - Add load to the bridges

  - Have to be drained when it melts
- Ice, which could jam between piers, adding a lateral load

The realigned segments of Highway 29, which would form part of the provincial highway system, and other permanent roads would be designed according to the B.C. Ministry of Transportation and Infrastructure (BCMoTI) standards for wind and snow loading on bridges and for road drainage.

Ice loads for design of the new bridges on the Highway 29 realignment segments would be in accordance with the Canadian Highway Bridge Design Code based on the worst predicted ice conditions on the reservoir during operation, described in Volume 2 Section 11.7 Thermal and ice Regime.

Wind-generated waves and wind set-up for the 200-year wind would be used for the design of the erosion protection of the portions of the Highway 29 realignment segments along the reservoir shoreline.

37.1.3.4 Hudson's Hope Shoreline Protection

Wind-generated waves and wind set-up for the 200-year wind would be used for the design of the riprap for the Hudson's Hope shoreline protection.

37.1.4 Drought

A potential effect of a drought on the Project would be financial, as low flows would result in reduced generation.

Williston reservoir has a multi-year storage capacity; consequently, it has the capability to store water in wet years for use in dry years. This storage capability partially mitigates the effect of a drought on the Project.

Volume 1 Section 7 Project Benefits describes the studies undertaken to determine the capacity and energy provided by the Project. The average annual energy of 5,100 GWh is based on historic inflows for a 60-year period. The firm energy of 4,700 GWh per year is based on the consecutive three-year sequence with the lowest inflows in that 60-year inflow series.

This shows that the upstream storage in the Williston Reservoir would allow the Project to generate approximately 92% of its average annual energy under drought conditions.

37.1.5 Sedimentation of the Reservoir

As discussed in Volume 2 Section 11.2 Geology, Terrain, and Soils, most of the proposed reservoir and tributary shorelines are surrounded by steep slopes, which are currently actively eroding and slumping. The terrain stability and post-reservoir erosion and stability impact lines are presented in Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 1 Terrain Stability Mapping, and Part 2 Preliminary Reservoir Impact Lines, respectively.

Sediments would enter the reservoir from the following sources:

- Tributary bedload and suspended load
- Material from shoreline erosion

- Material from potential landslides

The predicted behaviour of sediment and bedload in the proposed reservoir is described in Volume 2 Section 11.8 Fluvial Geomorphology and Sediment Transport Regime. It is predicted that coarser material from shoreline erosion would deposit adjacent to the shore and form a beach. Finer material would become suspended in the reservoir. Of the suspended reservoir sediments, all of the sand and the majority of the silt would settle in the reservoir. The remaining silt and clay would be suspended in the reservoir outflow and leave the reservoir. After 50 years of operation, it is predicted the thickness of reservoir sediment deposition would range from about 0.3 m to about 0.5 m in the main reservoir and from 3 m to 4 m near some shoreline sections. As described in Volume 2 Appendix I Fluvial Geomorphology and Sediment Transport Technical Data Report, the volume of the sediment deposited in the reservoir over 50 years would be 2.5% of the initial reservoir volume.

The actual mechanism for shoreline erosion is undercutting and over-steepening of the slopes by wave erosion, and shallow landslides from the over-steepened slopes. The shoreline erosion sediment volume estimate includes the contributions from both erosion and the associated landslides.

Considering only the slopes above the area of predicted erosion and the associated shallow landslides, it is estimated that there is about a 10% chance in 100 years of a landslide with sufficient mobility to put in the order of 2 million m^3 of material into the reservoir. By comparison, the 1973 Attachie slide deposited approximately 7 million m^3 on the valley bottom that would be occupied by the proposed reservoir, and it is estimated there is less than a 1% chance in 100 years of a similar-sized landslide with similar mobility.

The volume of sediment expected to enter the reservoir from landslides originating above the area of predicted erosion is small relative to the reservoir volume of 2,310 million m^3. Further, the benefits of the Project do not depend on reservoir storage, as the generation results from flows entering the reservoir from upstream and not from storage in the Project reservoir. Therefore, sedimentation of the reservoir will not have an adverse effect on the Project.

37.1.6 Seismic Activity

37.1.6.1 Dam, Generating Station, and Spillways

37.1.6.1.1 Lessons Learned from Major Earthquakes

~~37.1.7~~ Concrete Dams

Nineteen concrete dams have been shaken by peak horizontal ground accelerations greater than 0.3g (Hansen and Nuss 2011). Based on the performance of these dams, Hansen and Nuss report:

- In general, concrete dams have performed very well when subjected to high-intensity accelerations
- The threshold of no damage is project specific, but is probably substantially higher than 0.3g for properly designed and constructed concrete dams

- While a fault located directly below Shih Kang Dam in Taiwan caused a rupture and relative vertical displacement of 9 m, the remaining damaged concrete limited an immediate total and sudden release of the reservoir
- Peak horizontal ground accelerations are amplified from the base of the dam to the crest; in two cases, namely Pacoima arch dam in the USA (2.3g) and Kasho gravity dam in Japan (2.05g), this amplification produced measured peak accelerations at the crest in excess of 2.0g
- Peak accelerations at the crest, as expected, are greater with full reservoirs
- Several dams have been severely shaken on two occasions by separate major earthquakes with only minor damage, for example. Bear Valley Dam in the USA, with earthquakes one day apart, and Pacoima Dam in the USA, with earthquakes 23 years apart
- Many concrete dams have also been shaken by high-intensity aftershocks that occurred after the main earthquake without any additional damage
- Roller compacted concrete dams have performed no differently to date than a dam built of conventionally placed concrete
- Where damage has been identified, it has been cracking in the upper levels of the dam, and where additional features such as curbs, railings, gates, or guard or control houses are located
- Very little in the way of increased leakage has occurred in concrete dams subjected to major earthquakes, which can be attributed, in part, to the fact that any cracking caused by the earthquake has mainly been horizontal and located high in the dam, together with the reservoir not being full in many cases
- Some rock foundations have experienced a temporary increase in seepage following an earthquake
- There may be a number of reasons why concrete dams have performed well and invariably better than that predicted by design or analysis when shaken by an earthquake; the main reasons may be:
    - The redundancy of the damaged structure to redistribute load
    - The duration of strong shaking being too short to cause failure
    - The increase in the tensile strength of the concrete during dynamic loading that increases resiliency
    - An increase in the damping that reduces the seismic impact on the dam
    - Reduced seismic impact because the natural frequency of the dam does not match the frequency of the earthquake
    - Three-dimensional effects of canyon confinement or dam geometry (curvature) that help prevent failure.

A generally accepted potential failure mode for concrete dams during an earthquake is cracking of the concrete through the dam, forming removable blocks that slide during or after the earthquake. Hansen and Nuss report that severely shaken concrete dams to

date have cracked at locations of change in geometry (reentrant corners), but have not formed removable concrete blocks; thus, the entire potential seismic failure mode has not been fully achieved or experienced for concrete dams.

~~37.1.8~~ Earthfill Dams

Experience has shown that well-compacted earthfill dams perform well in earthquakes. However, dams constructed of hydraulic fill or founded on loose soil are susceptible to liquefaction.

Many dams have suffered deformation and damage resulting from earthquakes; for instance, the M8.0 Wenchuan earthquake of May 12, 2008 in China damaged over 1,500 dams, some seriously, but no dam failed (New South Wales Dams Safety Committee 2010).

Embankment dams can suffer two main types of damage, depending on the nature of foundation or fill materials, and the design and construction standards (New South Wales Dams Safety Committee 2010):

- Deformations, slumping, and cracking, which could lead to failure from loss of freeboard or piping along cracks
- Liquefaction of materials in either the foundation or the dam fill

Reports of the performance of dams under earthquake shaking have been provided by Seed (1980), Hinks and Gosschalk (1993), and USCOLD (1992). These sources show that there have been cases of damage, and some failures. In summary, there have been some 20 to 30 failures of earth dams, most less than 20 m high. Most, if not all, of the failures involved liquefaction of saturated, loose, fine-grained cohesionless soils.

Sherard (1967) listed the following as possible ways that an earthquake may cause failure of an earthfill dam:

- Disruption of the dam by movement on a major fault in the foundation
- Loss of freeboard due to differential tectonic ground movements
- Slope failures induced by ground motions
- Loss of freeboard due to slope failures or soil compaction
- Sliding of dam on weak foundation materials
- Piping though cracks induced by the ground motion
- Overtopping by seiches in the reservoir
- Overtopping due to slides into the reservoir
- Failure of spillway or outlet works

The lessons learned from the performance of dams during major earthquakes have been incorporated into the practices of modern dam engineering. The design of the dam, generating station, and spillways for the Project has been done in accordance with this practice.

37.1.6.1.2 ~~37.1.8.1.1~~ Performance Requirements

As described in the regional and site-specific seismic hazard analysis for the Project in Volume 2 Section 11.2 Geology, Terrain, and Soils, seismic hazards are:

- The mean earthquake ground motions calculated on the basis of probability
- Described by peak spectral accelerations over a range of vibration periods
- Often characterized by the peak ground acceleration

Peak ground acceleration and peak spectral accelerations are given in terms of a percentage or decimal fraction of the acceleration due to gravity, e.g., 5.4%g or 0.054g.

As described in Volume 1 Section 4 Project Description, the earthquake design ground motions adopted for the dam, generating station, and spillways have an annual exceedance frequency of 1/10,000. In accordance with Canadian Dam Association Dam Safety Guidelines, the performance requirement is that the dam structures subjected to the earthquake design ground motions should perform without uncontrolled release of the reservoir, although severe damage or economic loss may be tolerated. Lower seismic design criteria are used for those parts of the dam, generating station, and spillways where damage or economic loss would be tolerated during the earthquake design ground motions.

The seismic performance requirements for the dam, generating station, and spillways are:

- Operating basis earthquake – the earthquake ground motions with a mean annual exceedance frequency of 1/475. All structures, components, and primary operational and control systems required for the safe operation of the dam, generating station, and spillways will be designed to be capable of operating immediately after the operating basis earthquake.
- Design basis earthquake – the earthquake ground motions with a mean annual exceedance frequency of 1/2,475. All structures, components, and primary operational and control systems required for the safe shutdown of the generating station and for the safe operation of the dam and spillway will be designed to be capable of operating immediately after the design basis earthquake. Any expected damage to structures, components, and primary operational and control systems required for the safe operation of the generating station will be designed to be repairable within 72 hours after the design basis earthquake.
- Earthquake design ground motion – the earthquake ground motions with a mean annual exceedance frequency of 1/10,000. All structures, components, and related mechanical operation and control devices that contribute to retaining the reservoir will be designed such that there is no uncontrolled release of water from the reservoir during or after the earthquake design ground motions, and such that flows past the dam can be safely managed after the earthquake. The earthquake design ground motion is sometimes referred to as the maximum design earthquake.

The computed peak horizontal ground accelerations for the above annual exceedance frequencies from the site-specific seismic hazard analysis described in Volume 2 Section 11.2 Geology, Terrain, and Soils are:

- 1/475: 0.022g
- 1/2,475: 0.087g
- 1/10,000: 0.250g

The National Building Code of Canada uses a probability level of 2% in 50 years to achieve uniform reliability for building design across the country. This probability level is numerically equivalent to an annual exceedance frequency of 1/2,475. The 2010 National Building Code gives peak horizontal ground acceleration for Fort St. John of 0.054g. This value is lower than the 1/2,475-year earthquake from the site-specific seismic hazard analysis mainly because mean hazards are used for dam design and median hazards are used for the National Building Code of Canada.

37.1.6.1.3 ~~37.1.8.1.2~~ Earthfill Dam Performance

There are no known active faults at the dam site; therefore, disruption of the dam by movement on a major fault in the foundation or the loss of freeboard due to differential tectonic ground movements are unlikely.

Site investigations were carried out to enable a liquefaction assessment to be undertaken in general accordance with Seed's approach as recommended by Idriss and Boulanger (2008). The Idriss and Boulanger (2008) methodology is an update on the U.S. National Center for Earthquake Engineering Research guidelines for liquefaction assessment summarized in Youd et al. (2001).

The approach involves the evaluation of the cyclic stress ratio and the cyclic resistance ratio. The cyclic stress ratio is the ratio of earthquake-induced stresses to the effective overburden pressure, and is a function of depth and the earthquake peak ground acceleration. The cyclic resistance ratio is the soil's ability to resist the earthquake-induced stresses. Idriss and Boulanger (2008) proposed an empirical correlation between cyclic resistance ratio and standard penetration test resistances based on case histories of sites that have or have not liquefied during earthquakes. The correlation is standardized based on an M7.5 earthquake, 1 atmosphere overburden pressure, and fines content less than 5%. As a result, corrections for actual fines content, overburden pressure, in situ stresses, and earthquake magnitude have to be applied to the cyclic resistance ratio correlation to make it comparable to cyclic stress ratio for the design earthquake. If the cyclic stress ratio is greater than the cyclic resistance ratio, liquefaction is predicted.

The liquefaction assessment identified some thin discontinuous zones in the riverbed alluvium that could be liquefiable at low confining pressures during the earthquake design ground motions. This means that the dam would not fail, but there could be some slumping at the upstream and downstream toe where the confining pressure is low.

The earthquake design ground motions would not cause any slope failures or failures on weak planes in the foundation but there would be some movements, both vertical settlement and laterally.

Considering the possible ways that an earthquake could damage or cause failure to an earthfill dam described by Sherard and listed in Section 37.1.6.1.1, the following defensive measures have been incorporated into the design of the earthfill dam based on recommendations by Seed (1982) to allow for potential movements:

- Provision of ample freeboard to allow for settlement or slumping
- Use of wide transition zones using material not vulnerable to cracking, (Zone 2A and Zone 2B in Figure 4.14 in Section 4 Project Description)
- Use of chimney drains near the central portion of the embankment, (Zone 2A and Zone 2B in Figure 4.14 in Section 4 Project Description)
- Provision of ample drainage zones to allow to allow for possible flow of water through cracks, (Zone 2A, Zone 2B, and Zone 3 in Figure 4.14 in Section 4 Project Description)
- Use of a wide core zones of plastic materials not vulnerable to cracking, (Zone 1 in Figure 4.14 in Section 4 Project Description)
- Use of a well-graded filter zone upstream of the core to act as a crack stopper, (Zone 2A in Figure 4.14 in Section 4 Project Description)
- Use of fine, more plastic core material placed wet of optimum and compacted by special equipment in thin layers at the abutment contacts
- Widening the filters at the abutment contacts

The following methods were used to estimate the movements of the earthfill dam due to seismic events:

- Lateral displacement using the Newmark (1965) method and the Bray and Travasarou (2007) method
- Crest settlement using the Jansen (1990) method and the Swaisgood (2003) method

In the Newmark (1965) method, a mass of soil is assumed to behave as a rigid block sliding along a well-defined failure surface. A pseudo-static stability analysis is performed on the soil mass to calculate the yield acceleration. The yield acceleration is the acceleration applied to the soil mass that reduces the factor of safety against sliding to 1.0. A relationship is then applied to determine the earthquake-induced displacement from the calculated yield acceleration and the design earthquake parameters of peak ground acceleration and peak velocity. The relationship is based on case histories and model tests in the laboratory.

Bray and Travasarou (2007) is an update on the Newmark method. This method uses a semi-empirical relationship between the earthquake-induced displacements, the yield acceleration of the soil mass, and the peak ground acceleration of the design earthquake. Unlike the Newmark method, the relationship includes the input of the spectral acceleration of the design earthquake and the fundamental period of the soil mass. The relationship was validated through the re-examination of 16 case histories of earth dam and landfill performance.

Jansen (1990) developed an empirical equation for embankment settlement based on the earthquake magnitude, the earthquake acceleration at the crest of the embankment,

and the yield acceleration of the soil mass. The empirical equation was developed from 22 case histories where actual settlements were measured after an earthquake.

Swaisgood (2003) proposed an empirical correlation to estimate the settlement of an embankment dam crest due to an earthquake. Based on the analysis of 70 case histories, Swaisgood determined that the most important factors that appear to affect embankment crest settlement during an earthquake include the peak ground acceleration and the earthquake magnitude. Using these parameters, a mathematical equation and chart were produced to estimate the crest settlements as a percentage of the dam height and alluvium thickness.

These methods are commonly used for the design of dams where no liquefaction is expected to occur.

The analyses using these methods indicated that:

- Little or no displacement of the earthfill dam would be expected for earthquakes up to the design basis earthquake
- For the earthquake design ground motions, deformations are expected to be:
  - A maximum lateral displacement in the order of 7 cm
  - A maximum crest settlement in the order of 5 cm

The above displacements are small, as the earthfill dam would be able to accommodate much greater deformations:

- According to the U.S. Federal Energy Regulatory Commission (FERC 2006), seismic displacements of earthfill dams should normally not exceed 60 cm and should never be greater than one-half the fine filter width (Zone 2A in Figure 4.14 in Section 4 Project Description, which would be 5 m wide)
- Also, the estimated settlement of the crest of the dam is about 1% of the earthfill dam freeboard of 7.6 m (see Table 37.2)

Cracking due to these small movements is unlikely, and the fine filters upstream and downstream of the core would be designed to protect against piping.

As described in Volume 2 Section 11.2 Geology, Terrain, and Soils, seismic seiche would be less than 0.45 m. If a seiche was caused by the earthquake design ground motions, the combined seiche and crest settlement would be 0.5 m compared to the freeboard of 7.6 m.

Landslide-generated waves are discussed under Section 37.1.10.3.

The earthfill dam would meet the performance requirements.

37.1.6.1.4 ~~37.1.8.1.3~~ North Bank Performance

Deep-seated movements of the north bank due to the existing unstable slopes could adversely affect the abutment of the earthfill dam. The existing unstable material above the earthfill dam would be excavated to flatten the slope and prevent deep-seated movements.

Potential movements of the north bank during earthquakes were determined using the Bray and Travasarou (2007) method.

No movements of the north bank slope above the earthfill dam are expected for peak ground accelerations up to 0.06g (annual exceedance frequency of approximately 1/750).

For the design basis earthquake, movements in the order of 1 cm to 3 cm are expected for surfaces up to 15 m deep in the north bank overburden slope using the Bray and Travasarou (2007) method. No movements are expected for deeper surfaces.

For the earthquake design ground motions, the expected maximum displacements are:

- About 30 cm for shallow localized surfaces within about 3 m of the surface of the slope
- About 10 to 20 cm for surfaces 10 to 15 m, below the surface of the slope
- About 1 cm for surfaces 30 to 50 m below the surface of the slope

The shallower movements “represent little more than superficial slippage” (Jansen 1990) and is only half of the 60 cm displacement considered permissible by the Federal Energy Regulatory Commission (FERC 2006) for embankment dam slopes.

The expected movements during the maximum design earthquake could cause some damage to drainage works on the slope, which would require subsequent repair and replacement.

Some damage to the access road on the north bank slope could occur, but it is expected that, following some minor repairs, the road would be usable by four-wheel drive vehicles.

Movements of the slope above the earthfill dam would have no dam safety implications; therefore, the north bank would meet the performance requirements.

37.1.6.1.5 ~~37.1.8.1.4~~ RCC Buttress Performance

Based on the performance of existing dams subjected to major earthquakes described in Section ~~36~~37.1.6.1.1, major earthquakes could adversely affect the RCC buttress by causing:

- Cracking
- Increased foundation seepage
- Movements

Analysis of the buttress sections using the Fast Lagrangian Analysis of Continua (FLAC), which is an advanced two-dimensional computer program for geotechnical analysis of rock, soil, and structural support, indicated that there would be no permanent displacements for the design basis earthquake.

Due to the geometry of the buttress and the complex interaction of the structure and bedrock, time history seismic analyses were undertaken for the maximum design earthquake. In a time history analysis, earthquake records of ground acceleration versus time from actual earthquakes are scaled to the required peak ground acceleration and used to calculate the dynamic response of a structure or system.

To illustrate what an earthquake time history looks like, Figure 37.2 shows the horizontal components of the 1994 Northbridge earthquake scaled to 0.25 g. More details of this

earthquake scaled to 0.25g are given in Table 37.3. The Arias intensity shown in Figure 37.2 and Table 37.3 is a measure of the strength of ground motion. Arias intensity, which is determined by measuring the acceleration of transient seismic waves, has been found to be a fairly reliable parameter to describe the earthquake shaking necessary to trigger ground movements.

Based on the site-specific seismic hazard analyses described in Volume 2 Section 11.2 Geology, Terrain, and Soils, the parameters used for the selection and scaling of recorded earthquake time histories included:

- Similarity between scaled response spectrum to the response spectrum of the earthquake design ground motion for the range of vibration periods of the structures
- Earthquake magnitudes
- Epicentral distance
- Type of fault that caused the earthquake: reverse and strike slip faults
- Arias intensity 0.5 to 0.7
- Effective duration 10 to 14 seconds
- Peak ground accelerations, velocity, and displacement of the scaled time history

The largest earthquake recorded near the dam site is the M_W5.4 Dawson Creek earthquake, which occurred at about 70 km to the southeast of the dam site on April 14, 2001. The location of the Project, its tectonic setting, the presence of embedded faults, the recorded seismicity around the Project, and the results of the seismic studies suggest that the earthquakes contributing to the earthquake design ground motion cannot be defined by a limited combination of magnitudes and distances. The representative time histories needed to be selected from earthquakes with wide-ranging magnitudes (from M_W5.5 to M_W7.0) and wide-ranging distances from the recording seismograph (10 km to 150 km).

Table 37.3 summarizes the eight scaled earthquake records that were selected for the dynamic analysis based on the parameters listed above.

Two-dimensional analyses of the dam, powerhouse, and spillway buttresses were undertaken in Adina Structures, a finite element analysis program, to capture the interaction between each RCC buttress, the rock, and reservoir. This enabled the displacements and stresses in the RCC buttress sections and the rock to be examined for the earthquake design ground motion.

Table 37.3 Scaled Earthquake Records

Earthquake	Year	Magnitude	Distance (km)	Peak Ground Acceleration	Arias Intensity (m/s)	Effective Duration (sec)
Northridge	1994	6.7	23	0.25	0.40	10
San Fernando	1971	6.6	24	0.25	0.43	13
Coyote Lake	1979	5.7	13	0.24	0.26	6
Hector Mine	1999	7.1	69	0.23	0.69	17
Loma Prieta	1989	6.9	35	0.24	0.39	15
Nahanni	1985	6.8	22	0.24	0.58	8
Virginia	2011	5.8	122	0.23	0.38	10
Virginia	2011	5.8	54	0.23	0.69	12

The dynamic response to the earthquake design ground motion was analyzed by applying the horizontal and vertical accelerations of the eight time histories listed in Table 37.3. Based on these analyses, the earthquake design ground motion would cause:

- A permanent displacement of the RCC buttresses in the order of 1 cm
- Minor horizontal cracking
- The axis of the generating units to rotate about 0.5 mm over the height of the unit

The movements and cracking during the earthquake design ground motion would not impair the ability of the RCC buttress to retain the reservoir, and any damage to the generating units would be minor.

As described in Volume 1 Section 4 Project Description, seepage into the bedrock foundation of the buttress would be limited by the approach channel lining, and the drainage system would limit the groundwater pressure acting on the buttress. The approach channel lining and the drainage system would be designed to be functional after the deformations expected to occur under all loading conditions, including the earthquake design ground motion.

Nevertheless, the buttress has been designed to withstand the following severe loading condition, which represents a worst-case groundwater condition:

- Full reservoir hydrostatic pressure acting on a near vertical plane from the base of the approach channel to the foundation of the buttress
- Uplift from full hydrostatic at the base of the near vertical plane to tailwater level at the downstream limit of the buttress

This analysis demonstrated that the RCC buttress would retain the reservoir, whatever damage occurred to the approach channel lining and drainage system.

Therefore, it can be concluded that the buttress would meet the performance requirements.

37.1.6.1.6 ~~37.1.8.1.5~~ Other Reservoir Retaining Structures

The other reservoir retaining structures (the power intakes, spillway headworks, auxiliary spillway, and associated training walls) would be founded on the buttress. These structures would be designed to meet the performance requirements given above, and would not move relative to the buttress.

While the reservoir retaining structures would safely retain the reservoir after the maximum design earthquake, it is possible that the generating units and spillway gates would not be operational for a period. With none of the generating units and spillway gates operational, the Project could pass a total flow of 4,000 m^3/s through the auxiliary spillway and by overtopping of the spillway gates with the reservoir at the maximum flood level. This total flow is equivalent to Peace Canyon with all four units operating at full capacity plus the 20 return period flood from the Site C local catchment.

Therefore, the performance of the other reservoir retaining structures would meet the performance requirements even if the generating units and spillway gates were not operational after the earthquake design ground motion.

37.1.6.1.7 ~~37.1.8.1.6~~ Post-Earthquake Monitoring

In accordance with BC Hydro's emergency planning guide if a seismic event occurs within a region, an inspection of the dam, structures, and generating facilities must be conducted.

Information regarding the intensity of shaking would be available from seismic instrumentation located at or near the dam site and from the Pacific Geoscience Centre. The response would depend on the intensity of the seismic shaking.

A low-intensity earthquake is felt by only a few people in the local region, in which case an inspection, during daylight hours, would be required at the earliest opportunity (i.e., within 12 to 18 hours). A more thorough follow-up inspection may also be required by the Dam Safety Engineer.

A higher intensity earthquake, where the shaking is longer than a few seconds and is felt by nearly everyone in the region, would require an immediate inspection. A more thorough follow-up inspection, during daylight hours, would be required at the earliest opportunity (i.e., within 12 to 18 hours).

If, following these procedures, a dam safety hazard is identified, then response procedures would be implemented as outlined in the Emergency Preparedness Plan.

The Operation Maintenance and Surveillance Manual described in Volume 1 Section 4 Project Description would contain requirements for increased surveillance after any intense earthquake.

37.1.6.2 ~~37.1.8.2~~ Transmission Line and Substation

Earthquake shaking could damage towers or lines, causing a power outage.

The towers and foundations would be designed for earthquake ground motions with annual exceedance frequency of 1/2,475.

Typically loads due to extreme weather, not seismic forces, govern the design of lines and structures.

37.1.6.3 ~~37.1.8.3~~ Highway 29 Realignment and Other Roads

The realigned segments of Highway 29, which would form part of the provincial highway system, and other permanent roads, would be designed according to the BCMoTI seismic standards. ~~In accordance with these standards, bridges would be designed to withstand earthquakes with a return period of 50 years.~~

37.1.6.4 ~~37.1.8.4~~ Hudson's Hope Shoreline Protection

In accordance with the Association of Professional Engineers and Geoscientists of B.C.'s guidelines for landslide assessments for proposed residential developments in B.C., the Hudson's Hope shoreline protection would be designed for the earthquake with an annual exceedance frequency of 1/2,475 given in the National Building Code of Canada.

37.1.7 ~~37.1.9~~ Wildfire

Wildfires can pose a hazard to Project components including the dam, generating, station and spillways, transmission line and substation, and roads.

Wildfires result either from anthropogenic causes such as campfires, or from natural causes such as lightning. Factors that influence the probability and magnitude of wildfires include: vegetation type, ignition sources, and weather conditions, including forest moisture, precipitation, wind speed, and direction, as well as the relative humidity and air temperatures. On average, approximately 9,100 forest fires occur annually in Canada. Of these, over 60% result from anthropogenic causes, most by accident, and up to 35% of these forest fires result from lightning (The Canadian Encyclopedia 2012).

The 2009 Natural Resources Canada fire danger rating at and surrounding the Project area, with the exception of the area around Hudson's Hope, was classified as high. The area surrounding Hudson's Hope was classified as very high (Figure 37.3). Historical fires greater than 200 hectares that occurred in the Project area from 1959 through 1999 typically resulted from anthropogenic activities (Figure 37.4).

The dam site would be cleared during construction and a vegetation-free buffer would be maintained around the dam, generating station, and spillways to provide firebreaks.

The transmission line right-of-way would be cleared during construction and maintained through a vegetation management plan, which would provide a firebreak. In addition, the towers would be constructed of steel and thus would be less susceptible to fire. A firebreak would also be maintained around the substation.

37.1.8 ~~37.1.10~~ Floods During Construction

The following stages of construction of the dam, generating station, and spillways are described in Volume 1 Section 4 Project Description:

- Stage 1 – river channelization
- Stage 2 – river diversion

During river channelization, the river would be confined to its main channel by the north and south bank Stage 1 cofferdams.

During river diversion, the river would be diverted through two tunnels by the Stage 2 cofferdams to allow construction of the earthfill dam across the main river channel.

Floods during construction would have an adverse effect on the Project if the cofferdams were overtopped during either of these stages.

Studies to determine the magnitude of floods at the dam site during construction of the Project considered the inflows from the Site C local catchment and discharges from Peace Canyon Dam and separately. These are described in more detail below.

37.1.8.1 ~~37.1.10.1~~ Site C Local Catchment Floods

Flood frequency analysis was performed using annual peak flows for the Site C local catchment, which were obtained by subtracting flows measured at the Hudson's Hope gauge from flows measured at the Site C gauge. Refer to Section 37.1.2.2 and Volume 2 Section 11.4 Surface Water Regime for more information on these gauges. A 12-hour shift was applied to the records from the Hudson's Hope gauge to allow for the travel time between the two gauges.

Flow data were available for the period 1980 to 2012. Flows for 1986 were excluded due to missing records. All flows were increased by 2.9% to account for inflows from the drainage area between the Hudson's Hope gauge and Peace Canyon Dam, which is 2.9% of the Site C local catchment area.

The peak flows for each year were analyzed assuming a log-normal distribution. Table 37.4 presents the results of the analysis.

Table 37.4 Flood Frequency Analysis Results

Return Period (Years)	Mean Daily Flow (m^3/s)
2	887
5	1,453
10	1,905
20	2,382
50	3,062
100	3,620
200	3,886

The peak daily discharge of the largest flood from the Site C local catchment for the period 1980 to 2012 was 3,177 m^3/s in June 2001. The return period of this flood is approximately 60 years.

Table 37.5 shows some recent precedents for the selection of floods used for the design of cofferdams and diversion works for the construction of large dams.

The June 2001 flood was initially selected for the design of the cofferdams and the diversion tunnels based on Canadian precedent and the two-year period before the elevation of the earthfill dam would be higher than the Stage 2 upstream cofferdam.

Table 37.5 Precedents (Current Canadian Practice) for the Selection of Floods for the Design of Diversion Works and Cofferdams

Project	Design flood risk
Hydro Quebec	5% overall risk – 40-year flood for two-year exposure period, e.g., SM3, Romaine 2, Eastmain 1 & Peribonka
Manitoba Hydro	Conawapa 20-year flood & Wuskwatim 50-year flood
SaskPower	5% overall risk – 40-year flood for two-year exposure period, Fort-a-la-Corne dam
Nalcor	5% overall risk – 40-year flood for two-year exposure at Muskrat Falls and Gull Island + 3 m cofferdam freeboard to reduce risk

Figure 37.5 shows the hydrograph shapes for the four largest annual floods in the period 1980 to 2012 normalized by the ratio of peak daily discharge to maximum daily discharge. The June 2001 flood hydrograph shape was selected as being representative of the large floods. The flood hazard period, i.e., the time of year when the largest floods occur, is late May to early July.

37.1.8.2 ~~37.1.10.2~~ Stage 1 – River Channelization

The governing condition for the crest elevation of the Stage 1 cofferdams would be the maximum ice levels that would exist at the dam site if the ice front reached the dam site (see Section 37.1.3.1.3).

During the spring, one of the four generating units at Peace Canyon Dam is usually out of service for maintenance, which limits the maximum discharge to 1,487 m^3/s. However, due to low generation requirements in the spring, discharges are typically lower.

With the 100-year flood from the Site C local catchment combined with a discharge of 1,487 m^3/s from Peace Canyon Dam, the freeboard on the Stage 1 cofferdams would be 4 m. Based on 39 years of inflow data, the probability of the Peace Canyon Dam discharge exceeding 1,487 m^3/s would be 20%. Therefore, the annual probability of overtopping of the Stage 1 cofferdams is much less than 1/500.

37.1.8.3 ~~37.1.10.3~~ Stage 2 – River Diversion

Williston reservoir has inter-year storage capability and the reservoir would be operated during construction to:

- Limit normal inflows to Site C to less than the diversion tunnel capacity
- Provide a storage reservation to allow discharges to be reduced during the passage of floods from the Site C local catchment

Daily operation of Williston Reservoir was modelled for the Stage 2 diversion period using operations models that incorporated 39 years of historic inflow sequences and provided daily output. The models were used to optimize Williston Reservoir operation with a fixed storage reservation with pre-generation and pre-spill as required to meet the downstream discharge constraints under all 39 inflow sequences.

The discharge constraints were:

- Releasing a total flow of 600 m^3/s at Site C during construction of the upstream closure section

- Ramping up from 600 m^3/s to the diversion tunnel capacity during completion of the Stage 2 upstream cofferdam

See Section 4 Project Description for details of the river diversion activities.

The initial design of the Stage 2 diversion works (tunnels and cofferdams) was based on the following assumptions:

- A repeat of the June 2001 Site C local catchment flood
- Peace Canyon Dam discharges:
  - 1,487 m^3/s prior to flood arrival
  - Reduced to the minimum discharge of 283 m^3/s when the flood arrives at the dam site
  - Restored to 1,487 m^3/s after flood passage

A storage reservation in the Williston Reservoir of 0.56 m would be adopted to enable the above reduction in discharges from Peace Canyon Dam.

Figure 37.6 shows routing of the June 2001 flood through the diversion tunnels. The figure shows:

- Inflow from the Site C local catchment
- Peace Canyon Dam discharges
- The headpond (pool level) upstream of the diversion tunnels
- The discharge from the diversion tunnels

Note that the discharge from the diversion tunnels would be less than the inflow because water would be stored in the headpond.

A risk assessment of a flood from the Site C local catchment exceeding the June 2001 flood, and damaging or overtopping the Stage 2 cofferdam was undertaken. Based on this risk assessment, BC Hydro decided to increase the capacity of the diversion works by increasing the diameter of the diversion tunnels and by constructing the Stage 2 cofferdam to the maximum practical height.

With these changes, the risk assessment demonstrated that, with the Peace Canyon Dam discharges as listed above, the annual exceedance frequency of a flood that would reduce the freeboard to less than the required 0.9 m and thus potentially damaging or overtopping the Stage 2 upstream cofferdam would be 1/120. The operations modelling using 39 years of inflow data indicated a 20% probability of the Peace Canyon Dam discharge exceeding 1,487 m^3/s. Therefore, the annual exceedance frequency of a flood potentially damaging or overtopping the Stage 2 upstream cofferdam is approximately 1/600.

37.1.8.4 ~~37.1.10.4~~ Highway 29 Realignment and Other Roads

In accordance with BCMoTI, the Highway 29 realignment bridges would be designed for the following floods:

- 20-year return period for the two-year construction period

- 200-year peak discharge return period for the operation of the bridge

37.1.9 ~~37.1.11~~ Floods During Operation

As described in Volume 1 Section 4 Project Description, the Inflow Design Flood for the Project would be the probable maximum flood, which is defined as the most severe flood that may reasonably be expected to occur at a particular location (CDA 2007a). The probable maximum flood is evaluated by deterministic methods that maximize the various factors contributing to the generation of a flood. The probable maximum flood would be caused by hydrometeorological conditions (e.g., snowmelt and rainfall) that maximize soil moisture and thus maximize runoff.

The probable maximum flood at Site C would consist of regulated discharges from the W.A.C. Bennett Dam combined with unregulated inflows from the Site C local catchment. The studies to determine the probable maximum flood at Site C thus considered conditions in the Williston and Site C local catchments. In this section, the Site C local catchment refers to the catchment between the W.A.C. Bennett Dam and Site C, and the Site C catchment means the total catchment upstream of Site C, i.e., the Williston catchment plus the Site C local catchment.

The Williston and Site C local catchments are located in northeast British Columbia, approximately between latitudes 54° and 59° north and between longitudes 121° and 127° west. The catchments are described in Volume 2 Section 11.4 Surface Water Regime.

The prevailing west-east atmospheric circulation limits the arrival of moisture from the Atlantic or Gulf of Mexico, and moist air from the Pacific loses most of its moisture crossing the mountains. As a result, away from the mountains and foothills, the average annual precipitation ranges between 300 and 500 mm. However, this does not eliminate the risk of severe flooding. Most of the major river systems originate in areas where snow cover remains into May and June because of the high elevations and cool climate. The late spring snowmelt in the mountains corresponds with the June peak in monthly rainfall, creating the risk of combined snowmelt and rainfall flooding. Extensive heavy rainfall in the May through July period is usually the result of long-lived “cold lows” that persist in the foothills region for several days and draw moist Gulf of Mexico air into the area, causing prolonged heavy rainfall due to the presence of the low and upslope winds in the foothills and mountains. With or without contributions from snowmelt, these cold lows are the most common cause of flooding along the rivers flowing eastward. Away from major rivers in the eastern part of British Columbia, severe local thunderstorms are more likely to be the meteorological event associated with flooding.

The methods used to determine the probable maximum precipitation are described in Section 37.1.9.1. Determination of the probable maximum flood is described in Section 37.1.9.2.

37.1.9.1 ~~37.1.11.1~~ Probable Maximum Precipitation

37.1.9.1.1 ~~37.1.11.1.1~~ Methodology

Probable maximum precipitation is “theoretically the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular

geographic location at a given time of year", as defined in Hydrometeorological Report No. 55A (Hansen et al. 1988), which is referred to in this section as HMR-55A.

The methodology for determining the probable maximum precipitation consisted of:

- Identification of historical storms
- Maximizing the storms based upon the maximum moisture likely to be available to them
- Transposing the storms to the Site C catchment

In mountainous terrain, rainfall amounts for historic storms have significant contributions from local orographic effects that are not directly transposable to other regions. Restricting storm samples to those occurring directly over the study basin would not provide a large enough sample of severe storms to ensure that a storm of maximum efficiency has been identified, leading to underestimation of the probable maximum precipitation. To overcome this deficiency, a technique referred to as storm separation was used as recommended in HMR-55A and by the Canadian Dam Association (CDA 2007b). In this method, the precipitation from a storm occurring over an orographic region is considered to consist of two components:

- A convergence component, which is the part of the precipitation not due to terrain features
- An orographic component, which is caused by relief

The storm separation method used to determine the probable maximum precipitation is described in Section 37.1.9.1.3.

37.1.9.1.2 ~~37.1.11.1.2~~ Climate Data

Climate data were obtained for the catchment upstream of Site C and a large surrounding area. The larger area was needed to augment the database required for the analysis of the climate in the region. Climate data collected at the climate stations shown in Figure 37.1 were used for determining the probable maximum precipitation.

Canadian precipitation and temperature data were obtained from Environment Canada, Alberta Environment, and BC Hydro data collection platform (DCP) stations. Data for Montana and Idaho stations were obtained from the Western Regional Climate Center in Reno, Nevada. Daily precipitation records were obtained for all stations shown in Figure 37.1. Hourly data were obtained where available.

Dew point data were also obtained because there is a good relationship between dew point temperature and the amount of moisture in storms, particularly in the lowest layers of the atmosphere.

The good relationship between dew point measurement and moisture content is valid for summer and fall storms. However, direct precipitable water measurements collected from upper air balloons must be used for estimating storm moisture for spring storms. Upper air data were obtained from the National Climatic Data Center (NCDC 1997) for the period 1946 to 1997. More recent data were obtained from the U.S. National Oceanic and Atmospheric Administration Forecast Systems Lab.

The nearest radiosonde sites to the study area are Edmonton, Alberta and Glasgow, Montana in the U.S. Radiosonde data provide direct and more accurate measurements of atmospheric moisture at the measurement location. Assessment of spatial variation of maximum atmospheric moisture for spring months was aided by the use of a dew point analysis (Hopkinson 1999).

A total of 686 daily climate stations with a minimum of 20 years of records were examined, and 458 were included in the analysis. The reduction in the number of stations was due to several factors, including rejection of stations with obvious data problems such as apparent partial year records and the presence of outliers that could not be rationalized.

37.1.9.1.3 ~~37.1.11.1.3~~ Storm Separation Method

The steps used in the storm separation method were:

1. Create a digital GIS coverage of 100-year 24-hour total precipitation for the study area using station estimates of 100-year rainfall and relying on spatial patterns of season rainfall developed using PRISM, a statistical model (Daly et al. 1994; Daly et al. 2002).
2. Create a digital GIS coverage of 100-year 24-hour convergence precipitation for the study area by assuming that rainfall in the flat plains area of Alberta was unaffected by orographic enhancement.
3. Create a GIS coverage of T/C (100-year total precipitation “T” divided by 100-year convergence-only precipitation “C”) to determine the orographic component of precipitation.
4. Identify relevant historical storms during summer months (the largest rain storms) and spring (lower rainfall amounts than in summer, but a higher potential for rain-on-snow events).
5. Determine the ratio of observed precipitation to 100-year precipitation for each storm and select the storms with the largest ratios to identify the most intense recorded storm rainfalls.
6. Determine the convergence component of each selected storm using the T/C coverage.
7. Determine the observed precipitable water content for each storm and create a GIS coverage of maximum precipitable water available over the whole study area.
8. Maximize storms by multiplying the observed convergence precipitation by the ratio of maximum to observed precipitable water.
9. Transpose the maximized storms from their storm centres to the basin area by multiplying the maximized convergence precipitation by the ratio of maximum precipitable water on the basin under study to that at the storm location.
10. Develop depth-area-duration curves for the transposed and maximized convergence storms.
11. Convert convergence-probable maximum precipitation to total probable maximum precipitation using T/C ratios.

12. Map total probable maximum precipitation (PMP).

37.1.9.1.4 ~~37.1.11.1.4~~ 100-Year 24-Hour Precipitation Map

A grid of T/C ratios (100-year total precipitation "T" divided by 100-year convergence precipitation "C") is needed to account for the effect of orography on precipitation. The first step in deriving the T/C ratio is to create a GIS coverage of the 100-year 24-hour total precipitation.

The 100-year precipitation values were obtained by statistically analyzing hourly and daily rainfall data for all stations in the study area with more than 20 years of records. Considering the limited number of values in the samples and for homogeneity, a two-parameter distribution was assumed (Extreme value Type 1, also known as the Gumbel distribution, with parameters estimated by the method of moments). Daily values were adjusted by the 1.13 factor to convert once/day readings to estimates of 24-hour maxima (Hershfield 1961). Each station's 100-year value was compared to gridded monthly values for June from precipitation "normals" coverages developed using the PRISM model (Daly et al. 1994; Daly et al. 2002). The inherent assumption is that precipitation patterns in extreme storms are similar to mean precipitation patterns and that the month of June was selected because of the more frequent occurrence of large storms in that month.

The ratios of 100-year 24-hour precipitation to monthly averages were distributed using inverse distance weighting with GIS software (ArcView), producing a gridded coverage of these ratios for the study area. This coverage was then multiplied by the monthly normals grid to obtain a gridded 100-year 24-hour rainfall coverage consistent with orographic precipitation enhancement, as identified by the PRISM mapping of monthly normal precipitation. The resulting map of 100-year 24-hour precipitation is shown in Figure 37.7.

37.1.9.1.5 ~~37.1.11.1.5~~ 100-Year 24-Hour Convergence Precipitation Map

The convergence component of probable maximum precipitation is often referred to as "free atmosphere forced precipitation" and is a key component of the Storm Separation Method. According to HMR-55A, the key to the convergence analysis is finding areas where the effects of topography are minimal or absent. This was done by estimating precipitation in "least orographic" areas (well east of the Rockies, for example) and by assuming that precipitation in those areas was 100% convergence related.

Areas in homogeneous terrain were selected first. The relationship between summer precipitation and maximum persisting dew point was found to be quite strong. By applying this relationship to estimated maximum dew points in areas with complex terrain, convergence values in those areas could be calculated. The latter step was carried out by manually, extending in the orographic areas the isocontour lines of the 100-year convergence precipitation along the lines of potential maximum dew point. This produced coverages with smooth contours that generally were aligned with lines of potential moisture (such as maximum persisting dew point). The resulting map of 100-year convergence-only precipitation is shown in Figure 37.8.

37.1.9.1.6 ~~37.1.11.1.6~~ Development of T/C Map

The T/C coverage was obtained by dividing the 100-year 24-hour total precipitation grid (Figure 37.7) by the 100-year 24-hour convergence grid (Figure 37.8). The resulting T/C map is presented in the upper part of Figure 37.9. The lower part of the figure, which covers the state of Montana, was derived based on the 100-year 24-hour map from the U.S. National Oceanic and Atmospheric Administration Atlas No. 2 "Precipitation-Frequency Atlas of the Western United States" Volume 1 – Montana.

The magnitude of the convergence precipitation was estimated from the values on the open plains, decreasing gradually in a westerly direction following the procedure described in HMR-55A. The resulting contours were smoothed relative to the original 100-year map and the final T/C map was obtained by dividing the 100-year 24-hour map by the convergence coverage.

37.1.9.1.7 ~~37.1.11.1.7~~ Historical Storm Analysis

Storm selection consisted of the following steps:

1. Begin with a list of the largest storms in British Columbia, Alberta, Saskatchewan, and Montana.
2. Collect all available daily precipitation data for those events.
3. Calculate the maximum 1-, 2-, and 3-day total precipitation for each storm for all available stations.
4. Multiply the 1-day values by 1.13 to convert daily values to 24-hour estimates.
5. Create spatial coverages of storm precipitation for the largest storms (based on ratios to 100-year values).
6. Select the most extreme storms according to the magnitude of normalized precipitation (observed divided by 100-year) and area of influence.

A few other important storms were identified by scanning rainfall data over a few consecutive days at several stations in the region defined in step 1 above.

A list comprising 39 storms was selected for the study. Two sets of storms were considered, namely summer (primarily June–July) and spring (April–May).

The largest storm was the Gibson Dam storm of June 7, 1964, where 17% of the 100-year precipitation was measured at the storm centre.

Knowledge of the approximate path of a storm assists in identifying the amount of moisture in the air mass that created the storm. Surface dew point data are obtainable from relatively few locations, but by choosing one upwind of the storm centre and relatively near the path of the storm, the dew point of the air mass can be estimated.

Three sources of data are available for information regarding the path of storms that occurred in recent decades:

1. Upper-air maps are available from a variety of sources. These display wind speed, wind direction, and other information. In general, these have been available since the mid-1940s.

2. The hysplit model (ARL 2007) and Canada's CMC trajectory model (Côté et al. 1998) form complete systems for computing simple air parcel trajectories to complex dispersion and deposition simulations. Both models provide an estimate of the path that air parcels followed as they moved toward a particular location (in this case, a storm centre).
3. Balloon radiosonde data are available for selected locations, generally twice per day. These include reports of winds and other parameters at various levels in the atmosphere.

Prior to the 1940s, none of these types of data were available. For older storms, it was assumed that flow was from the southeast quadrant in summer. This is consistent with HMR-55A's discussions of general flow patterns.

Dew point temperature data were used to determine the moisture content for summer storms. The storm path information was used to select a dew point observing station. For each station, the maximum persisting 12-hour dew point was determined using reported hourly data. The value considered most representative for the storm, based on the storm trajectory, was chosen as the "storm representative dew point".

37.1.9.1.8 ~~37.1.11.1.8~~ Storm Maximization

It is customary in calculating probable maximum precipitation to employ surface dew point observations to estimate atmospheric moisture. Typically, the maximum 12-hour persisting dew point is used as a surrogate for precipitable water, which represents total moisture available for precipitation. Precipitable water is calculated by assuming a saturated atmosphere from the surface upward. A saturated atmosphere is a situation in which the dew point equals the air temperature: that is, relative humidity is 100%.

However, the assumption of complete saturation over the whole air column is a very conservative one that tends to overstate precipitable water (Chen and Bradley 2006).

Nonetheless, for consistency with other probable maximum precipitation studies (including all those published by the National Weather Service), dew point-based precipitable water calculations were performed.

Maximum 12-hour persisting 1,000 millibar (mb) dew points were obtained from a coverage based on Hopkinson (1999). These were compared to the "storm representative dew point" values, adjusted to 1,000 mb. Each dew point was converted to precipitable water using the approach from HMR-55A. The ratio of maximum to storm representative precipitable water was calculated (maximization ratio).

37.1.9.1.9 ~~37.1.11.1.9~~ Storm Transposition

GIS was used to transpose maximum free atmosphere forced precipitation from each storm horizontally using variations in 100-year precipitable water. The free atmosphere forced precipitation value at any given transposed location was the original (storm centre) value multiplied by the ratio of the maximum precipitable water at the transposed location to the maximum precipitable water at the storm centre. Ideally, the transposed free atmosphere forced precipitation should actually be the ratio of precipitable water in the moist air inflow to the storm compared to the maximum precipitable water at an identical location in a relative sense for the transposed storm. The ratios will be the

same as those for storm centre ratios, provided the gradients of precipitable water are similar in both locations.

37.1.9.1.10 ~~37.1.11.1.10~~ Dominant Storms

Depth-area-duration curves were developed for the convergence PMP using the maximized and transposed grids of all convergence summer/fall and spring storms. Convergence probable maximum precipitation depth-area-duration curves were then used to identify the storms most likely to generate the maximum precipitation over the Site C catchment. The dominant storms identified were those whose depth-area-duration curves are on or close to the envelope of the depth-area-duration curves.

37.1.9.1.11 ~~37.1.11.1.11~~ Probable Maximum Precipitation

Transposition of the dominant storms provided the convergence precipitation. The effect of orography was taken into account by multiplying convergence precipitation by the T/C ratios.

As detailed in HMR-55A, an adjustment to T/C is normally used prior to the final conversion of convergence to total precipitation. The “storm intensification factor” is intended to account for the reduction in orographic effects when convective events dominate. The data required to evaluate the storm intensification factor with confidence were found to be insufficient at the latitude of Williston or Site C; therefore, it was not used, which is conservative because not including this factor yields higher total precipitation.

Since the total precipitation on a catchment will vary with the location of the storm centre and storm orientation, several potential storm centres and orientations were tested for each dominant storm. This was done by developing software that automatically computed the three-day rainfall over the catchments for a large number of storm centres and for changes in storm orientation up to 40°, either clockwise or counter-clockwise. The latter limit was set after looking at the orientation of all storms included in the study. It was found that the storms are generally orientated according along a south-north line, with variations on each side by a maximum of about 40°.

Table 37.6 presents the catchment average precipitations for the dominant storms.

The probable maximum precipitations would be produced by the following storms, which are shown in bold in Table 37.6 below:

- In spring. by maximization and transposition of a storm that occurred in northeastern Alberta (52.17° N, 113.52° W) on May 1, 1964
- In summer, by maximization and transposition of the Gibson Dam storm that occurred in northern Montana (47.60° N, 112.75° W) on July 6, 1964

Table 37.6 Application of the Dominating Storms Over the Basins

Maximum Positioning Search Summary					
Tested Storms	**Williston**		**Site C**		**Observations**
	3 Day Total Precipitation (mm)	**Orientation**	**3 Day Total Precipitation (mm)**	**Orientation**	
Summer/Fall					
1944-06-13	152.2	Right 20°	186.7	Right 30°	
1960-09-04	129.7	Left 40°	153.6	Left 20°	
1962-06-03	147.6	Left 10°	178.2	Left 20°	
1963-07-21	144.5	Right 10°	185.0	Left 30°	
1964-06-07	**155.0**	Right 30°	**232.7**	Left 20°	Summer / Fall PMP – Gibson Dam Storm [a]
1969-08-03	142.6	Left 40°	198.5	Left 40°	
1970-06-27	140.3	Right 20°	184.3	Right 30°	
1973-08-04	145.2	Left 30°	186.1	Left 40°	
1990-06-11	144.7	Left 40°	190.4	Left 40°	
1995-06-05	113.3	Right 30°	159.8	Right 30°	
Spring					
1932-04-22	90.4	Left 20°	118.5	Left 30°	
1964-05-01	**109.1**	Right 40°	**143.5**	Left 30°	Spring PMP
1967-04-28	106.4	Right 40°	129.7	Left 40°	
1974-04-26	105.9	Original	137.1	Right 30°	

NOTE:
[a] The Gibson Dam storm lasted 36 hours, and 88% of the rain fell during the first 24 hours

Figure 37.10 and Figure 37.11 show the probable maximum precipitation over the Williston and Site C local catchments, respectively. These figures also show the boundaries of the sub-basins in the two catchments.

The summer probable maximum precipitation estimates apply primarily to mid-July. If snowmelt is delayed by unusual cold temperatures in April and May, the probable maximum flood could occur in June. Some reduction factors may then be required to adjust the summer probable maximum precipitation intensity to account for the generally lower temperatures in June.

The World Meteorological Office recommends including hydrometeorological factors such as the monthly distribution of maximum persisting dew point, the monthly distribution of 50-year return period 24-hour precipitation, or the monthly distribution of mean wind in the analysis of seasonal variation of probable maximum precipitation.

The hydrometeorological factors considered included:

- Wind, which often amplifies storm rainfall in mountainous regions, with higher winds resulting in higher precipitation. Data show that the mean wind speed over the Peace River basin decreases during the summer months.
- Maximum persisting dew point, which determines the moisture content of storms. Rainfall increases with increasing maximum persisting dew point, which normally

increases through the spring and is highest in the second half of July, and then decreases in the succeeding months.

For Site C, the distribution of the maximum persisting dew point and the distribution of maximum winds trend in opposite direction; therefore, it was decided not to apply a reduction factor to derive the June probable maximum precipitation from the mid-July probable maximum precipitation, which is conservative.

37.1.9.2 ~~37.1.11.2~~ Probable Maximum Flood

37.1.9.2.1 ~~37.1.11.2.1~~ Scenarios

The probable maximum flood results from the combination of a severe hydrometeorological event preceded by, or followed by, an extreme event.

As described in the CDA Guidelines (CDA 2007b), the severe event generally has a return period of about 100 years. The extreme event corresponds to the largest physically possible hydrometeorological event and is estimated by deterministic methods.

The CDA Guidelines recognize three probable maximum flood scenarios applicable to Canada:

1. Melting of the 100-year return period snowpack by a critical temperature sequence (severe event) followed by the spring probable maximum precipitation (extreme event).
2. Melting of the probable maximum snow accumulation by a critical temperature sequence (extreme event) followed by the 100-year return period spring rainfall (severe event).
3. The 100-year return period summer or fall rainfall (severe event) followed by the summer or fall probable maximum precipitation (extreme event).

A 15-day critical temperature sequence is normally sufficient to melt the entire snow cover in the boreal regions of Canada (i.e., most of the territory east of the Rocky Mountains) (Chow and Jones 1994).

Due to the large variations in altitude in the Site C catchment, it may take more than 15 days to melt the snowpack. With snowmelt starting in early May, a summer probable maximum precipitation occurring in June could be combined with melting of the 100-year snowpack. Therefore, the following scenario was included:

4. Melting of the 100-year return period snowpack by a critical temperature sequence longer than 15 days (severe event) followed by the summer probable maximum precipitation (extreme event) in early June.

Scenarios 1, 2, and 4 require the development of a critical temperature sequence for melting the snowpack. The same base sequence can be used for these three scenarios, with some variations to hasten or delay snowmelt depending on the timing of the probable maximum precipitation.

37.1.9.2.2 ~~37.1.11.2.2~~ Snowpack

The probable maximum snow accumulation was determined by maximization of the snowstorms that occurred during particularly snowy winter seasons. The four snowiest winters were first identified and all snowstorms that occurred in these winters were identified based on snow depths recorded at all climate stations in the region. A snowfall was maximized if 5 cm or more snow was recorded at a minimum of three stations over the catchment. Each snowstorm was maximized and the total maximized accumulation for each winter season was computed.

The methodology used for estimating the probable maximum snow accumulation was that proposed by the World Meteorological Organization (WMO 1969), as amended by Chow and Jones (1994) for application in Canada's boreal region.

Upper air data were used to estimate the moisture content in the air masses that produced the snow storms and to estimate the 100-year return period air moisture content. Maximization was done using upper air data because they provide more reliable estimates of moisture content than dew point temperature-derived estimates during the winter and spring. Air moisture from winter snow storms comes mostly from the Pacific Ocean. The upper air stations used for moisture content estimates were Prince George in central British Columbia, Fort Nelson in northern British Columbia, Port Hardy on the coast of southern British Columbia, and Annette Island in southern Alaska.

For Williston, the 2006 to 2007 winter season generated the probable maximum snow accumulation, and for the Site C local catchment, the governing winter season was 2004/05. Table 37.7 and Table 37.8 show the probable maximum snow accumulation for the Williston catchment and the Site C local catchment.

Table 37.7 Probable Maximum Snow Accumulation (cm) Over the Williston Catchment

Sub-basin	Probable Maximum Snow Accumulation
Finlay	733
Natpak	656
Willoc	659
Parsnip	1142

Table 37.8 Probable Maximum Snow Accumulation (cm) Over the Site C Local Catchment

Sub-basin	Probable Maximum Snow Accumulation
Upper Halfway	466
Graham	487
Lower Halfway	339
Moberly	268
Middle Peace	249

The 100-year return period snowpack was derived from data collected at snow course stations in and around the Williston and Site C catchments. In order to be conservative, the maximum snow water equivalent value in each year at each station was used in the analysis, whatever time during the winter season it was measured. The 100-year return

period snow water equivalent at each station was estimated using the Log-Pearson Type III distribution. Coverage of the snow water equivalent over the two catchments was prepared using ArcGIS. Table 37.9 and Table 37.10 show the snow water equivalent of the 100-year return period snowpack over each sub-basin for the Williston and Site C sub-basins.

Table 37.9 100-Year Snow Water Equivalent (mm) for Williston Sub-Basin

Sub-basin	100-year Snow Water Equivalent
Finlay	603
Natpak	533
Willoc	539
Parsnip	944

Table 37.10 100-Year Snow Water Equivalent (mm) for Site C Sub-Basin

Sub-basin	100-year Snow Water Equivalent
Upper Halfway	341
Graham	469
Lower Halfway	183
Moberly	253
Middle Peace	208

37.1.9.2.3 ~~37.1.11.2.3~~ Rainfall

Figure 37.10 and Figure 37.11 show the probable maximum precipitation over the Williston and Site C catchments, respectively.

As described above, 100-year rainfalls are used in Scenarios 2 and 3 in addition to the probable maximum precipitation.

The 100-year return period summer/fall rainfall over three days was first estimated at several meteorological stations in the Site C local catchment. The 100-year estimates were obtained with the Generalized Extreme Value distribution, based on visualization of the curve fits. Table 37.11 shows the distribution of the 100-year three-day summer rainfall over the sub-basins.

Table 37.11 100-Year Three-Day Summer Rainfall Distribution on Site C Local Catchment

Sub-basin	Three-Day Summer Precipitation (mm)
Upper Halfway	190.9
Graham	225.4
Lower Halfway	139.3
Moberly	117.0
Middle Peace	114.8
Catchment average	156.9

The 100-year return period spring rainfall was estimated based on a statistical analysis of the maximum three-day rainfall totals recorded between March 1 and May 31 on each

year at the climate stations. The 100-year estimates were obtained with the Log-Pearson Type III distribution, based on visualization of the curve fits. Table 37.12 shows the distribution of the 100-year three-day spring rainfall over the sub-basins.

Table 37.12 100-Year Three-Day Spring Rainfall Distribution on Site C Local Catchment

Sub-basin	Three-Day Spring Precipitation (mm)
Upper Halfway	92.0
Graham	89.1
Lower Halfway	71.6
Moberly	65.2
Middle Peace	65.0
Catchment average	80.0

ArcGIS was used to prepare coverage of the 100-year return period daily rainfall. The three-day storm distribution was assumed to be similar to the distribution of the spring or summer/fall probable maximum precipitation, as applicable. The three-day rainfall was assumed to start seven days before the beginning of the probable maximum precipitation, which means that there were four days without rainfall between the 100-year rainfall and the probable maximum precipitation.

37.1.9.2.4 ~~37.1.11.2.4~~ Critical Temperature Sequences

Critical temperature sequences were constructed to obtain severe sequences of high temperatures to accelerate melting of the snowpack. The methodology used for deriving the sequences was developed by Chow and Jones (1994).

The sequences were constructed by extracting for each year and at each meteorological station the maximum annual cumulated temperatures over 1, 2, 3, 4, 5, 10, 15, 30, and 45 consecutive days starting from the beginning of snowmelt. A statistical analysis for each series of cumulative temperatures was then carried out to obtain the 100-year return period estimate.

Progressively increasing sequences were constructed, culminating with the maximum temperature four or five days before the probable maximum precipitation, or any other intense rainfall, occurred. In all cases, the maximum daily temperature for any single day was limited to 24°C (Chow and Jones 1994). A slight drop in temperature is expected just before the probable maximum precipitation to reflect the passage of a cold front preceding the probable maximum precipitation (SNC-Shawinigan 1992). The probable maximum precipitation should last three days after the passage of the cold front and the temperature during the probable maximum precipitation should be close to the maximum dew point temperature at that location and time (Chow and Jones 1994).

The total duration of the temperature sequences depends on the snow water equivalent of the snowpack at the start of the melting period and on the snowmelt rate. As the duration of the melting period depends on the temperature sequence, temperature sequences of 15, 30, and 45 days' durations were constructed and tested successively to determine which sequence is critical, i.e., which sequence results in the greatest runoff. The time taken by the melt water to pass through the snowpack is taken into account by the calibrated hydrological model (see Section 37.1.9.2.5). In all cases,

snowmelt is assumed to start on May 1, based on an analysis of snow on ground at selected meteorological stations.

Figure 37.12 and Figure 37.13 present the 30-day and 45-day, respectively, temperature sequences derived for all meteorological stations used in the calibrated hydrological model for Site C. In some scenarios, these sequences were modified to make sure that the snowpack had not melted completely before the arrival of the probable maximum precipitation. The basic criterion, whether developing and modifying the sequences, is to make sure that the rain event occurs about the same day as the maximum runoff due to snowmelt. When modifications to the sequences were needed, they generally consisted of subjectively decreasing the temperatures in the first segment of the sequence. The highest temperatures in the last segment of the sequences were kept untouched because these generate higher melt rates and therefore maximize snowmelt runoff when the probable maximum precipitation occurs.

37.1.9.2.5 ~~37.1.11.2.5~~ Watershed Model

Determination of the probable maximum flood requires modelling of the physical processes that convert precipitation into streamflow. The physical processes producing floods are simulated using a watershed model that converts precipitation input into runoff from a watershed. The ability of the watershed model to provide an accurate representation of watershed hydrology is checked by comparing measured streamflow to streamflow simulated by model.

The UBC Watershed Model was used to determine the probable maximum flood for Site C because it:

- Is used by BC Hydro to forecast daily inflow to almost every BC Hydro reservoir and has proven ability to represent watershed hydrology
- Has been used the determine the probable maximum flood for many of BC Hydro's projects (Micovic 2005), as well as for numerous international projects

The UBC Watershed Model was originally developed for daily streamflow forecasting on the Fraser River system (Quick and Pipes 1977). The model calculates watershed outflow using point measurements of precipitation and temperature data combined with physical watershed characteristics as input. The model was designed primarily for the calculation of streamflow from mountainous watersheds where streamflow consists of snowmelt, rain, and glacier outflow. Since the hydrological behaviour of the mountainous watershed is a function of elevation, the model uses the area-elevation bands concept. This concept accounts for orographic gradients of precipitation and temperature, which are assumed to behave similarly for each storm and are dominant gradients of behaviour in mountainous areas. The physical description of a watershed is input for each elevation band separately in the form of different variables such as area of the band, forested fraction and forest density, glaciated fraction, band orientation, and fraction of impermeable area.

The UBC Watershed Model is a continuous hydrologic model, which means that it provides continuous streamflow output for the duration of the meteorological input.

In addition to the UBC Watershed Model, a flood routing model based on the Muskingum-Cunge method (Cunge 1969) was used to convey flood hydrograph estimates along the main tributaries through the sub-basins of the Site C local catchment

to the points of interest (i.e., hydrometric stations). The flood routing model computes attenuation effects and delays in peak arrival times.

Figure 37.14 shows the location of the pertinent meteorological stations and their respective areas of influence over the sub-basins in the Site C local catchment. The areas of influence of each station were established by application of the Thiessen Method (Thiessen 1911) and used to determine the weighting of their respective meteorological stations data over the modelled sub-basins.

Morphological characteristics, such as aereal extent, elevation bands, channel length and slope, and watershed boundaries in the Site C local catchment, were derived from a Digital Elevation Model with a resolution of 0.75 arc seconds, acquired from Natural Resources Canada. A GIS database of soil types and land cover was set up for each of the sub-basins, and percentages of impermeable and forested areas were calculated for each elevation band. Land cover and soil type information were collected from GIS databases available through Natural Resources Canada.

All sub-basin models in the Site C local catchment were calibrated and verified within the period ranging from September 1983 through September 2004. No verification was performed to the Upper Halfway sub-basin model due to the lack of streamflow data. The calibration of the hydrological model was aimed mainly at obtaining a satisfactory fit between computed and observed peak flows and volumes. For an ongoing evaluation of the calibration performance, statistical scores were used all along the process to assess the goodness-of-fit between computed results and observed data. Figure 37.15 compares simulated and observed hydrographs for the Middle Peace, Lower Halfway, and Moberly sub-basins for the 2000 to 2001 water year, which includes the flood of record at Site C.

The UBC Watershed Model gives daily streamflows. An analysis of hourly discharge at the gauging stations located in the Site C catchment revealed that the average ratio of peak instantaneous to maximum daily discharge ranged from 1.01 (Moberly River near Fort St. John) to 1.10 (Halfway River above Graham River). Therefore, daily discharges from the UBC Watershed Model were multiplied by 1.10 to give peak instantaneous discharges.

37.1.9.2.6 ~~37.1.11.2.6~~ Probable Maximum Flood

The four scenarios listed in Section 37.1.9.2.1 were run in the watershed model to determine the scenario that governed the probable maximum flood. In running these scenarios, the inputs that maximized runoff (e.g., storm centering and orientation and temperature sequence) were used. The following paragraphs describe the results.

For Scenario 1, melting of the 100-year return period snowpack was assumed to start on May 1, with the temperature increasing until it reached a maximum in late May following the sequence shown in Figure 37.12. The spring probable maximum precipitation was then assumed to occur with the timing of the rainfall set to maximize both the snowmelt runoff and the rainfall runoff on the same day. Figure 37.16 presents the results of the application of this scenario for the Site C local catchment. The maximum daily flow is 5,200 m^3/s, which, when multiplied by the 1.10 conversion factor, translates to a peak instantaneous flow of 5,720 m^3/s.

For Scenario 2, melting of the probable maximum snow accumulation was assumed to start on May 1, with the temperature increasing until it reached a maximum in late May following the sequence shown in Figure 37.12. The 100-year return period spring rainfall was then assumed to occur with the timing of the rainfall set to maximize both the snowmelt runoff and the rainfall runoff on the same day. The daily distribution of the 100-year return period spring rainfall was prorated as for the spring probable maximum precipitation. Figure 37.17 presents the results of this scenario for the Site C local catchment. The maximum daily flow is 4,410 m^3/s which, when multiplied by the 1.10 conversion factor, translates to a peak instantaneous flow of 4,850 m^3/s.

For Scenario 3, the 100-year return period summer rainfall was assumed to occur in late July, followed by the summer probable maximum precipitation at the beginning of August. Based on an analysis of storms carried out for the Site C basin, the average time period between successive storms was found to be seven days. As a storm generally lasts three days, it was assumed that there would be four days without rain between the two successive storms. Figure 37.18 presents the results of this scenario for the Site C local catchment. The maximum daily flow is 9,920 m^3/s, which, when multiplied by the 1.10 conversion factor, translates to a peak instantaneous flow of 10,900 m^3/s.

For Scenario 4, a period with relatively cool temperatures in the first three weeks of May was considered in order to delay the maximum runoff due to snowmelt until the summer probable maximum precipitation, which is more intense than the spring probable maximum precipitation. Such a cool three-week period was observed over the Site C local catchment in May 1996. The scenario assumed melting of the 100-year return period snowpack starting on May 1, with the temperature sequence consisting of the low temperatures measured in the first three weeks of May 1996 followed by the fourth week of the critical temperature sequence shown in Figure 37.12. The resulting temperature sequence is shown in Figure 37.19. The summer probable maximum precipitation was then assumed to occur with the timing of the rainfall set to maximize both the snowmelt runoff and the rainfall runoff on the same day. Figure 37.20 presents the results of the application of this scenario for the Site C local catchment. The maximum daily flow is 11,500 m^3/s, which, when multiplied by the 1.10 conversion factor, translates to a peak instantaneous flow of 12,560 m^3/s.

The preceding discussion presents the results for the Site C local catchment, which indicates that Scenarios 3 and 4 give the highest flow from that catchment. However, outflows from W.A.C. Bennett Dam influence the ranking of the scenarios and are required to give the total probable maximum flood inflow to Site C.

The various components involved in a probable maximum flood scenario have to be consistent for both the Williston and the Site C basins. For example, the storm generating the probable maximum precipitation is so large that it can only be centred at one location (i.e., worst location over the Site C local catchment or worst location over the Williston catchment). When the probable maximum precipitation is centred on one catchment, the other catchment receives associated rainfall (see Figure 37.11, which shows the summer probable maximum precipitation centred on the Site C local catchment). On the other hand, it is reasonable to assume that the probable maximum snowfall accumulation can occur in the same year in both basins. This is because snow accumulates over several months during the winter and shows some consistency over a very large area. For the same reason, the 100-year return period snowpack can also be

assumed to occur in the same year in both basins. Critical temperature sequences must also be consistent for both basins.

The results of several simulations conducted for both Williston and Site C showed that four scenarios generate the largest total inflows at Site C. Table 37.13 presents the results of these four scenarios. The corresponding outflows from W.A.C. Bennett Dam are dependent on Williston reservoir levels and operating rules. Releases from Peace Canyon Dam occur simultaneously with releases from W.A.C. Bennett Dam and flow changes are expected to occur at the Halfway River within an hour. Therefore, no delay was assumed between flow leaving W.A.C. Bennett Dam and arriving at Site C.

In the governing case shown in Table 37.13, the Williston Reservoir would fill prior to the probable maximum precipitation and spill. As shown in Figure 37.20, the peak discharge from the W.A.C. Bennett Dam would coincide with the peak flow from the Site C local catchment.

Table 37.13 Site C Probable Maximum Flood Scenarios

Scenario	Maximum Daily Flow (m^3/s)	Maximum Instantaneous Flow (m^3/s)
Scenario 3 with summer probable maximum precipitation centred over Site C local catchment	18,000	19,300
Scenario 3 with summer probable maximum precipitation centred over Williston catchment	10,130	10,350
Scenario 4 with June probable maximum precipitation centred over Site C local catchment	12,800	14,200
Scenario 4 with June probable maximum precipitation centred over Williston catchment	9,690	9,810

37.1.9.3 ~~37.1.11.3~~ Likelihood

The determination of the probable maximum precipitation followed accepted guidelines and standard practice. Reasonable but conservative assumptions (i.e., assumptions that would result in higher runoff estimates) were made. Key assumptions are:

- Precipitation patterns for extreme storms would be similar to mean patterns for June. This was considered reasonable because large storms occur most frequently in June.
- Precipitation in areas without orographic features is assumed to be 100% convergence related
- Precipitable water was calculated assuming a completely saturated air column. This is a conservative assumption and is consistent with other PMP studies, but could overestimate the precipitable water (Chen and Bradley 2006)
- The moisture supply in a storm can be increased without altering the dynamic structure of the storm, but if too much moisture is added, the storm can be altered. The moisture maximization ratio was limited to 1.50, which is consistent with HMR-55A.

- The storm intensity factor was not considered, which is conservative, as it yields high total precipitation
- It was assumed that the probable maximum precipitation in June would be the same as July

In addition, GIS was used to centre and orientate the probable maximum precipitation on the catchments. Software was developed that automatically computed the three-day rainfall over the catchments for a large number of possible storm centres and storm orientation. This identified the worst possible combination of storm location and orientation, and gave the largest possible estimates of total precipitation.

Derivation of the probable maximum flood also followed accepted guidelines and standard practice. As described above, the probable maximum flood is evaluated by deterministic methods that maximize the various factors contributing to the generation of the flood and, therefore, the methodology does not define an annual exceedance probability.

The governing condition was Scenario 3 with a storm producing the 100-year return period rainfall over the entire Site C catchment (severe event), followed by the probable maximum precipitation (extreme event) centred over the Site C local catchment starting four days after the end of the first storm. For the Site C local catchment, the probable maximum precipitation is 1.5 times the average 100-year return period rainfall. The precipitation from the two storms is equivalent to 84% of the mean annual rainfall at Fort St. John.

The likelihood of these two events occurring four days apart is less than 1/10,000.

The occurrence of the probable maximum flood is therefore considered to be extremely unlikely.

37.1.10 ~~37.1.12~~ Slope Stability and Mass Wasting Events

Slope instability and mass wasting could adversely affect the Project by:

- Landslides damaging the dam, generating station, and spillways
- Landslides on the reservoir shoreline damaging property, including Highway 29
- Failure of a landslide dam, releasing high flows that overtop the Stage 2 upstream cofferdam
- Landslide-generated waves damaging the Stage 2 upstream cofferdam
- Landslide-generated waves on the reservoir damaging the earthfill dam

Volume 2 Section 11.2 Geology, Terrain, and Soils, and Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 1 Terrain Stability Mapping describe:

- The terrain mapping techniques that were used to delineate areas with distinct surficial geology and terrain stability
- The terrain stability assessment (the potential for landslide initiation following disturbance by road construction or removal of forest cover) in the Project activity zone, including the dam site, reservoir, transmission line, and Highway 29 realignment segments

Volume 2, Section 11.2, Geology, Terrain, and Soils, and Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 2 Preliminary Reservoir Impact Lines describe:

- The geology and a detailed landslide inventory of the proposed reservoir shoreline
- The assessment of reservoir shoreline erosion and stability
- The assessment of potential landslide-generated waves

The following subsections describe the planning and design strategies that would be used to minimize the potential effects of slope instability and mass wasting on the Project, based on the information listed above.

37.1.10.1 ~~37.1.12.1~~ Dam Site Slope Stability

There are active slumps and small slides on the north bank of the Peace River at the dam site. The terrain stability mapping showed that the north bank is expected to contain areas with moderate likelihood (Class IV) to high likelihood (Class V) of landslide initiation following disturbance.

As described in Volume 1 Section 4 Project Description, the north bank above the earthfill dam would be excavated to remove unstable material and flatten the slope. The slope would be designed so that there would not be any slides with the potential to cause an adverse effect on the Project. Refer to Section 37.1.6.1.4 ~~37.1.2.3~~ for the performance of the north bank during earthquakes.

On the south bank at the dam site, the slopes between the Peace River and the terrace were assessed as having a low likelihood of landslide initiation following disturbance, with minor slumping expected along road cuts, especially for one or two years following construction (Class III). The majority of this slope is comprised of colluvium and weathered shale overlying shale bedrock.

Colluvium and weathered shale would be excavated for construction of the dam. Disposal area R5b, the earthfill dam, the RCC buttress, and disposal area R6 would support the slope after construction. As a result, no slides from this slope are expected that could adversely affect the Project.

On the south bank downstream of the dam site, the slopes between the river and the terrace were assessed as having areas with a high likelihood of landslide initiation after disturbance (Class V). No disturbance of these slopes is currently planned. However, if any disturbance is required during construction, drainage and other slope stabilization methods would be utilized so that there would not be any potential slides that would have the potential to cause an adverse effect on the Project.

Above the terrace on the south bank of the Peace River at the dam site, the lower parts of the slopes were assessed as having very low likelihood of landslides following road construction, with minor slumping expected along road cuts, especially for one or two years following construction (Class II) and the upper parts of the slopes were assessed as containing areas with a moderate likelihood of landslide initiation following disturbance (Class IV). The approach channel has been located so that the excavations for this part of the Project would not disturb the slope. However, it is possible that some disturbance may occur during construction, in which case drainage and other slope

stabilization methods would be utilized so that there would not be any potential slides that would have the potential to cause an adverse effect on the Project.

37.1.10.2 ~~37.1.12.2~~ Reservoir Shoreline Stability

Reservoir shoreline stability is described in Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 2 Preliminary Reservoir Impact Lines.

As described, shoreline protection beneath the community of Hudson's Hope would be constructed prior to filling the reservoir. The shoreline protection would extend from a location where the proposed reservoir shoreline transitions from bedrock to interbedded sand, silt, and clay materials at the upstream end, downstream to beyond the current location of the municipal sewage treatment facility, for a total length of about 2,650 m. As the proposed shoreline protection offsets the potential effects of the reservoir on erosion and slope stability, the potential for effects of slope stability on the Project in the area of Hudson's Hope is very unlikely.

The proposed realigned segments of Highway 29 have been located outside of the preliminary impact lines where practical. Some existing segments of Highway 29 are currently situated on marginally stable slopes and are located within the Stability Impact Line. Each of these segments has been reviewed by BC Hydro and the BCMoTI. It has been determined by BC Hydro that the potential effects of the proposed reservoir on the stability of these highway segments are low, and an approach to ongoing highway monitoring and maintenance has been established in collaboration with the BCMoTI. Therefore, through design and monitoring, adverse effects on the Project due to slope instability in this area are very unlikely.

The 1973 Attachie slide is described in Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 2 Preliminary Reservoir Impact Lines. Since the 1973 slide blocked the Peace River, the potential occurrence of a similar slide at Attachie during the Stage 2 diversion was considered. For the purpose of determining the required capacity of the diversion tunnels, it was assumed that another similar size slide could occur at Attachie and form a landslide dam across the Peace River upstream of the Halfway River. The size of the landslide dam was based on LiDAR mapping of the remnants of the 1973 slide. A dam break analysis was performed to provide an estimate of the outflow from overtopping and washing out of the landslide dam. The Peace Canyon discharges recorded on May 23, 1973, the date of the Attachie slide, were used as the inflow.

The analysis showed that the diversion tunnels have sufficient capacity to pass the flood wave resulting from the breach of a landslide dam caused by a similar slide at Attachie, even if the peak of the landslide breach outflow coincided with the peak of a 50-year flood, because:

- Inflow from Peace Canyon Dam to the headpond at the cofferdam would be cut off by the landslide and the headpond level would drop
- The volume of water stored behind the landslide dam and released when the landslide dam breached would be about 41 million m^3, which is a small proportion of the storage volume of the headpond, which would be about 401 million m^3 at the crest level of the Stage 2 upstream cofferdam

As described in Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 2 Preliminary Reservoir Impact Lines, the 1973 Attachie landslide was a very unusual event in terms of the volume of the slide and the high velocity, which combined to block the Peace River. The capacity of the diversion works would be sufficient to accommodate a similar event without overtopping of the Stage 2 upstream cofferdam.

37.1.10.3 ~~37.1.12.3~~ Landslide-Generated Waves

Potential landslide-generated waves are described in Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 2 Preliminary Reservoir Impact Lines. This section discusses the potential effects of landslide-generated waves on the Project during construction and operations.

During Stage 2 diversion, there would be no water impounded opposite Attachie or Bear Flat, so landslide waves could not be generated.

Physical hydraulic model studies of Attachie and Bear Flat undertaken in 1983 indicated that landslide-generated waves at either of these locations would travel down the reservoir and attenuate before reaching the dam and, therefore, potential landslides at these locations would not have an adverse effect on the dam.

Based on the terrain stability mapping, potential sources of extremely rapid landslides (landslides originating in over-consolidated glaciolacustrine deposits) were considered along the Peace River and Moberly River within approximately 5 km of the proposed dam site. Three key potential sources (due to their potential large volume, proximity to the dam, and orientation relative to the dam) were identified for detailed analysis. The governing source area of these three sources is located at the crest of the ridge on the left bank of the Moberly River at its confluence with Peace River. The estimated maximum landslide volume from the governing source area is 2 million m^3.

The techniques for estimating potential landslide-generated waves described in Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 2 Preliminary Reservoir Impact Lines were used to estimate the waves that could be generated by a landslide from the governing source area. Two parameters are required to estimate the potential landslide generated wave due to a landslide from this source area:

- Landslide volume that could enter the reservoir or headpond
- The velocity of the landslide entering the reservoir or headpond

These parameters were estimated from historic landslides.

Considering the typical deposit distribution of large overburden landslides in the reservoir area, it is expected that 50% of the potential landslide volume could reach the reservoir or headpond surface, with the remainder being deposited in the source area and on the slope between the source area and the reservoir or headpond. As a result, potential landslides that could impact the reservoir or headpond with volumes ranging between 0.5 million m^3 and 1 million m^3 were analyzed, in increments of 0.25 million m^3.

Corresponding estimated impact velocities ranged between 20 m/s and 25 m/s. These velocity estimates were based on a set of peak landslide velocities that were back-calculated using energy grade line methods at sites in the reservoir area where overburden landslide deposits are well preserved, in addition to velocities simulated during back-analyses of the 1973 Attachie slide using a numerical model. Note that not

all (and most likely only a small proportion) of previous overburden landslides in the reservoir area reached such extremely rapid velocities. The conservative assumption of extremely rapid behaviour was made for the purposes of determining the largest possible wave-generating event.

The analyses indicate that that the normal freeboards provided listed in Table 37.2 are sufficient to prevent a landslide-generated wave from overtopping:

- The Stage 2 cofferdam during diversion
- The earthfill dam during operation

The proposed highway realignment at the Halfway River crossing is situated inside the Wave Impact Line described in Volume 2 Appendix B Geology, Terrain Stability, and Soil Reports, Part 2 Preliminary Reservoir Impact Lines. The potential for landslide-generated waves has been considered in determining the highway embankment elevation, bridge elevation, and bridge design parameters. The proposed highway and bridge design at Halfway River has been reviewed by the BCMoTI and satisfies provincial design codes and safety guidelines.

37.1.10.4 ~~37.1.12.4~~ Transmission Line

The transmission lines cross a gentle plateau area underlain by glaciolacustrine silts and clays or glacial till, with bogs scattered throughout the area. Much of the plateau area is very gently sloping, and no landslides are present. As described in Volume 2 Appendix B, Geology, Terrain Stability, and Soil Reports, Part 1 Terrain Stability Mapping, these areas are mapped as stable (Class I). Steeper slopes exist where streams cross the plateau.

The location of transmission towers will be selected to avoid unstable areas. This may require the adjustment of span lengths near stream crossings and other areas with steep slopes.

The risk of landslides adversely affecting the transmission line will be reduced by locating towers away from unstable areas and steep slopes.

37.1.11 ~~37.1.13~~ Climate Change

The potential for climate change to affect the Project is described in this section. The potential effects of climate change on the Project are:

- Changes to the generation of electricity
- Changes to the magnitudes of extreme floods

Climate change in the Peace Region is discussed in Volume 2 Appendix T Climate Change Summary Report. As discussed in that report, as a result of climate change:

- Annual streamflow in the Peace River will likely increase through the 21st century
- Temperatures will increase into the 21st century
- Warming will be greatest in winter and will fall outside the range of historical variability

- Median projected precipitation changes range from 11% to 14% by 2050 and from 14.5% to 19% by 2080, depending on emission scenario
- Winter and spring will see the highest increases in flow
- Summer and spring precipitation will show less spatial variation
- Precipitation increases will not fall outside the range of historical variability

37.1.11.1 ~~37.1.13.1~~ Electricity Generation

Increases in Peace River streamflow would increase the generation of electricity from the Project.

Thus, potential effects of climate change on electricity generation would be beneficial.

37.1.11.2 ~~37.1.13.2~~ Floods

Given the anticipated schedule for construction of the Project, climate change is unlikely to affect the floods used for the design of the diversion works.

As described in Section 37.1.9, the Project would be designed to pass the probable maximum flood which is an extreme event determined by analysis of historical climate data. With climate change it is possible that estimates of the probable maximum flood may increase (due to higher precipitation) or decrease (due to lower snowpack or lower summer precipitation).

BC Hydro's dam safety program and the B.C. Dam Safety Regulation require periodic dam safety reviews (currently every seven years for a dam in Site C's category). As part of these reviews, it would be normal practice to review any changes to design standards and whether there is any evidence of changes to climatic conditions or floods that would affect the estimate of Inflow Design Flood. If there was any evidence to suggest that the Inflow Design Flood for the Project may have increased, studies would be undertaken to determine the new Inflow Design Flood.

The peak instantaneous inflow to Site C during the probable maximum flood is estimated to be 19,300 m^3/s (Section 37.1.9.2.6). As described in Section 4 Project Description, the spillways would have a capacity of 17,300 m^3/s at the maximum flood level because routing the probable maximum flood through the reservoir attenuates the flood, resulting in a peak instantaneous spillway discharge of 17,300 m^3/s. Spillway discharge would be less than inflow due to storage of flood water between the maximum normal reservoir level and the maximum flood level.

Should future estimates of the Inflow Design Flood be greater than the current estimate of the probable maximum flood, there would be a number of measures that could be undertaken in the future to enable Site C to pass the larger flood as described below:

1. A gate could be added to the auxiliary spillway to increase spillway capacity by about 1,000 m^3/s, which would also provide addition spillway capacity prior to reservoir surcharging.
2. The auxiliary spillway could be widened and rebuilt to increase spillway capacity by 20% or more.

3. The current probable maximum flood routing at Williston is conservative, in that all reservoir inflow is assumed to pass straight through the reservoir so that outflow equals inflow. Any temporary Williston reservoir storage would decrease flood peaks at Site C. For example, a slower ramp-up of Williston spillway discharge resulting in a 24-hour delay in reaching peak Williston discharge could result in reducing the current Site C probable maximum flood peak inflows by up to 3,700 m^3/s.
4. The earthfill dam could be raised to allow a higher maximum flood level, which would provide additional attenuation due to larger flood storage and additional spillway discharge capacity.
5. Use improved forecasting to pre-spill or pre-generate to provide more flood storage capacity to attenuate floods.

Based on the above measures it is considered that the Project would be able to accommodate changes to extreme floods due to climate change.

37.2 Potential Accidents and Malfunctions

37.2.1 Introduction

This section:

- Identifies the parts of the Project that have the potential, through accident or malfunction, to adversely affect the environment
- Discusses the likelihood and circumstances under which those accidents or malfunctions could occur
- Provides an overview of the measures that would be implemented to reduce the likelihood and those that could be implemented to mitigate the potential occurrence of an accident or malfunction
- Identifies the potential effects of accidents and malfunctions

Dam breach analyses that have been undertaken for the Project are described in Section 37.2.2:

- Section 37.2.2.1 describes the data and methodology
- Section 37.2.2.2 presents the results for the Stage 2 upstream cofferdam
- Section 37.2.2.3 presents the results for the earthfill dam

Accidents and malfunctions that could occur during construction are discussed in Section 37.2.3.

Accidents and malfunctions that could occur during operation are discussed in Section 37.2.4.

Section 37.2.5 identifies the potential effects of accidents and malfunctions on valued components.

37.2.2 Dam Breach Analyses

As described in Volume 1 Section 4 Project Description, an understanding of the consequences of dam failure underlies several principles in the Canadian Dam Association Dam Safety Guidelines (CDA 2007c). Dam breach analyses are used to assess the consequences of dam failure by the characterization of a hypothetical dam breach and routing the resulting flood wave. The consequence classification of a dam is based on the consequences of a dam failure and determines not only the design standards but also the requirements for dam operation, maintenance and surveillance, and emergency preparedness planning. As described in Volume 1 Section 4 Project Description, BC Hydro has adopted the highest classification for the Project.

The requirements for operation, maintenance, and surveillance for dam safety, and for emergency preparedness plans, are described in Volume 1 Section 4 Project Description.

Failure of the Stage 2 upstream cofferdam or failure of the earthfill dam would have an adverse effect on the environment. As described in Section 37.1.8.3, the annual exceedance frequency of a flood potentially damaging or overtopping the Stage 2 upstream cofferdam is approximately 1/600.

As described in Volume 1 Section 4 Project Description, the dam would be designed to the highest standard for the inflow design flood and earthquake design ground motion contained in the Canadian Dam Association Dam Safety Guidelines, namely earthquake ground motions with an annual exceedance frequency of 1/10,000 and the probable maximum flood. As described in Section 37.1.9.3, the probable maximum flood is derived deterministically and the methods do not directly provide an annual exceedance frequency. However, the likelihood of the two consecutive storms occurring four days apart used in the determination of the probable maximum flood is less than 1/10,000.

Failure of the dam is very unlikely, given the extreme flood and earthquake for which it would be designed and given the rigorous operation, maintenance and surveillance requirements that would be implemented for the Project.

Despite the unlikely failure of the Stage 2 upstream cofferdam and the very unlikely failure of the earthfill dam, these possibilities are considered for assessing the potential effects of accidents and malfunctions and for preparing emergency preparedness plans.

37.2.2.1 Data and Methodology

This section describes the data and methodology used for the dam breach analyses that were undertaken to assess the potential effects of hypothetical failures of the Stage 2 upstream cofferdam and the earthfill dam. The methodology is well understood and in accordance with the state of practice for dam breach assessments.

The method consists of:

- Estimating dam breach parameters and the resulting flood wave
- Routing the flood waves from hypothetical dam breaches down the river

The MIKE 11 model, which is a one-dimensional hydrodynamic model that was used for routing the flood waves from hypothetical dam breaches down the Peace River from the

dam site to Peace Point, Alberta. The MIKE 11 model is described in Volume 2 Appendix D Surface Water Regime Technical Memos.

The MIKE 11 program has been used extensively for dam break analysis by BC Hydro and others, and is accepted by the U.S. Federal Emergency Management Agency for use in the National Flood Insurance Program (FEMA 2012). To gain acceptance by the Federal Emergency Management Agency, a model must be reviewed and tested. A model is certified by another U.S. government agency that is familiar with the model. The certifying agency reviews and tests the model to determine whether the model is scientifically correct and technically sound, and whether the model can provide adequate information to support National Flood Insurance Program studies and mapping (FEMA 2004).

As described below, the MIKE 11 model was modified for the dam breach study to improve accuracy for higher discharge scenarios.

37.2.2.1.1 Bathymetric and Topographic Data

The bathymetric data at cross-sections across the Peace River used to set up the MIKE 11 model are described in Volume 2 Appendix D Surface Water Regime Technical Memos. Approximately 50 sections were added in the Peace River reach in Alberta, mainly near the developed areas. Most of these sections have interpolated bathymetry.

Since the flood waves from hypothetical dam breaches would flood areas outside the normal river channel, topographic data were required to extend the river cross-sections in the MIKE 11 model to higher elevations. The areas above the normal river banks are referred to as overbank areas.

Topographic data along the Peace River in B.C. consist of a LiDAR digital elevation model collected in 2006 and 2007 (vertical accuracy approximately +/- 0.3 m). A provincial TRIM (Terrain Resource Inventory Mapping) digital elevation model was used in B.C. to fill gaps at higher elevations (vertical accuracy of approximately +/- 5 m).

The LiDAR digital elevation model collected between 2005 and 2009 (vertical accuracy approximately +/- 0.3 m) was acquired for over 50% of the 970 km length of the Peace River corridor in Alberta. This LiDAR digital elevation model covers the entire Peace River reach between the B.C.–Alberta border to 25 km downstream of the Town of Peace River and the main developed areas further downstream such as near Carcajou and Fort Vermilion. For study areas in Alberta not covered by LiDAR data (i.e., undeveloped areas downstream of the Town of Peace River), the Canadian Digital Elevation Data, which is available online, were used (vertical accuracy of approximately +/- 5 m).

LiDAR data were not readily available for a few small developed areas along the Peace River downstream of Fort Vermilion. Local surveys were performed in these areas to provide elevations along roads and near buildings.

Flood protection dikes for the Town of Peace River were represented in the model based on as-built drawings prepared by Bekevich Engineering Ltd. in 1999.

37.2.2.1.2 Hydrometric Data

Hydrometric data were required for hydrodynamic model calibration.

Volume 2 Section 11.4 Surface Water Regime describes the surface water regime of the Peace River and lists the Water Survey of Canada gauges along the Peace River between Peace Canyon Dam and Peace Point. Calibration of the MIKE 11 model relative to the Water Survey of Canada gauge rating curves is described in Volume 2 Appendix D Surface Water Regime Technical Memos.

Hourly flow data for Water Survey of Canada gauged tributaries and stations along the Peace River were obtained for the July 2002 and June 1990 periods for additional hydraulic model calibration for flood wave routing. A spill at Peace Canyon Dam in July 2002 had a maximum total outflow of about 3,200 m^3/s, while June 1990 represents the flood of record for the Peace River at Peace Point.

37.2.2.1.3 Hydrodynamic Model Development and Calibration

The Project reservoir and the downstream Peace River channel and overbank areas were represented in the model with a series of cross-sections. Cross-sections were developed using the bathymetric and topographic data sources described above. The Peace River corridor was represented in the MIKE 11 model by more than 400 sections.

The boundary conditions for the MIKE 11 model consisted of an inflow to the Project reservoir (i.e., Peace Canyon Dam outflow plus local inflows) and an assumed constant water level more than 100 km downstream of Peace Point. As described in Volume 2 Appendix D Surface Water Regime Technical Memos, the downstream boundary was set far enough downstream to not influence simulated flows and levels at Peace Point. Gauged tributary flow estimates were added to the model as lateral inflows.

Manning's coefficients for channel roughness were set in the model through the calibration process described below. Manning's roughness is an empirically derived coefficient that is dependent on many factors, including the surface roughness and sinuosity of a river channel. It is used in hydraulic calculations for open channel flow. The velocity of flow in an open channel is inversely proportional to the roughness. The channel roughness used was approximately 0.03 in the reach between Site C and the B.C.–Alberta border and decreased to about 0.02 at Peace Point. Roughness coefficients in the 0.07 to 0.1 range were used to represent overbank areas with medium to dense vegetation.

The MIKE 11 model was calibrated and checked by:

- Comparing the model representation of the Site C Reservoir with the storage curve
- Matching the Water Survey of Canada rating curves along the Peace River downstream of Site C Dam
- Running hourly Peace Canyon discharges from July 2002 and June 1990 with corresponding lateral inflows from gauged tributaries (i.e., with no dam at Site C)

The calibration at Peace Point was extended up to the maximum discharge in the Water Survey of Canada rating curve, approximately 16,000 m^3/s.

The MIKE 11 representation of the Site C Reservoir was compared to the storage curve by computing the modelled dam breach outflow hydrograph volume immediately downstream of Site C dam. The modelled volume was found to be within 1% of the storage curve volume, which is shown in Figure 4.24 in Volume 1 Section 4 Project

Description. The storage volume curve was calculated from the LiDAR digital elevation model for the reservoir area.

For the rating curve comparisons simulations, modelled results were generally within about 0.2 m of the full range of the Water Survey of Canada rating curves along the Peace River.

For the July 2002 flow routing tests, modelled flows at Water Survey of Canada stations followed the observed flow patterns very well to the Town of Peace River. Modelled flows at Peace Point were within 200 m^3/s of measured values. The Water Survey of Canada gauge at Fort Vermilion was out of operation in 2002, so no comparison could be made between model and observed data at this location.

Modelled flows for June 1990 generally followed the observed flow patterns at Water Survey of Canada stations to the Alces River. Modelled maximum flows at Dunvegan and the Town of Peace River were approximately 1,400 m^3/s and 3,700 m^3/s less than the observed maximum flows, likely due to local inflows from ungauged tributaries downstream of the Alces River during the very large storm event that occurred in this reach during the June 12–14, 1990 period that resulted in the flood of record for the Smoky River. Ungauged flow contributions are typically relatively low in this reach.

The Water Survey of Canada measured discharges for the Peace River at the Town of Peace River for June 1990 were input into the MIKE 11 model to route the flows further downstream. There was very good agreement between modelled and measured flows at Peace Point. Daily measured flow data for June 1990 were used for comparison with modelled results at Fort Vermilion since the hourly measured data are unreliable. Modelled peak water levels through the developed areas at Fort Vermilion for the June 1990 event were within 0.5 m of high water marks reported in Alberta Environment (2000).

37.2.2.1.4 Dam Breach Parameters

Breach parameters (breach width and breach formation time) for the cofferdam and earthfill dam were estimated using USBR (1988) to represent the hypothetical failures.

37.2.2.2 Failure of the Stage 2 Upstream Cofferdam

37.2.2.2.1 Scenarios

The following analyses are for the Stage 2 River Diversion shown on the construction schedule in Figure 4.40 in Volume 1 Section 4 Project Description.

As described in Section 37.1.8, part of the strategy for managing floods during Stage 2 diversion is to reduce discharges from Peace Canyon Dam on the arrival of a flood from the Site C local catchment. The following assumptions were made to assess the potential effects of a hypothetical failure of the Stage 2 upstream cofferdam:

- 50-year flood routed through the headpond and tunnels
- Three units at Peace Canyon Dam at full discharge without any reduction in discharges on flood arrival at Site C leading to overtopping of the upstream cofferdam, i.e., the flow management measures to reduce Peace Canyon discharges on flood arrival at the dam site are not implemented

Two sets of downstream tributary inflows were considered:

- 50-year floods with the peaks coinciding with the dam breach flood wave arrival at the confluence of the tributary and the Peace River from all gauged tributaries downstream to the Town of Peace River, Alberta, and coincident historical maximum flows from the tributaries downstream from the Town of Peace River
- Constant average June flows

The dam breach parameters were based on the first year of diversion, when construction of the portion of the earthfill dam across the main river channel would commence. In subsequent years, partial construction of the main earthfill dam downstream of the cofferdam would lead to a multi-stage failure and likely lower downstream peak flows and water levels.

For comparative purposes, flow routings were also completed for:

- The natural river conditions, which give the water levels and flows for the 50-year flood that would naturally occur without the Project
- The expected conditions, which give the water levels and flows for the 50-year flood with the river diverted through the tunnels and the flow management measures implemented as planned

37.2.2.2.2 Results

Table 37.14 shows the maximum water levels and peak flows for the 50-year flood scenario with 50-year tributary inflows as listed above for the natural river conditions, i.e., with no cofferdam. These are the water levels and flows that would naturally occur without the Project.

Table 37.14 Inundation Modelling Results for 50-Year Flood (No Cofferdam) with 50-Year Tributary Inflows

Location	Distance from Site C (km)	Max. Water Level (m)	Peak Flood Flow (m^3/s)
Site C	0	414.7	4,850
Old Fort	7	411.0	4,840
Taylor	18	406.8	7,980
B.C.–Alberta border	62	387.4	10,050
Dunvegan	191	348.4	10,540
Town of Peace River	291	318.1	17,330
Carcajou	546	267.1	17,010
Fort Vermilion	726	254.5	15,640
Peace Point	1,031	219.6	16,060

Table 37.15 shows the maximum water levels and peak flows for the 50-year flood scenario with 50-year tributary inflows as listed above with the cofferdam. These are the water levels and flows that would occur with the flow management measures implemented as planned.

Table 37.15 Inundation Modelling Results for 50-Year Flood with Cofferdam and 50-Year Tributary Inflows

Location	Distance from Site C (km)	Max. Water Level (m)	Peak Flood Flow (m^3/s)
Site C	0	430.5	2,440
Old Fort	7	408.9	2,440
Taylor	18	406.0	5,850
B.C.–Alberta border	62	386.2	7,980
Dunvegan	191	347.3	8,480
Town of Peace River	291	317.3	14,900
Carcajou	546	266.3	14,790
Fort Vermilion	726	253.9	13,630
Peace Point	1,031	218.7	13,330

Table 37.16 and Table 37.17 present the results of the analyses for the 50-year flood overtopping scenario listed above with the two tributary inflow scenarios described above.

Table 37.16 Inundation Modelling Results for the Stage 2 Upstream Cofferdam Overtopping with 50-Year Tributary Inflows

Location	Distance from Site C (km)	Initial Water Level [a] (m)	Max. Water Level (m)	Water Level Increase (m)	Flood Arrival Time [b] (hr.)	Time to Flood Peak [b] (hr.)	Peak Flood Flow (m^3/s)
Site C	0	–	433.9	–	–	–	22,840
Old Fort	7	409.2	417.4	8.2	0.5	2.3	18,950
Taylor	18	405.9	410.3	4.4	1.0	3.9	18,770
B.C.–Alberta border	62	386.0	391.1	5.1	2.2	9.5	17,830
Dunvegan	191	346.4	349.4	3.0	8.2	19.1	15,580
Town of Peace River	291	316.8	319.6	2.8	13.0	26.2	21,390
Carcajou	546	262.6	267.8	5.2	27.0	43.2	19,670
Fort Vermilion	726	250.3	254.9	4.6	48.0	67.8	16,960
Peace Point	1,031	215.5	219.7	4.2	77.7	124.5	16,370

NOTES:
[a] Prior to cofferdam breach
[b] From start of cofferdam breach

Table 37.17 Inundation Modelling Results for the Stage 2 Upstream Cofferdam Overtopping with Average June Tributary Inflows

Location	Distance from Site C (km)	Initial Water Level [a] (m)	Max. Water Level (m)	Water Level Increase (m)	Flood Arrival Time [b] (hr.)	Time to Flood Peak [b] (hr.)	Peak Flood Flow (m^3/s)
Site C	0	–	433.9	–	–	–	22,840
Old Fort	7	409.1	417.3	8.2	0.5	2.4	18,950
Taylor	18	404.6	409.6	5.0	1.0	3.8	17,020
B.C.–Alberta border	62	382.1	389.4	7.3	3.0	9.7	14,620
Dunvegan	191	342.2	347.8	5.6	11.0	20.3	11,790
TPR	292	313.2	316.3	3.1	18.0	28.3	11,800
Carcajou	546	259.5	263.0	3.5	35.0	51.8	9,000
Fort Vermilion	726	249.2	251.2	2.0	50.0	81.2	7,250
Peace Point	1,031	214.1	216.0	1.9	80.0	123.9	6,990

NOTES:
[a] Prior to cofferdam breach
[b] From start of cofferdam breach

37.2.2.3 Failure of the Earthfill Dam

37.2.2.3.1 Scenarios

The following dam breach scenarios for Site C were assessed:

- Sunny day failure (for example, after an earthquake or piping through the earthfill dam or its foundation)
- Overtopping failure during the probable maximum flood

Mean annual flows in the Peace River system were assumed for the sunny day failure scenario, since it could occur at any time of the year. The Site C reservoir was assumed to be at the maximum normal reservoir level of elevation 461.8 m prior to dam failure. These initial conditions are consistent with the Canadian Dam Association Guidelines (CDA 2007c).

For the purposes of assessing the overtopping failure scenario during the probable maximum flood, malfunction or debris blockage of all spillway gates and the auxiliary spillway overflow was assumed. This is a very conservative assumption to trigger the hypothetical dam breach during the probable maximum flood.

Peace River gauged tributaries between Site C and the Town of Peace River (i.e., as far downstream as the Heart River) were assumed to be coincidentally at 200-year flood conditions. The 200-year maximum daily flood estimates for the Peace River tributaries between Site C and the Town of Peace River are listed below:

- Pine: 4,720 m^3/s
- Beatton: 2,110 m^3/s
- Kiskatinaw: 720 m^3/s
- Pouce Coupe: 460 m^3/s
- Clear: 530 m^3/s
- Smoky: 10,380 m^3/s
- Heart: 200 m^3/s

The locations of the above tributaries are listed in Volume 2 Section 11.4 Surface Water Regime.

Gauged tributaries downstream of the Town of Peace River were considered too far from Site C to assume 200-year flood conditions concurrent with the probable maximum flood, since the storm providing the probable maximum precipitation would not extend that far. Therefore, the historical maximum daily flows were used for these tributaries. The timing of the peak flows for all gauged tributaries was set to coincide with the Site C dam breach flood arrival at the corresponding tributary confluence locations. The shapes of the tributary hydrographs were derived from analysis of large historical events.

For comparative purposes, dynamic flow routings of the probable maximum flood were also performed to provide baseline inundation levels for:

- The natural river conditions that give the water levels and flows for the probable maximum flood that would naturally occur without the Project
- The expected conditions that give the water levels and flows for the probable maximum flood with operation of the spillway as designed

According to the Canadian Dam Association Dam Safety Guidelines (2007b), powerhouse discharge can be included for flood passage, subject to limitations of power system operations and demand. The spillway capacity for the Project was selected based on the conservative assumption that the powerhouse would be out of operation during the probable maximum flood due to the severe weather conditions that cause the

flood. For the dam breach analysis, it was also assumed that there would be no powerhouse discharges for the probable maximum flood scenarios.

37.2.2.3.2 Results

Sunny Day

Model results at key locations for the sunny day failure are presented in Table 37.18.

Table 37.18: Inundation Modelling Results for the Sunny Day Failure Scenario

Location	Distance from Site C (km)	Initial Water Level a (m)	Max. Water Level (m)	Water Level Increase (m)	Flood Arrival Time b (hrs.)	Time to Flood Peak b (hrs.)	Peak Flood Flow (m3/s)
Site C Dam	0	–	461.8	–	–	–	77,090
Old Fort	7	407.7	425.8	18.1	0.9	2.8	72,590
Taylor	18	403.3	418.2	14.9	1.5	3.9	64,500
B.C.–Alberta border	62	380.0	402.3	22.3	3.8	9.0	46,370
Dunvegan	191	340.2	356.8	16.6	9.5	17.7	34,800
Town of Peace River	291	311.9	322.2	10.3	15.7	25.8	31,510
Carcajou	546	256.9	267.8	10.9	30.8	46.6	23,190
Fort Vermilion	726	247.4	254.0	6.6	42.2	73.9	14,300
Peace Point	1,031	212.3	218.1	5.8	73.5	111.1	11,730

NOTES:
[a] Prior to dam breach
[b] From start of Site C Dam breach

The peak outflow for the Site C sunny day failure scenario would be about 77,090 m^3/s at Site C and would attenuate to 46,370 m^3/s at the B.C.–Alberta border, 31,510 m^3/s at the Town of Peace River, and 11,730 m^3/s at Peace Point, which is less than the peak discharge for the June 1990 flood of record at Peace Point.

Peak water levels would be 15 m to 22 m above the initial water levels within B.C. and 6 m to 17 m above the initial water levels near the populated areas in Alberta, and the dikes at the Town of Peace River would be overtopped by 1.7 m.

Probable Maximum Flood

Model results at key locations for the probable maximum flood are given in Table 37.19, Table 37.20 and Table 37.21.

Table 37.19 Inundation Modelling Results for the Probable Maximum Flood without the Project

Location	Distance from Site C (km)	Maximum Water Level (m)	Peak Flood Flow (m^3/s)
Site C Dam	0	420.5	18,730
Old Fort	7	417.7	18,710
Taylor	18	412.1	23,210
B.C.–Alberta border	62	396.0	25,350
Dunvegan	191	354.5	25,380
Town of Peace River	291	322.9	33,900
Carcajou	546	271.1	32,200
Fort Vermilion	726	257.5	29,390
Peace Point	1,031	222.1	25,280

The results in Table 37.19 show that, under natural conditions, the probable maximum flood would cause extensive flooding at all communities downstream. The dikes at the Town of Peace River would be overtopped by 2.4 m.

Table 37.20 Inundation Modelling Results for the Probable Maximum Flood with the Project

Location	Distance from Site C (km)	Maximum Water Level (m)	Peak Flood Flow (m^3/s)
Site C Dam	0	466.3	17,100
Old Fort	7	417.2	16,970
Taylor	18	411.6	21,440
B.C.–Alberta border	62	395.1	23,350
Dunvegan	191	352.3	23,400
Town of Peace River	291	322.5	32,480
Carcajou	546	270.9	31,190
Fort Vermilion	726	257.4	28,920
Peace Point	1,031	222.1	25,210

The results in Table 37.20 show that with the Project operating as designed, flooding would be attenuated to some extent relative to natural conditions; therefore, flooding would be less extensive. The maximum spillway discharge would be 17,100 m^3/s. Peak flows in the Peace River would increase downstream to the Town of Peace River due to the assumed coincident lateral peak inflows from the tributaries. Lateral inflows downstream of the Town of Peace River would be relatively less and hence the peak flows would gradually attenuate. The modelled peak flows at the Town of Peace River and Peace Point are 32,480 m^3/s and 25,210 m^3/s, respectively. The dikes at the Town of Peace River would be overtopped by about 2 m.

Table 37.21 Inundation Modelling Results for the Overtopping of the Earthfill Dam

Location	Distance from Site C (km)	Initial Water Level [a] (m)	Max. Water Level (m)	Water Level Increase (m)	Flood Arrival Time [b] (hrs.)	Time to Flood Peak [b] (hrs.)	Peak Flood Flow (m^3/s)
Site C Dam	0	–	469.8	–	–	–	111,160
Old Fort	7	406.5	429.4	22.9	0.6	2.8	106,610
Taylor	18	405.6	422.3	16.7	1.0	3.8	97,760
B.C.–Alberta border	62	387.2	409.6	22.4	3.6	8.6	75,020
Dunvegan	191	348.3	364.1	15.8	8.2	16.3	61,920
Town of Peace River	291	317.8	328.5	10.7	12.7	23.7	67,000
Carcajou	546	262.3	274.2	11.9	24.6	38.6	51,680
Fort Vermilion	726	250.8	259.5	8.7	37.0	60.5	42,520
Peace Point	1,031	216.2	223.1	6.9	74.4	122.5	29,830

NOTES:
[a] Prior to dam breach
[b] Start of Site C Dam breach

The peak outflow for the Site C probable maximum flood failure scenario would be 111,160 m^3/s and would attenuate to 75,020 m^3/s at the B.C.–Alberta border and 67,000 m^3/s at the Town of Peace River.

Peak water levels would be 17 m to 23 m above initial water levels within B.C. and 7 m to 16 m above initial water levels near the populated areas in Alberta, and the dikes at the Town of Peace River would be overtopped by 8 m.

Sensitivity Analysis and Uncertainty Assessment

Dam Breach Parameters

The sensitivity of the results of the analyses to the dam breach parameters was assessed.

A 50% increase in the breach width for the probable maximum flood overtopping failure scenario resulted in a 40% higher peak dam breach discharge, from 111,160 m^3/s to 156,460 m^3/s. However, the peak discharge would be attenuated to 69,980 m^3/s at the Town of Peace River (approximately 4% higher than shown in Table 37.21). Peak water levels at key downstream locations within B.C. would be approximately 2.6 m to 3.3 m higher with the increased breach width. Peak water levels would be 1.1 m and 0.4 m higher at Dunvegan and the Town of Peace River, respectively, with no increased flooding below the Town of Peace River.

A piping failure scenario was evaluated to test the sensitivity of the probable maximum flood failure assumptions. Piping is internal erosion of an earthfill dam leading to the formation of a “pipe” through the dam that eventually leads to a progressive failure.

Piping failure was assumed to initiate on the day of peak probable maximum flood inflow with no blockage of the spillway. Similar tributary flow assumptions were made as the probable maximum flood overtopping failure scenario. Since the maximum reservoir

level for a piping failure scenario would be less than the overtopping case, peak downstream water levels would either be slightly lower or very similar to peak water levels presented in Table 37.21.

The Site C sunny day failure scenario was re-evaluated with breach parameters obtained from the Von Thun and Gillette equation (Von Thun and Gilette 1990). A breach width of 189 m (17% larger than the USBR estimate) and a breach formation time of 0.9 hours (twice as fast as the USBR estimate) were assessed. The peak dam breach outflow was increased by about 15%. However, at the Town of Peace River the peak discharge would be increased by only 4% due to attenuation along the Peace River.

Tributary Discharges

The sensitivity of assumed tributary flows on downstream inundation was tested for the sunny day and probable maximum flood failure scenarios. The sunny day failure scenario was re-evaluated by using twice the annual average tributary flows. The higher tributary flows resulted in an increase in peak water levels at key locations along the Peace River by about 0.1 m, with no minimal in the extent of inundation.

The probable maximum flood failure scenario was re-evaluated with 25% lower tributary discharges than the assumed 200-year or historical maximum daily flows. Reduced tributary discharges resulted in 0.15 m to 0.4 m reduction in peak water levels in B.C. and 0.5 m to 0.8 m reduction in peak water levels at key locations in Alberta. There were nominal reductions in the maximum discharge at key locations for the probable maximum flood failure scenario; for example, from 75,020 m^3/s to 73,340 m^3/s at the B.C.–Alberta border, and from 67,000 m^3/s to 62,040 m^3/s at the Town of Peace River.

Roughness Coefficients

As stated above, the MIKE 11 model was calibrated for the Peace River system using Manning's roughness coefficients in the range of 0.02 to 0.03 in the river, and 0.07 to 0.1 for overbank areas. For sensitivity testing, the MIKE 11 model was rerun with 20% higher roughness coefficients for the Site C probable maximum flood (no failure) scenario. The modelled results indicate that the peak flood levels at key locations in the Peace River would be approximately 1 m higher within B.C. and 0.6 m to 1.4 m higher in Alberta.

37.2.2.3.3 Downstream Extent of Studies

The EIS Guidelines require a description of the effects of a dam break at Site C by tabulating the expected flood arrival time and water surface elevation at downstream communities along the Peace River to Peace Point, Alberta until the estimated water surface is within the estimated 200 year flood level (the level used in British Columbia to delineate natural flood hazard areas).

The peak flow at Peace Point from the sunny day dam break shown in Table 37.18 would be less than:

- The flood of record at Peace Point, which was 12,600 m^3/s mean daily flow on June 17, 1990, measured at Water Survey of Canada gauge 07KC001

- The 200-year flood, which is estimated to be 14,000 m^3/s based on a flood frequency analysis using the log normal method for the daily flows from Water Survey of Canada gauge 07KC001 for the period 1968 to 2012

For the PMF, it is not pertinent to extend the analysis beyond Peace Point to a 200-year flood level, since the initiating event itself has an annual exceedance frequency of less than 1/10,000, and the tributary inflows are 200-year inflows from Site C to the Town of Peace River with the historical maximum daily flows for the tributaries between the Town of Peace River and Peace Point.

37.2.2.4 Measures to Reduce Likelihood or Mitigate Potential Occurrence of a Dam Breach

37.2.2.4.1 Inflow Flood Forecasting and Flood Warning

BC Hydro operates a hydrometric data collection system in the Williston and Site C local catchments. Three additional data collection platforms were installed in the Site C local catchment in 2012 and three more will be installed in 2013 to provide more data for streamflow forecasting from the Site C local catchment. The data from the hydrometric data collection system would be used to improve the calibration of the watershed models and improve flood forecasting in the Site C local catchments.

As described in Section 37.1.9.2.5, the UBC Watershed Model is a continuous hydrologic model that provides continuous streamflow output for the duration of the meteorological input. BC Hydro runs the UBC Watershed Model to predict streamflow into Williston Reservoir. Calibrated UBC Watershed Models would be used to predict streamflow into Site C after it came into operation.

These calibrated models are capable of predicting flood flows from hydrometeorological data and would be used to predict inflows to the Project. After each flood these models would be updated with meteorological and streamflow data as a continuous process of improving the reliability of the models to predict floods.

Extensive heavy rainfall in the May through July period is usually the result of long-lived “cold lows” that persist in the foothills region for several days and draw moist Gulf of Mexico air into the area, causing prolonged heavy rainfall due to the presence of the low and upslope winds in the foothills and mountains. With or without contributions from snowmelt, these “cold lows” are the most common cause of flooding along the rivers flowing eastward (Section 37.1.9).

With modern weather system tracking, the development of such severe weather conditions would be monitored on a real time basis. The River Forecast Centre of the B.C. Ministry of Forests, Lands and Natural Resource Operations collects and interprets snow, meteorological and streamflow data to provide warnings and forecasts of stream and lake runoff conditions around the province. The River Forecast Centre issues flood advisories and warnings.

During construction flood forecasts would be used so that operating personnel at Peace Canyon Dam are on alert and ready to reduce discharges prior to flood arrival at the dam site. Reducing the Peace Canyon discharges earlier than described in Section 37.1.8.3 would result in lower headpond levels when the flood arrives, which would reduce the peak water levels and reduce the risk of overtopping. As shown in

Figure 37.61, it would take approximately four days for the headpond levels to peak after the start of the flood from the Site C local catchment. In the event that a larger flood than the capacity of the diversion works was forecast, there would be sufficient time to implement the emergency preparedness plans and evacuate potentially flooded areas downstream.

The governing condition for the probable maximum flood would be a storm producing the 100-year return period rainfall over the entire Site C catchment (severe event), followed by the probable maximum precipitation (an extreme event with 1.5 times the average 100-year return period rainfall) centred over the Site C local catchment starting four days after the end of the first storm (Section 37.1.9.2). Due to the severity of the weather conditions leading to a probable maximum flood, there would be sufficient time to implement the emergency preparedness plans and evacuate potentially flooded areas downstream.

37.2.2.4.2 Spillway Gate Reliability

The probable maximum flood failure scenario is based on a very conservative assumption of malfunction or debris blockage of all spillway gates so that there would be no spillway discharge, even over the auxiliary spillway, and the entire flood would flow over the top of the earthfill dam. This assumption is reasonable to define worst-case flooding for emergency preparedness planning purposes, but is not realistic.

BC Hydro has adopted reliability principles for the design, operation, maintenance, inspection, and testing of flood discharge gate systems to attain and maintain reliability.

The reliability target for the spillway gate systems at the Project would be a probability of failure on demand during a severe flood in the order of 1/1,000 to 1/10,000.

The measures used to obtain this level of reliability include:

- Equipment and defence in depth:
  - Use of well-proven equipment with proven track records
  - Any single random failure of gate operating equipment, control, or instrumentation should not prevent gate operation
  - Redundancy incorporated in the design of gate systems
  - Segregation and diversity as appropriate to protect against common cause failure
  - Gate equipment designed to avoid potential hazards and failures and maintain reliable operation through inherent features without reliance on control action or personnel intervention
  - Conservatively designed gate equipment with significant margins allowed for both predicted duty and abnormal situations
  - In a multiple gates system, the essential services and controls for individual gates segregated and protected from common cause failure
  - Local control of the flood discharge gate systems available, separate from buildings housing the main controls, such that the occurrence of hazards should not prejudice adequate means of control

- Hazards:
  - Design of gate equipment accounts for all external and internal hazards
  - Layout of system equipment and related services to minimize the effects of hazards
  - Gate operating and control equipment should be protected against natural and human hazards
- Control, monitoring, and instrumentation:
  - Equipment failure, gate movements, and other actions on the gate system communicated to the appropriate personnel when, or shortly after, they occur
  - Flood discharge gate systems to be operated by the standard controls and according to the established procedures
- Maintenance, inspection, and testing:
  - Management of equipment maintenance emphasizes the safety critical nature of the systems and adopts a proactive approach to achieving and maintaining a highly reliable performance
  - Preventive (time-based) maintenance carried out on critical items, plus regular functional checks to detect degradation of components
  - Abnormal conditions of equipment investigated promptly, and urgent action taken where necessary, to restore the system to a fully reliable state
  - The design of equipment and control interfaces incorporates good ergonomics practice to facilitate reliable operation, fault recovery, and ease of maintenance
  - Critical tasks performed under the guidance of written procedures
  - Flood discharge gate systems regularly inspected and tested
- Training:
  - Staff required to perform tasks to be trained and certified competent
  - Staff have regular hands-on experience in performing the tasks or receive periodic refresher training
  - Regular emergency exercises performed to practice the use of procedures and provide more realistic opportunities for staff training

High gate reliability would materially reduce the risk of overtopping of the earthfill dam during a large flood.

Further, in the event that a spillway gate was inoperable during a flood, a spillway undersluice in that bay could be operated to offset the loss of capacity from the spillway gate.

37.2.2.4.3 Debris management

Volume 1 Appendix A Vegetation, Clearing, and Debris Management Plan describes the waterborne debris clearing strategy that would be implemented during construction and the first five to 10 years of reservoir operations.

The primary debris management strategy for Site C during operation would be debris containment on the reservoir and regular removal rather than debris passage through the spillway. This is consistent with BC Hydro's approach on other reservoirs.

Debris containment would consist of a primary cable-reinforced debris boom for debris collection and containment just upstream of the approach channel. Where possible, the boom would be a shear boom directing debris to the upstream face of the earthfill dam, where a dedicated debris removal facility would be located.

The primary debris boom design would follow BC Hydro practice for a cable reinforced double stick log boom. The debris boom, while a key component of the debris management strategy, would be combined with a number of secondary components to effectively manage debris. The secondary components would include:

- Development of comprehensive proactive debris management strategy to address the following:
  - Debris inventory (estimates of current and potential future debris quantities)
  - Debris management approaches, e.g., limiting debris inflow, debris control and collection, debris disposal, and discharge facility requirements
- Inventorying and managing debris before it reaches the main boom, which could include a second upstream shear boom for debris collection and debris removal

Debris that bypasses the debris boom would require removal in the approach channel or passage by the spillway. To facilitate debris removal in the approach channel, the gantry used for the installation of intake and spillway stoplogs would be equipped with a jib arm outfitted for debris removal.

Spillway features that facilitate debris passage or mitigate potential blockage are:

- Wide spillway gates
- Wide overflow auxiliary spillway
- Pier noses that extend upstream from the gates to retain debris before it can reach and potentially block the gates

Debris testing in the physical hydraulic model of the approach channel and spillway indicates that the above features would be successful in preventing blockage by debris.

Successful debris management would materially reduce the risk of overtopping of the earthfill dam during a large flood.

37.2.2.4.4 Powerhouse Operation

As discussed in Section 37.2.2.3.1, for sizing the spillway capacity, it was assumed that the powerhouse would not operate during the probable maximum flood. However, to the extent possible, the powerhouse would be operated during large flood to provide additional discharge capacity and possibly draw the reservoir down prior to flood arrival. Powerhouse operation during the probable maximum flood would reduce the risk of overtopping.

37.2.2.4.5 Increasing Cofferdam Freeboard

Measures that could be implemented to reduce the likelihood of damage or overtopping of the Stage 2 upstream cofferdam during construction include adding temporary walls using concrete lock blocks or similar materials to reduce wave overtopping.

37.2.2.4.6 Earthquake Design

As shown in Table 37.18, due to the proximity of Old Fort and Taylor and the resulting short flood arrival times, there would be limited time available for warning and evacuation for the sunny day failure. There would be additional time available for warning and evacuation downstream of the B.C.–Alberta border. For example, the flood arrival times at Dunvegan and the Town of Peace River are about 10 and 16 hours, respectively, from the start of dam failure.

As described in Section 37.1.6.1, the deformations of the reservoir retaining structures during the earthquake design ground motion would be small. The earthfill dam and RCC buttress could withstand much larger movements without releasing the reservoir.

As part of the design, larger ground motions than the 1/10,000 annual exceedance frequency earthquake design ground motion are being considered to provide robust structures that can withstand larger ground motions than recommended by the Canadian Dam Association Dam Safety Guidelines.

The robustness of the seismic design reduces the risk of a sunny day failure due to earthquake.

37.2.2.4.7 Sunny Day Failure Due to Piping

The design of the earthfill dam and the materials used to construct it are described in Volume 1 Section 4 Project Description. The dam has been designed to reduce the risk of piping as follows:

- Based on testing in a large permeameter at hydraulic gradients several times greater than expected in the core of the dam, the glacial till in the impervious core would be internally stable and resistant to piping
- The wide filter zones on the downstream side of the impervious core that are widened at the abutments
- Foundation treatment including dental excavation and concrete, and grouting would mitigate piping in the foundation

Failures due to piping rarely occur without warning. Dam surveillance and monitoring pursuant to the operation, maintenance, and surveillance manual would likely see evidence of distress well in advance of a potential failure.

Measures to mitigate potential piping would include drawing down the reservoir using the spillway undersluices. Lowering the reservoir level would mitigate in two ways:

- Reducing the gradients through the dam and foundation, reducing piping potential, which is a function of gradient
- The dam breach flood wave and inundation levels downstream would be reduced due to lower volumes in the reservoir

Given the likely warning of potential piping problems, there would be time to implement the emergency preparedness plan and evacuate areas that could be flooded downstream.

37.2.3 Potential Accidents and Malfunctions During Construction

The following accidents and malfunctions that could occur during construction are discussed in this section:

- Release or spills of chemicals and hazardous materials
- Containment of pond leakage or failure
- Sediment control failure
- Fire and explosion

Volume 5 Section 35 Summary of Environmental Management Plans discusses the environmental management framework for the Project. The construction environmental management plans would be designed and implemented to reduce the likelihood or mitigate the potential occurrence of the above accidents or malfunctions. The emergency response plan would provide the framework for managing accidents and malfunctions that may occur during construction.

The following descriptions of accidents and malfunctions should be read in the context of the environmental management plans discussed above.

The following sections describe the measures to reduce the likelihood or mitigate the potential occurrence of the above accidents or malfunctions.

37.2.3.1 Release or Spills of Chemicals or Hazardous Materials

Large quantities of chemicals and hazardous materials would not be stored on the construction sites; rather, they would be brought to site in quantities similar to the rate at which they are consumed. Chemicals would be transported to site either by rail or by truck and stored in appropriate containers according to applicable standards and regulations. Temporary construction infrastructure that would store chemicals and hazardous materials include:

- Vehicle maintenance facilities – chemicals stored would include lubricants, antifreeze, petroleum products, and other materials required to maintain construction equipment
- Tank farm where diesel, gasoline, and other liquid fuels required for construction would be stored. Proper secondary containment such as double-wall tanks or lined containment areas would be installed to minimize fuel seepage into the environment.
- Explosive magazines for secure storage of ammonium nitrate and stick explosives
- Storage facilities for cement, fly-ash, and other concrete additives
- Waste treatment facilities – for camp, truck washing, and cement wastewater, as well as antifreeze, lubricants, and other liquid and solid wastes generated on-site

Secondary containment of hazardous substances would be provided as recommended in the Canadian Council of Ministers of the Environment (CCME) Environmental Codes of Practice (CCME 1993; CCME 1994)

Spills most likely to occur would be from activities such as refuelling. Fuel trucks would transport diesel from the tank farm to the various heavy equipment on-site. Pickup trucks and other more mobile equipment would refuel at the tank farm directly.

Other likely spill events would result from breaks in hydraulic lines of excavators or other large construction equipment. To minimize the effect of hazardous material spills, with the exception of the initial construction of the construction berms, all construction areas would be contained behind a dike or other confinement structure, and therefore the spill would not be exposed to the environment outside of the construction site.

All vehicles would be equipped with an appropriately sized spill kit, and personnel operating would have applicable training. Spills would be cleaned up as soon as possible.

37.2.3.2 Containment Pond Leakage or Failure

Containment ponds would be constructed to manage storm water runoff and wastewater collected on-site. A potential source of wastewater collected on-site would be from the cement plants. Both the storm water and cement plant wastewater containment ponds would be settlement ponds.

The water surface level and containment structure would be monitored on an ongoing basis for both its geotechnical stability and chemical properties. Should a containment pond leak or fail, the Project emergency response plan would be followed.

37.2.3.3 Sediment Control Failure

Management structures to control sediment from entering the Peace River and other watercourses would be installed, monitored, and maintained at various locations throughout the Project.

The magnitude and impact as a result of a failure to a sediment control structure would depend on the location and purpose of the structure. The impact to the environment in the event of a sediment control device failure may be the release of a pulse of sediment into the water.

37.2.3.4 Fire and Explosion

Fire control measures for the construction of the Project would be designed and implemented.

Fire controls at the dam site would include strategically positioned heat and smoke detectors and alarms, as well as automatic sprinklers and suitable extinguishers. Planning for work areas would ensure that personnel could easily evacuate in the event of an emergency.

Debris from clearing activities would be managed to minimize risk of fire and magnitude of fires should one occur. Construction sites would be cleared with adequate buffers to minimize wildfire spreading at the construction sites.

Explosives would be used at the dam site, and West Pine, Wuthridge, and Portage Mountain quarries. Although the majority of the rock that would be excavated at the dam site would be ripped with heavy equipment, drilling and blasting would be required for unweathered rock that is too hard to rip on the north and south bank rock excavations as well as in the diversion tunnels.

To minimize the risk of explosion, dynamite and detonators would be stored on-site in explosives magazines and, when required, would be loaded and transported to the blasting sites. Explosive magazines would be stored a safe distance from other facilities. Ammonium nitrate and fuel oil would be delivered by truck and only mixed during delivery down the blast hole.

37.2.4 Potential Accidents and Malfunctions During Operation

The following accidents and malfunctions that could occur during operation are discussed in this section:

- Release or spill of chemicals or hazardous materials
- Fire and explosion
- Transmission line or generating station outage

Dam Safety Incidents are defined as abnormal conditions or performance of the dam (including mis-operation or component failure) with the potential to jeopardize the safety of the dam but that, at this time, is not expected to lead to a breach of the dam. The Operations, Maintenance and Surveillance Manual described in Volume 1 Section 4 Project Description would contain requirements for increased surveillance after any Dam Safety Incidents and for the preparation of responses applicable to the incident to mitigate dam failure. The dam breach analyses described in Section 37.2.2 indicate the consequences if the responses are insufficient to prevent dam breach.

Volume 5 Section 35 Summary of Environmental Management Plans discusses the environmental management framework for the Project. The operation environmental management plans would be designed and implemented to reduce the likelihood or mitigate the potential occurrence of the above accidents or malfunctions. The emergency response plan would provide the framework for managing accidents and malfunctions that may occur during operation.

The following descriptions of accidents and malfunctions should be read in the context of the environmental management plans discussed above.

The following sections describe the measures to reduce the likelihood or mitigate the potential occurrence of the above accidents or malfunctions.

37.2.4.1 Release or Spills of Chemicals or Hazardous Materials

37.2.4.1.1 Overview

Large quantities of chemicals and hazardous materials would not be stored at the dam, generating station, and spillways but would be delivered to the site in the quantities required for operations and maintenance.

Hazardous materials that would have limited on-site storage are expected to include:

- Oils and lubricants
- Miscellaneous paints
- Solvents
- Varsol
- Lead-acid storage batteries
- Compressed gases including SF_6 for the gas-insulated switchgear
- Fuels, including diesel for backup power generation

Lubricants and hydraulic oils are used in governors, intake, and undersluice gates, and would be present in equipment bearing and gearbox oil reserves. Transformer insulating oils are typically Voltesso (used in each station service transformer and in the system power transformers). Equipment containing polychlorinated biphenyl (PCB), asbestos, and mercury, which were commonly used in electrical equipment in the past, would not be used by the Project.

Hazardous substances associated with transmission line operation include herbicides for vegetation management. Herbicides would be stored and used in accordance with manufacturer's written recommendations and BC Hydro management standards and practices.

Transportation, handling, storage, and disposal of hazardous substances, including hazardous wastes, would be done in accordance with BC Hydro management standards and practices.

Hazardous materials would be stored in a secure location in accordance with the manufacturer's instructions. In addition, all hazardous materials management would comply with the WCB Occupational Health and Safety Regulation.

Secondary containment of hazardous substances would be provided as recommended in the Canadian Council of Ministers of the Environment (CCME) Environmental Codes of Practice (CCME 1993; CCME 1994), and as required by applicable Fire Codes.

The dam, generating station, and spillways would have drainage and sump systems for all structures, including provisions for emptying the intakes, penstocks, and the spiral cases and draft tubes in the powerhouse. Sources of water with the potential for oil contamination would be directed to oil-water separator systems. Transformers would be equipped with spill containment systems that would drain fluids into oil-water separators. The oil-water separator systems would be regularly inspected and maintained. Oil would be collected from separators for recycling or off-site disposal at an approved disposal facility.

Oil would be used in major mechanical and electrical components of the generating station and spillways. The power transformers would be filled with electrical insulating oil as a heat transfer/cooling medium. The governors that would control and regulate the operation of the turbines would have self-contained hydraulic systems that include accumulators for storing pressurized hydraulic oil. The turbine and generator bearings for each unit would have their own lubricating systems. The intake gates would probably be hydraulically operated, but could be mechanical hoist operated. Each of these oil systems would have piping, the majority of which would be oil-filled at all times.

In addition to the oil in major components of the generating station, there would be small quantities of oil in crane gearboxes, spillway gate gearboxes, and in air compressors; oil would also be present in varying amounts in other mobile equipment and vehicles periodically brought onto the site for maintenance and general service purposes.

Measures incorporated into the design and construction of the generating station would include oil control and containment systems sized appropriately to address potential oil spill scenarios. Oil containment basins and oil-water separators having individual capacities to contain all of the oil used in the systems they serve would either be built as part of the powerplant structures or provided by commercially available tank systems. Under normal operating conditions, these systems would function to contain and separate all oil flowing into them. Any oil leaking from the hydraulic hoist for the intake gates would float on the surface of the gate well where it could be cleaned up with sorbent materials.

37.2.4.1.2 Worst Case Oil Spill

Depending on their design, the generator step-up transformers on the draft tube deck of the powerhouse would contain up to 90,000 L of oil and would be located close to the river. The transformers would be situated over sumps filled with gravel and provided with a deluge system. Each step-up transformer would be provided with a drainage system that would have a dedicated oil-water separator with the capacity to simultaneously:

- Contain the total volume of oil from one step-up transformer
- Simultaneously process the step-up transformer oil together with the deluge flow from the step-up transformer fire protection system
- The combined maximum expected rain and snowmelt inflow

In considering the operation of the powerplant, all potential oil leaks and spills would be securely contained during all reasonably foreseeable conditions, with the possible exception of a loss of transformer insulating oil under the following scenarios:

- Spill onto the draft tube deck of the powerhouse during a transformer oil change
- Spill onto a road or parking area from a tanker truck
- Spill resulting from a transformer explosion

Transformer oil changes are likely to occur once every 40 to 50 years, with intermediate oil processing once every 7 to 10 years.

In the first scenario, a spill could result from an improperly connected hose, or a hose or hose connection failure. Spilled transformer oil would flow to the transformer oil containment system and oil-water separator, or to powerhouse drains and their oil-water separator. Given the supervision that would be provided during a transformer oil change, an oil spill with a volume greater than the capacity of the oil-water separators, resulting in a spill to the river, is considered to be extremely unlikely.

In the second scenario, a spill could result from a ruptured tank or broken or leaky valve on a tanker truck travelling to the powerhouse or parked adjacent to the powerhouse. Spilled oil would have the potential to flow or seep to the Peace River. Large tanker trucks are compartmentalized, with individual tanks usually containing about 10,000 L. Tanker transfer systems include valves on each compartment and valves would be

provided as part of the transformer oil containment and cooling system. Under a worst-case remote probability scenario, it is possible that 10,000 L of oil could be lost from a tanker truck. Oil handling procedures would require the provision of a suitable spill kit with containment socks/booms. In spite of these and other precautions that would be taken, it is conceivable that some oil could reach the river. Given the low probability of a spill under this scenario, and considering all precautions that would be taken, it is likely that no more than 10% of the worst-case oil spill (i.e., about 1,000 L) would reach the river.

In the third scenario, a spill could result from a transformer fire and explosion. Transformer oil would be spilled onto the draft tube deck and some could oil could be projected to the river. Oil spilled onto the draft tube deck would flow to the oil-water separators and not enter the river. It is possible that 10% of the oil in the transformer could be ejected out of the blast enclosure in this very rare event and enter the river. Since transformer oil cooling system operation and, in particular, oil temperature would be closely and continuously monitored, the probability of such an explosion will be extremely low.

Based on the above considerations, a release to the surrounding environment of up to about 9,000 L of transformer insulating oil is a remote possibility. This oil would flow or seep into the river system and would be dispersed by river flow and carried downstream. The type of oil, its low viscosity, and the quantity involved are such that a spill would not result in a concentrated slick of oil passing downriver. Since the oil involved would be relatively ‘light’, some oil sheen would be observed downstream as the oil rises to the surface and is caught in low-velocity areas and back eddies.

In the scenario described above, the characteristics of the oil and the large river surface over which the spill would be spread are such that the relatively small volume of oil would produce a very thin sheen that would likely be detectable. Wind, evaporation, and the action of sunlight on the oil sheen should soon disperse and break down any residual oil within one to three days after the spill. The spilled oil should not be damaging to riparian habitat. The low toxicity of the oil and its distribution would be such that it should not be damaging to fish or wildlife, including waterfowl.

Post-spill investigation and monitoring programs, involving over-flying the area and conducting selective shoreline and in-river monitoring, would be initiated to check potential effects. This program would be extended as necessary to mitigate any local adverse effects identified by monitoring, such as effects on waterfowl. Past experience with small spills of this nature suggests that only a small amount of the spill would be recoverable, and that mounting a full-scale spill containment and collection operation would be largely ineffective due to the very rapid dispersion of the oil. Effort would be better spent on isolating and cleaning up any pockets of oil detected while over-flying the area and on attending to any identified adverse effects on wildlife.

37.2.4.1.3 Release of SF_6 Gas

Sulphur hexafluoride (SF_6) is an inorganic, colorless, odorless, and non-flammable greenhouse gas that is a dielectric material in gaseous state. SF_6 would be used as an electrical insulator in the high voltage circuit breakers and switchgear connecting the unit transformers on the draft tube deck to the transmission lines.

The circuit breakers and switchgear would be designed in a compartmentalized manner to limit the amount of SF_6 that could escape in the event of a leak from the equipment.

BC Hydro has established practices for the handling and management of SF_6 in an environmentally sound and responsible manner.

In summary, the main principles for managing SF_6 are:

- Purchasing of SF_6 is centralized, so that it is captured by BC Hydro's SF_6 tracking system
- Using SF_6 in a manner that minimizes risks to human health and the environment
- Developing a loss reduction strategy that identifies SF_6 loss reduction objectives and targets
- Training employees in all procedures related to the way they have to handle or use SF_6 in the course of their duties
- Minimizing the intentional venting of SF_6, except for small amounts needed to do gas sampling or to protect human life in an emergency
- Handling and using SF_6 to minimize contamination with air, moisture, oil, or other unwanted substances
- Recovering for reuse or reclaiming as much SF_6 as possible when equipment is maintained or decommissioned
- Tracking use of SF_6
- Storing contaminated SF_6 until there is a way to separate the SF_6 from the contaminants
- Treating SF_6 that cannot be separated from contaminants in a way that destroys the SF_6 and removes environmental concerns associated with it

37.2.4.2 Fire and Explosion

Fire control measures for operation of the Project would be designed and implemented in compliance with BC Hydro management standards and practices.

Fire controls at the dam, generating station, and spillways would include:

- Strategically positioned heat and smoke detectors
- Alarms to trigger fire warning and response
- Automatic water deluge system on each generator and power transformer
- Sprinklers over areas containing combustible materials
- Readily available and fully functional dry chemical hand-held fire extinguishers, pressurized water extinguishers where appropriate, and retardants located throughout the facility, particularly in areas used for storing paints, lubricants, oil, diesel, and gasoline or other combustible materials
- Pressurized fire hydrant connections at strategic locations throughout the facility

Effective response protocols would be developed to respond to fire according to BC Hydro management standards and practices. Response would provide for:

- Shut down of part or all of the facility, depending on the severity of a fire
- Call out for external emergency assistance
- Evacuation of personnel

Operating personnel would receive effective fire response training.

Transmission system fire could potentially originate from such sources as lightning, arcing, a downed line, or other inadvertent contact between the energized line and surrounding vegetation (e.g., as a result of a danger tree falling onto the line).

Preventive measures include vegetation management (e.g., to maintain a safe distance between energized lines and surrounding trees) and periodic inspection of poles and other transmission system elements. Fires would most likely be reported by observers to local departments, or to the provincial fire centre, who would coordinate the response.

The effects of a fire at the powerplant or along the transmission line corridor would mostly be contained locally, at the scene. A major fire at the powerplant would result in smoke in the local atmospheric environment and it is possible that oil and other products and materials in the powerplant would burn and release airborne contaminants. However, deluge systems and other fire controls would be in place to quench starting fires, and it is considered unlikely that an initial fire would develop into a major fire. A major fire at the powerplant would be no worse than a major fire at any industrial facility, and area fire departments and firefighters are equipped to deal with such occurrences. A major fire at the powerplant and a fire along the transmission line corridor that affected transmission would probably result in a station shutdown.

37.2.4.3 Transmission Line or Generating Station Outage

As described in Section 11.4 Surface Water Regime, the proposed minimum flow downstream from the Project is 390 m^3/s.

The Project would be capable of maintaining this minimum flow under any foreseeable equipment malfunctions:

- The minimum flow is less than the discharge capacity of one of the six generating units. Hydroelectric generating units are very reliable. The generating station would be capable of passing the minimum flow after a sudden outage of up to four units, with one unit out of service for an annual inspection and maintenance.
- There are two 500 kV transmission lines, so an equipment malfunction on one line would not lead to a total generating station outage
- The spillway gate controls would open spillway gates as required to pass the minimum flow in the event of a total generating station outage
- Backup power, e.g., a diesel generator, would be provided for the spillway gates

37.2.5 Effects of Accidents and Malfunctions on Valued Components

The results of the potential effects of accidents and malfunctions on valued components are summarized in Figure 37.22. The effects were categorized as follows:

- 0 – No interaction is predicted
- 1 – An adverse effect may result from an interaction, but standard measures to avoid or minimize the potential effect are available and are well understood to be effective, and residual effects are negligible
- 2 – Interactions may result in an adverse effect and mitigation measures are not well understood to be effective

For the interactions categorized as “2”, the following subsections describe the reasons for those categorizations.

37.2.5.1 Construction Phase

37.2.5.1.1 Breach of Stage 2 Upstream Cofferdam

A breach of the Stage 2 upstream cofferdam would result in a flood wave that would cause temporary incremental flooding of areas between the maximum water levels presented in Table 37.14, which would naturally occur, and the maximum water levels presented in Table 37.16, which would result from a breach. In addition to temporary flooding, the flood wave from a breach would cause erosion in overbank areas and changes to the river channel due to erosion as well as the deposition of alluvial materials and debris. Erosion of the toe of steep banks could trigger landslides.

Fish and fish habitat would be adversely affected:

- By changes to the river channel
- By a pulse of sedimentation due to overbank erosion and landslides
- Because fish in the flooded areas would be displaced, injured, or killed by the force of the flood wave or by impingement on obstacles, or stranded when the flood waters recede as the wave moves downstream

Fish stocks would gradually recover as fish move in from tributaries and from unaffected areas upstream and downstream.

Vegetation and ecological communities would be adversely affected, as flooding and erosion of overbank areas would damage and potentially destroy vegetation and ecological communities. In the flooded areas, some vegetation such as grasses and shrubs would regrow fairly quickly, but damaged or uprooted trees would take longer to re-establish. Some ecological communities might not re-establish.

Wildlife within the flooded area could be killed or injured by the force of the flood wave or impingement against obstacles, or could be drowned. Wildlife would move back into the area after the flood had receded and vegetation re-established.

Current use of lands and resources for traditional purposes would be adversely affected by the adverse effects on fish, wildlife, and vegetation described above. The adverse effects would diminish with time as the fish, wildlife, and vegetation recovered.

Agriculture in the flooded areas would be adversely affected by:

- Erosion of agricultural land
- Crop damage
- Cattle killed or injured by the force of the flood wave or impingement against obstacles, or drowning

Depending on the amount of erosion, effects on agricultural land or crops may last one or more seasons.

Oil and gas infrastructure located downstream within the flooded area would be damaged, but could be repaired and back in operation within a relatively short period of time.

The harvest of fish and wildlife resources would be adversely affected by the adverse effects on fish and wildlife described above. The adverse effects would diminish with time as the fish and wildlife resources recovered.

Outdoor recreation and tourism would be adversely affected due to the effects on wildlife, fisheries, navigation, visual aesthetics, and damage to downstream recreation sites. Damaged recreation sites would be repaired, and outdoor recreation and tourism would recover as the affected valued components recovered.

Navigation on the river downstream would be temporarily impaired as a result of debris deposition and changes to the river channel.

The visual and aesthetic resources of the river valley downstream would be adversely affected by erosion, deposition of debris, and damage to vegetation and infrastructure in the flooded areas, which would alter the landscape. The adverse effects would diminish with time as the vegetation recovered and damages were repaired.

It is likely that some downstream heritage sites would be adversely affected by erosion and deposition of debris.

Human health would be adversely affected because residences and recreational areas are located within the flooded area. The emergency preparedness plan would be implemented, and people within the potentially flooded areas would be evacuated. Nevertheless, some loss of life or injury is possible and some evacuees may be affected by post-traumatic stress.

37.2.5.1.2 Fire of Explosion

During the construction phase, fires caused by the Project would most likely start within a cleared construction site and, therefore, would only have an adverse effect on a valued component if the fire spread outside of the cleared area.

If an accidental explosion occurred, it would not cause an adverse effect on valued components unless the explosion caused a fire that spread from the construction area.

Vegetation and ecological communities would be adversely affected by fires that spread from a construction site. The adverse effects would diminish with time as the vegetation recovered.

Wildlife would be adversely affected if an extensive fire spread from a construction site. The adverse effects would diminish with time as the wildlife returned to the area.

The current use of lands and resources by Aboriginal persons for traditional purposes may be adversely affected by the adverse effects on vegetation and ecological communities and wildlife. The adverse effects would diminish with time as wildlife move back into the area and as vegetation recovered, although some ecological communities could be lost.

Visual and aesthetic resources may be adversely affected by the burnt vegetation. The adverse effects would diminish with time as the vegetation recovered.

37.2.5.2 Operations Phase

37.2.5.2.1 Breach of Earthfill Dam

A breach of the earthfill dam would result in flood wave that would cause:

- Temporary flooding of the areas between the initial water levels and the maximum water levels presented in Table 37.18 for a sunny day failure
- Temporary incremental flooding of the areas between the maximum water levels presented in Table 37.19, which would occur during the probable maximum flood under natural conditions, and the maximum water levels presented in Table 37.21, which would result from a breach due to overtopping of the earthfill dam during the probable maximum flood

Breach of the earthfill dam would rapidly drain the reservoir, exposing un-vegetated areas to erosion by wind and precipitation. The rapid drawdown of the reservoir would likely cause landslides in areas where colluvium and glaciolacustrine deposits had been saturated by the reservoir. Some of the material deposited in reservoir arms would erode, adding to the sediment load in the water outflowing from the breach.

The physical effects of the temporary flooding downstream (i.e., erosion, the deposition of alluvial materials, and debris) would be as described for the breach of the Stage 2 upstream cofferdam; however, the extent of these effects would be much greater.

The effects of a breach of the earthfill dam on the valued components where the interaction is categorized as “2” in Figure 37.22 would be similar to the effects described for a breach of the Stage 2 upstream cofferdam, except that the extent would be much greater. Effects additional to those described for the Stage 2 upstream cofferdam are described below.

Greenhouse gases would be emitted during reconstruction of the dam and damaged infrastructure, and during restoration of aquatic and terrestrial habitat. There would also be a short-term increase in greenhouse gases due to the exposure or previously submerged vegetation, which would decompose and release greenhouse gases at a greater rate. After reconstruction and refilling of the reservoir, the greenhouse gas emissions from the reservoir would return to the levels prior to the breach.

There would be additional adverse effects on oil, gas, and energy because a breach of the earthfill dam would likely damage or destroy the Spectra pipeline that crosses the Peace River at Taylor. The pipeline crossing could be repaired or replaced shortly after the breach, so that the effect would be short term.

Navigation upstream would be adversely effected by draining of the reservoir, which would leave navigation infrastructure on the former shoreline unusable. As well, the bathymetry of the river and the stability of the banks would be unknown and would present hazards to navigation.

There would be additional adverse effects on visual and aesthetic resources upstream of the dam due to draining of the reservoir leaving the formerly inundated zone exposed, un-vegetated, and susceptible to erosion. After reconstruction and refilling of the reservoir, the upstream visual and aesthetic resources would return to the levels prior to the breach.

There would be an adverse effect on community infrastructure because the flood wave from a breach of the earthfill dam would overtop the dikes at the Town of Peace River, which would damage or destroy infrastructure in the flooded area.

There would be an adverse effect on transportation, as the CN rail bridge across the Peace River southeast of Fort St. John and the Highway 29 bridge across the Peace River at Taylor would be damaged or destroyed. It is also likely that the bridges at Clayhurst would be seriously damaged or destroyed by a breach of the earthfill dam.

37.2.5.2.2 Release or spills of chemical and hazardous materials

As described in Section 37.2.4.1, the worst-case oil spill would be 9,000 L into the river from the explosion of a generator step-up transformer on the draft tube deck of the powerhouse. Due to the light nature of the oil, any adverse effects on the valued components will be of short duration, as any oil that is not collected will disperse and break down fairly rapidly.

Fish and fish habitat could be adversely affected by oil.

Riparian vegetation and ecological communities that come into contact with the oil could be adversely affected.

Current use of lands and resources for traditional purposes could be adversely affected to the extent that oil adversely affects fish and riparian vegetation.

37.2.5.2.3 Fire or Explosion

A fire or explosion during operation of the Project would only have adverse effects on the valued components if an extensive fire spread from the source.

Adverse effects would be similar to those described for the construction phase.

37.3 Cumulative Environmental Effects

37.3.1 Approach to Assessment of Cumulative Effects of the Project

Section 19(1) of the *Canadian Environmental Assessment Act* (CEAA 2012) requires the assessment of "… any cumulative environmental effects that are likely to result from the designated project in combination with other physical activities that have been or will be carried out". In accordance with this statutory requirement, Section 8.5.3 of the EIS Guidelines requires "… an assessment of the cumulative effects that are likely to result from the Project in combination with other projects or activities that have been or will be carried out …" In order to meet this requirement, and to predict the cumulative effects of

the Project on each VC where the Project is likely to result in a residual adverse effect, a Regional Assessment Area (RAA) and three "Cases" were developed. These are described below.

37.3.1.1 The Boundaries of the RAA for Each VC

The spatial boundaries of the RAA for each VC (see Table 37.22) have been developed based on consideration of:

- Where possible interactions with other projects or activities overlap spatially or temporally
- Ecologically defensible boundaries (e.g., wildlife range boundaries)

Table 37.22 Regional Assessment Areas

Valued Component	Regional Assessment Area
Fish and Fish Habitat	Peace River from Peace Canyon Dam, B.C. to Vermilion Chutes, Alberta, which is a distance of approximately 865 km
Vegetation and Ecological Communities	The proposed dam, reservoir, transmission line, Highway 29 realignment, temporary access roads, and quarries occur within five Wildlife Management Units – designated 7-31, 7-32, 7-33, 7-34, and 7-35, which includes most of the Peace Lowlands ecosection
Wildlife Resources	Vegetation and Ecological Communities RAA, as described above
Greenhouse Gases	National
Local Government Revenue	City of Fort St. John, District of Taylor, District of Hudson's Hope, District of Chetwynd, City of Dawson Creek, and Peace River Regional District
Labour Market	Peace River Regional District, Northern Rockies Regional Municipality, and Fraser-Fort George Regional District
Regional Economic Development	Peace River Regional District, Northern Rockies Regional Municipality, and Fraser-Fort George Regional District
Current Use of Lands and Resources for Traditional Purposes	Fish and Fish Habitat RAA and Wildlife Resources RAA, as described above
Agriculture	Peace River Regional District, the Northern Rockies Regional District, and the Peace River Agricultural Region (Statistics Canada Agricultural Region 8)
Forestry	Dawson Creek Timber Supply Area, Fort St. John Timber Supply Area, and Peace River supply block of Timber Forest Licence 48
Oil, Gas, and Energy	Project activity zone, area within Five-Year Beach Line, and Spectra Energy's Taylor water intake
Minerals and Aggregates	Fort St. John, District of Hudson's Hope, District of Taylor, Peace River Regional District, Area "C" [a]
Harvest of Fish and Wildlife Resources	Peace River Regional District
Outdoor Recreation and Tourism	Peace River Regional District
Navigation	Navigation – Project activity zone, downstream to Peace Island Park, and the Shaftesbury and Tompkins Landing ice bridges Aviation – Area from the North Peace Regional Airport (Fort St. John airport) to the crest of the potential Project construction site

Valued Component	Regional Assessment Area
Visual Resources	LAA and Visual Landscape Inventory viewpoints within or adjacent to Project activity zone
Population and Demographics	Peace River Regional District
Housing	Peace River Regional District
Community Infrastructure and Services	City of Fort St. John, District of Taylor, District of Hudson's Hope, District of Chetwynd, City of Dawson Creek, and Peace River Regional District
Transportation	Peace River Regional District
Heritage Resources	Project activity zone
Human Health	RAA corresponds to relevant technical study areas for air quality, noise, water quality, electric and magnetic fields, country foods, and mercury

NOTE:

[a] Area C is a regional district electoral area in the Peace River Regional District. The electoral area covers the area of Fort St. John.

37.3.1.2 The Three Cases

Baseline Case: The Baseline Case demonstrates the current status of the VC, which necessarily ensures that the effects of all projects and activities that "…*have been … carried out* …" are accounted for in the assessment of the cumulative effects of the Project.

September 5, 2012 was chosen to demarcate the Baseline Case from the future cases described below, because 1) this was the date that the EIS Guidelines were issued by the federal Minister of Environment and the Executive Director of the BCEAO, and 2) by this date, BC Hydro had already substantially developed the assessment of potential effects and cumulative effects of the Project.

Future Case without the Project: In order to predict the status of the VC, taking into account the residual effects of projects and activities that "have been" <u>and</u> "will be" carried out, the Future case without the Project was developed.

Projects and activities that "will" be carried out are necessarily more certain than the Project itself. Consequently, available information about the residual effects of projects and activities that are at least as foreseeable as the Project was gathered. The list of those projects and activities was developed in accordance with the requirements of Section 8.5.3.2 of the EIS Guidelines.

For each VC:

- The projects and activities on the Project Inclusion List found within the RAA for that VC were identified
- Using the information available, the potential residual effects of those projects that may overlap in time and space with the potential residual effects of the Project were identified
- The status of the VC, taking into account the effects of projects and activities that have been and will be carried out, was predicted by considering the impact of those residual effects on the Baseline Case for the VC

Project Case: To demonstrate the cumulative effects that are likely to result from the Project, the Project Case for each VC was developed. In this case, the potential cumulative effects of the Project on each VC have been predicted by taking into account the potential residual effects of the Project that are likely to combine in time and space with those identified in the Future Case without the Project.

37.3.1.3 The Project Inclusion List

Screening criteria were used to identify other projects and activities for consideration in the cumulative effects assessment (Table 37.23).

Effects from potentially overlapping projects or activities that are recently operational may not be fully reflected in baseline conditions. For each VC, those projects and activities have been evaluated to determine whether they should be included in Baseline Case or the Future Case without the Project and the Project Case.

Table 37.23 Screening Criteria Used to Identify Other Projects and Activities For Consideration in the Cumulative Effects Assessment

Type of Overlap	Excluded	Included
Spatial	• Project or activity is outside the largest RAA. By using a conservatively large RAA, all potential cumulative effects for VCs will be captured. • Project or activity is located in Alberta beyond 100 m of the Peace River high water mark or further downstream than Fort Vermilion (i.e., downstream of the RAA boundary of the Fish and Fish Habitat VC).	• Project or activity is within the largest RAA. • Project or activity is located in Alberta within 100 m of Peace River high water mark and as far downstream as Fort Vermilion.
Temporal	• Project was in operation or activity was occurring prior to September 5, 2012; therefore, associated residual effects may be reflected in baseline case conditions.[a] • Project or activity is not reasonably foreseeable (i.e., not as likely to proceed as Site C Clean Energy Project).	• Active projects in federal or provincial environmental assessment or other regulatory process. • Approved projects and activities that are: o not constructed o under construction or o constructed, but not operational • Project or activity is reasonably foreseeable (i.e., at least as likely to proceed as the Site C Clean Energy Project).

NOTES:

[a] Effects from potentially overlapping projects or activities that are recently operational may or may not be fully reflected in baseline conditions. Those projects and activities have been evaluated in the VC cumulative effects sections to determine whether they should be included in Baseline Case or the Future Case without the Project and the Project Case.

Projects and activities that may be carried out within the RAA in the Future Case without the Project and in the Project Case were identified in accordance with Section 8.5.3.2 of the EIS Guidelines. The following information sources were reviewed to identify other

projects and activities located within the largest RAA to be taken into account in the Future Case without the Project and in the Project Case:

- Registered active projects listed on the BCEAO and CEA Agency websites, including hydroelectric projects such as the Dunvegan Hydroelectric Project (Volume 2 Sections 10.5.2.2 and 10.5.2.10)
- Registered oil and gas applications listed on the British Columbia Oil and Gas Commission or National Energy Board websites (Volume 2 Section 10.5.2.4)
- Registered water licence applications listed on the Ministry of Environment's Water Stewardship Division website (Volume 2 Section 10.5.2.5)
- Projects or activities associated with existing or "accepted" applications for land tenure under the B.C. *Land Act* or the B.C. *Range Act* (e.g., range tenures, grazing licenses, wind, gravel) as provided by GeoBC (2012) (Volume 2 Section 10.5.2.6)
- Current harvest plans associated with tenured forest operations and timber sales (Volume 2 Section 10.5.2.7)
- Official Community Plans, and parks and recreation plans (Volume 2 Section 10.5.2.8)
- Large waste discharges into the Peace River from Peace Canyon Dam to Vermilion Chutes, Alberta (Volume 2 Section 10.5.2.9)

A list of all other future foreseeable projects and activities considered in the assessment of the potential residual effects of the Project is provided in Volume 2 Section 10.5.3 Cumulative Effects Assessment, Table 10.7. The locations of these other projects and activities are shown in Figure 10.3, Figure 10.4, Figure 10.5, and Figure 10.6.

37.3.2 Description of Potential Cumulative Effects

For those VCs for which residual adverse effects are predicted, the potential cumulative effects of the Project have been assessed and are discussed in the respective VC-specific chapters of this EIS.

37.3.3 Identification of Mitigation Measures

With respect to certain VCs, BC Hydro has recommended possible regional approaches to mitigation. These mitigation measures may involve government departments and/or third parties in independent or collaborative initiatives. The potential possible regional approaches to mitigation are discussed in the respective VC-specific chapters of this EIS.

37.3.4 Characterization of Cumulative Residual Effects

Residual cumulative effects have been characterized using the criteria described in Volume 2 Section 10.4.2.2 and in Table 10.3. The potential residual adverse cumulative effects are discussed in the respective VC-specific chapters of this EIS.

37.3.5 Determination of Significance of Cumulative Residual Effects

BC Hydro provides a determination of the significance of the potential residual adverse cumulative effects that may result from the Project, and its rationale for the determination, in the respective VC-specific chapters of this EIS.

37.3.6 Summary of the Potential Cumulative Effects of the Project

A summary of the assessment of potential cumulative adverse effects that are likely to result from the Project in combination with other projects or activities that have been or will be carried out for relevant Project VCs is presented in Table 37.24.

Table 37.24 Summary of the Potential Cumulative Effects of the Project

Valued Component	Residual Effect of the Project	Cumulative Effect of the Project	Significant Cumulative Effect (Y/N)
Section 13 Vegetation and Ecological Communities	Alteration and fragmentation of habitat for the terrestrial ecosystems and rare plants	The anticipated residual effects to Vegetation and Ecological Communities from other future projects and activities combined are considered significant, even without the Project. This is because the potential residual effects of other projects and activities that include road construction, forestry and land clearing activities, cannot be fully mitigated and the future loss of rare plants and rare and sensitive ecosystems due to these other projects have the potential to further elevate provincial or federal listings. The cumulative effect with the Project is also considered significant.	Y
Section 14 Wildlife Resources	Habitat alteration and fragmentation, disturbance or displacement, and mortality for key species groups: butterflies and dragonflies, amphibians and reptiles, migratory birds, non-migratory game birds, raptors, bats, fur-bearers; ungulates, and large carnivores.	The anticipated residual effects to Wildlife Resources from other future projects and activities combined are considered significant, even without the Project. The footprints of other projects and activities within the regional assessment area would result in the loss and fragmentation of habitat for wildlife. The Project would potentially result in the alteration and fragmentation of habitat, disturbance or displacement, and mortality for certain key indicator species or species groups. The cumulative effect with the Project is also considered significant.	Y

Valued Component	Residual Effect of the Project	Cumulative Effect of the Project	Significant Cumulative Effect (Y/N)
Section 15 Greenhouse Gases	The residual quantities of GHGs released to the atmosphere are low, and are a small fraction of provincial, national, and global emissions.	Increasing GHG emissions from the many sources globally and the resulting increase in GHG concentrations in the atmosphere, and the consequent changes to the global climate, are currently believed to be a significant cumulative environmental effect, even without the Project. While the Project's contribution to a net change in global GHG emissions is relatively small, and the environmental effect of the Project related GHG emissions on global climate is not measurable, the cumulative effect with the Project is also considered significant.	Y
Section 24 Harvest of Fish and Wildlife Resources	Decrease in hunting opportunities.	Decrease in access to hunting areas (0.8% of Limited Entry Hunting area 7-20a is in the LAA), and displacement of hunting activities and wildlife away from the Project activity zone outside the LAA.	N
Section 27 Visual Resources	Increase in the amount of visible anthropogenic disturbances.	Increase in the amount of visible anthropogenic disturbances in the short term.	N
~~*Section 27 Visual Resources*~~	~~Increase in the amount of visible anthropogenic disturbances.~~	~~Increase in the amount of visible anthropogenic disturbances in the long term.~~	~~N~~

37.4 Capacity of Renewable Resources

This section summarizes the results of the assessment of the potential effects of the Project on the capacity of renewable resources to meet the needs of present and future generations. Renewable resources are natural resources, living (e.g., fish, wildlife, and forest) and non-living (agricultural soils, scenic landscapes), that are replenished on an ongoing basis, either naturally or by human action. Renewable resources are used by people either consumptively (e.g., fishing, hunting, or forestry) or non-consumptively (recreational and visual resources).

This section also summarizes the Project's beneficial use of a renewable resource for present and future generations.

The potential effects of the Project were assessed, taking into account predicted changes to the physical environment as described in Volume 2 Section 11 Environmental Background.

37.4.1 Effects of the Project on VCs that Contain, or Pertain to, Renewable Resources

Table 37.25 lists the VCs that are considered in this EIS and that contain, or pertain to, renewable resources. In the table, for each renewable resource, the use of the resource is described, whether the uses are consumptive or non-consumptive is indicated, and

the results of the assessment of the potential effects on the Project on the VC are summarized.

Table 37.25 Summary of Consideration of Renewable Resources Within the EIS

Valued Component	EIS Section	Resource Use, Consumptive (C), Non-Consumptive (NC)	Potential Significant Adverse Effects
Fish and Fish Habitat	Volume 2 Section 12	Fishing (C, NC)	Construction: • Loss of habitat due to construction headpond and reservoir filling • Reduced fish health and survival due to sediment inputs from construction headpond and reservoir filling • Hindered fish movement due to obstruction to fish passage Operations: • Hindered fish movement due to obstruction to fish passage
Vegetation and Ecological Communities	Volume 2 Section 13	Landscape viewing, plant gathering (C)	• Habitat alteration and fragmentation to certain ecological communities and rare plants
Wildlife Resources	Volume 2 Section 14	Wildlife viewing, hunting, trapping (C)	Construction and Operations • Alteration and fragmentation of habitat for Yellow Rail (SARA-special concern, Red-listed), Canada Warbler (SARA-threatened, Blue-listed), Cape May Warbler (Red-listed), Bay-breasted Warbler (Red-listed), and Nelson's Sparrow (Red-listed).
Current Use of Lands and Resources for Traditional Purposes	Volume 3 Section 19	Fishing, hunting, trapping, plant gathering for Aboriginal purposes (C)	Changes to other cultural and traditional uses of the land during construction and operations
Agriculture	Volume 3 Section 20	Cultivation for agricultural purposes (C, NC)	No significant adverse residual effects
Forestry	Volume 3 Section 21	Timber harvesting for industrial purposes (C)	No significant adverse residual effects
Harvest of Fish and Wildlife Resources	Volume 3 Section 24	Fishing, hunting for recreation, hunting for commercial guiding purposes, trapping for commercial purposes (C)	No significant adverse residual effects
Outdoor Recreation and Tourism	Volume 3 Section 25	Landscape and wildlife viewing, fishing, hunting, outdoor recreation/tourism activities in the outdoor environment (C, NC)	No significant adverse residual effects

Valued Component	EIS Section	Resource Use, Consumptive (C), Non-Consumptive (NC)	Potential Significant Adverse Effects
Navigation	Volume 3 Section 26	Boating on navigable waterways (NC)	No significant adverse residual effects
Visual Resources	Volume 3 Section 27	Landscape viewing	No significant adverse residual effects
Human Health	Volume 4 Section 33	Air quality, recreational, and drinking water quality	No significant adverse residual effects

37.4.2 Project Beneficial Use of Renewable Resources

The Peace River provides a reliable, long-term, and renewable source of water. Since 1968, surface water resources on the Peace River have been used non-consumptively to produce renewable electricity for past and present generations, and will continue to produce electricity for future generations. The Project would allow BC Hydro to reuse the same water used at two upstream hydroelectric stations to produce hydroelectric energy, in a non-consumptive manner, to provide substantial renewable electricity benefits to both present and future generations.

BC Hydro commissioned a climate study to understand potential future changes to the climate in key watersheds, including the Peace River, and related potential changes to the annual cycle and volume of inflows. This study confirmed that, with predicted global climate change, reliable inflows will continue on the Peace River (Volume 2 Appendix T Climate Change Summary Report).

37.5 Requirements of Any Follow-Up Programs

A follow-up program is designed to verify the accuracy of the effects assessment and to determine the effectiveness of the measures implemented to mitigate the adverse effects of the Project, as described in Section 23.5 of the EIS Guidelines. The rationale for recommending a follow-up program for a particular VC is provided in Volume 2 Section 10 Effects Assessment Methodology. Follow-up programs for relevant VCs are provided in the EIS in Volume 2 Sections 12–15, Volume 3 Sections 16–27, Volume 4 Sections 28–33, and in Volume 5 Section 39 Complete List of Mitigation and Follow-Up Measures.

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