APPENDIX 6.2.6-A

Saskatchewan River Modeling Study

SASKATCHEWAN RIVER HYDROTECHNICAL AND DISPERSION MODELING STUDY STAR DIAMOND PROJECT – 2010

Submitted to:

Shore Gold Inc.
Saskatoon, Saskatchewan

Submitted by:

AMEC Earth & Environmental
Edmonton, Alberta

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SX0373305

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EXECUTIVE SUMMARY

This report presents the results of a hydrotechnical and dispersion modeling study undertaken on the Saskatchewan River, east of Prince Albert, Saskatchewan, near the proposed **Shore Gold Inc.** *Star Diamond Project*. Operation of the mine will require some facilities to discharge brackish groundwater into the Saskatchewan River near the Duke or FALC Ravines. A hydrotechnical and dispersion modeling study was required to assess the local river hydraulics and quantify chloride concentrations in the receiving waterway in support of the Environmental Impact Study (EIS) and feasibility study for the *Star Diamond Project*. This report is presented in two parts as described below.

PART 1 – HYDROTECHNICAL MODELING STUDY

Part 1 of this report discusses the hydrotechnical modeling study, which includes a detailed two-dimensional bathymetric survey of an 8 km reach of river near the proposed effluent discharge location and hydrodynamic modeling using River2D and HEC-RAS applications. Bathymetric survey data and findings from the hydrotechnical study were used as a foundation for two-dimensional dispersion modeling of chloride concentrations in the Saskatchewan River that corresponds to the proposed effluent discharge.

Some key findings of the hydrotechnical modeling study are that:

- the Saskatchewan River near the proposed effluent discharge location has a steeper upper reach, with characteristically higher velocities and shallower depths, and a more mild-sloping lower reach that is wider, deeper, and has lower velocities;
- flow through the reach is relatively uniform across the channel under most conditions, and there are no islands or mid-channel features to divide the flow and promote transverse mixing;
- some degree of backwater effect from the downstream hydropower reservoir was evident at the time of survey, which increased depths and water levels throughout the study reach;
- based on available sediment data and methodologies for assessing probable bed form type and size, the predominant bed material is expected to be sand with a median grain size of 0.28 mm and bed forms are anticipated to be dunes of 0.5 to 1 m in height in the vicinity of the outlet structure; however, direct observations and sampling of bed material during a low-flow period are recommended for the detailed design phase of an outfall or diffuser structure at this site; and,
- there appear to be no available observations documenting the river ice regime in the study area, but the channel geometry, hydraulic characteristics, and conceptual modeling of ice jams suggest that ice action should be considered during the detailed design phase; to better understand how ice activity might impact the detailed design of an outfall or diffuser, observations should be made along the study reach in the winter and/or immediately after break-up.

PART 2 – DISPERSION MODELING STUDY

Part 2 of this report discusses the results of a dispersion modeling study using AQUASEA, undertaken to assist **Shore Gold Inc.** with subsequent design of a river outfall or diffuser located on the Saskatchewan River and permitting for the *Star Diamond Project*. Bathymetric survey data and findings from the hydrotechnical study were used as a foundation for a two-dimensional dispersion modeling study which addresses the prediction of chloride concentrations in the Saskatchewan River that are expected to result from the discharge of groundwater to the river.

A 70 m long outfall and a 40 m long outfall were considered separately in the dispersion modeling study. Some key findings of the study are that:

- during low flows, the predicted maximum chloride concentration at a point 500 m downstream of the 70 m outfall, was 90 mg/L near the river bank for an effluent discharge rate of 199 000 m^3/d and an effluent chloride concentration of 1725 mg/L. For this scenario, the plume centreline is located approximately 25 m from the river bank at this location where the maximum chloride concentration is 99 mg/L and the plume width is 95 m;

- for the 40 m outfall, the plume generated was closer to the river bank with two to three times higher river concentrations within 500 m downstream of the outfall. Concentrations predicted approach those predicted for the 70 m outfall further downstream;

- for the low-flow condition, the river bank chloride concentration at the end of the study reach (6000 m downstream) was reduced to 33 mg/L (approximately 2% of initial chloride concentration); and,

- under average-flow conditions the plume was narrower with lower chloride concentrations.

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PART 1

HYDROTECHNICAL MODELING STUDY

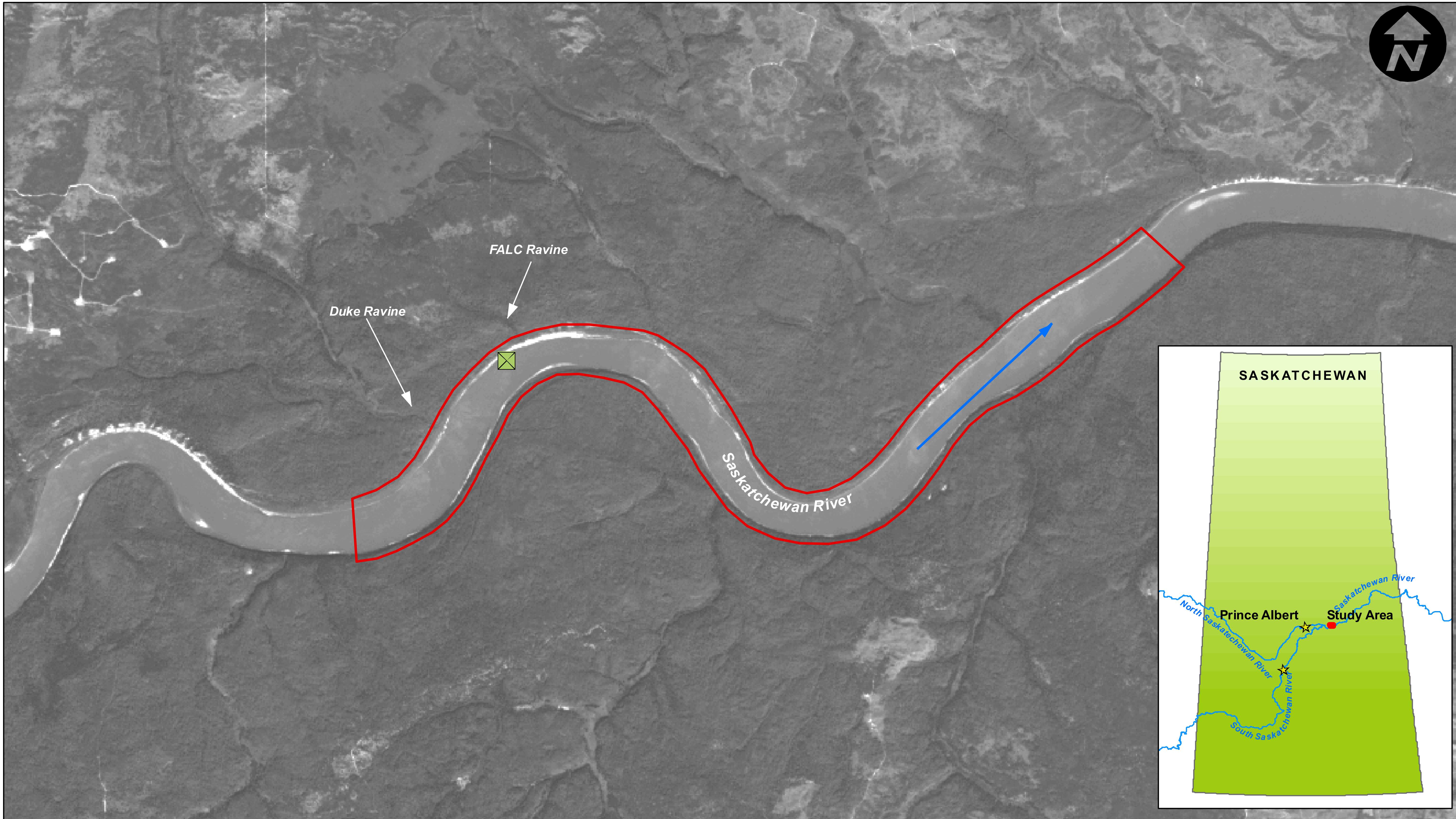
1.0 INTRODUCTION

AMEC Earth & Environmental (AMEC) was retained by **Shore Gold Inc.** to conduct a preliminary modeling study for a proposed river outfall or diffuser structure on the Saskatchewan River for the *Star Diamond Project*. Effluent that must be discharged from a proposed diamond mining operation is expected to be brackish. The goal of this study was to evaluate effluent mixing in the Saskatchewan River from a proposed outfall or diffuser and to build a reliable and detailed hydraulic model that can be used to evaluate the characteristics of the river under a wide range of flow scenarios that are important to a more detailed design of the structure. The proposed outfall or diffuser structure site is located along the Saskatchewan River, approximately 600 m downstream of the mouth of Duke Ravine and approximately 40 km downstream of the confluence of the North Saskatchewan and South Saskatchewan rivers. **Figure 1.1** provides an overview of the study area, including the boundary of the study site and the approximate location of the diffuser site.

Baseline bathymetry data in the study area was collected by CanNorth in August 2008. This data consisted of depth-based mapping extending approximately 0.5 km upstream and 1.5 km downstream of the proposed outfall location. To demonstrate to regulators that an adequate reach of the river is being considered, a detailed bathymetric survey and discharge measurements extending 1.0 km upstream and 6.5 km downstream of the proposed outfall location were carried out in late June 2010 by AMEC, with field support provided by CanNorth.

Using the information obtained from the field program, a reliable two-dimensional hydrodynamic model was constructed, calibrated, and used to provide necessary information that aided effluent dispersion modeling. River hydrology analyses were carried out in order to supply the model with relevant flow scenarios; including low flows where mixing is least efficient and high flows in which the diffuser must be adequately designed to withstand. Bed sediment material was analysed for the purpose of predicting the type and size of bed forms in the channel. One-dimensional ice jam modeling was carried out in order to explore the potential effects that an ice jam may have at the proposed outfall or diffuser site.

This hydrotechnical modeling study focuses on details of the field program, river hydrology, river hydraulics, sediment characteristics and river ice jam considerations with respect to the design of the outfall structure. In addition, recommendations for various aspects of a detailed final design for the proposed outfall structure are made. Instream mixing and details of effluent dispersion are covered in Part 2 of this report.



Study Area Boundary
Outfall/Diffuser (Approximate)

0 500 1,000 2,000 3,000 metres

Background Data from GeoBase®

CLIENT: Shore Gold Inc.

AMEC Project No.: SX0373305

DWN BY: RBA CHK'D BY: RBA

SHORE GOLD RIVER SURVEY

Study Area Map

November 2010

FIGURE 1.1

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2.0 DESCRIPTION OF STUDY REACH

The 7.5 km long reach of the Saskatchewan River chosen for the study, extending 1.0 km upstream and 6.5 km downstream of the proposed outfall location, is shown in **Figure 1.1**. This site is located approximately 40 km downstream of the confluence of the North Saskatchewan and the South Saskatchewan Rivers. The Saskatchewan River at Nipawin, which is located approximately 60 km downstream of the study area, has an effective drainage area of 287 000 km^2, as reported by the Water Survey of Canada. The streamflow in the North Saskatchewan River is regulated by the Bighorn and Brazeau Dams located in Alberta and the flow in the South Saskatchewan River is regulated by the Gardiner Dam located in Saskatchewan. Downstream of the site, there are two reservoirs for hydroelectric power generation: the Francois Finlay Dam at Nipawin which is located 60 km downstream and the E.B. Campbell Dam at Tobin Lake, which is located 130 km downstream of the site. There are a number of small creeks within the reach, entering the river on both sides of the channel.

In the study reach, the Saskatchewan River flows east through the south region of Fort à la Corne forest. This reach includes two sections: a steep upper reach resulting in shallower depths and a more mildly-sloping lower reach with deeper wide sections. There are two large meanders in the upper part of the reach before entering a relatively straight downstream portion. The river is entrenched in its valley, with little floodplain on either side of the channel and very steep banks in certain areas. Throughout the study site, the flow is relatively uniform across the channel and there are no islands that divide the flow. The reach is relatively remote with the closest access point located approximately 20 km downstream near a bridge along Highway 6 at Wapiti Valley.

Figure 2.1 shows a photograph of the area of the proposed outfall structure. This area is located 600 m downstream of the mouth of Duke Ravine and 300 m upstream of the mouth of FALC Ravine on the left bank of the Saskatchewan River. The coordinates of this area are: 53°13'26" N, 104°42'27" W and it can be found on NTS map sheet number 073H02.

Figure 2.1 Photograph of Outfall/Diffuser Site

3.0 FIELD PROGRAM

3.1 Introduction

In order to construct computer models capable of simulating river hydraulics and mixing, a comprehensive field program providing adequate river geometry and calibration data was required. Such a program was undertaken for the 8 km long study reach over the period of 24 through 28 June 2010. During this time, the field crew completed a detailed bathymetric survey, water level surveys, and discharge measurements. Throughout the survey, horizontal and vertical positioning was achieved using survey-grade Trimble® R6/R8 GNSS RTK-GPS receivers.

All survey data was recorded in the UTM NAD83 coordinate system (Zone 13) with elevations as geoid heights referenced to the HT2.0 datum. Coordinates stated in this report are converted to NAD27, which is being used in other work on the Star Diamond Project. Local survey control points were established within the reach (see **Figure 3.1**) and tied into provincial benchmarks. These control points consisted of iron bars which were driven into the ground to a depth sufficient to resist frost movements so as to provide a semi-permanent control that can be referenced during future programs. Positional accuracy of survey points for the bank and bathymetry surveys is better than ±1 cm in the horizontal and ±2 cm in the vertical directions.

3.2 Bathymetric Surveys

The bathymetric survey comprised two parts: a bank survey for the above-water portion of the channel and a channel bed survey for the portion below the water. A feature-based survey was conducted, complete with appropriate annotation, that captured the details of the topography of the channel such as top of bank, bottom of bank, edge of water and channel bed. The bank survey was carried out using the RTK-GPS system, consisting of a base station communicating by radio with several mobile ‘rover’ units. The spacing between bank feature points was approximately 50 m. In total, 1800 ground points were surveyed.

The channel bed survey was conducted from a boat with an OHMEX SonarLite 2000 depth sounder and RTK-GPS attached. The transducer was attached to a bracket on the boat, and placed at a depth approximately 0.2 m below the water level. The RTK-GPS antenna was then positioned a known distance directly above this transducer, allowing for the bed elevation to be determined at each point. In total, 22 000 discrete bed points were measured and recorded over the reach. **Figure 3.2** displays all of the surveyed bathymetry points.

3.3 Water Level Surveys

In addition to the control points set up for the bathymetry survey, temporary benchmarks with less permanence were established along the reach to facilitate open water surveying. These points were spaced approximately 700 m apart and included points at the upstream and downstream limits of the survey (see **Figure 3.1**, Temporary Benchmarks 20 through 30). A rod and level were used to survey water levels relative to these control points in order to obtain a detailed water surface profile. **Table 3.1** presents a summary of the water levels obtained on 28 June 2010.

Table 3.1
Water Surface Profile of the Study Reach Obtained on 28 June 2010

Benchmark Name	River Station (m)	Elevation (m)
1	255	353.165
30	622	352.776
29	1583	352.023
28	2201	351.731
27	2941	351.43
26	3670	351.146
25	4468	350.835
24	5543	350.630
23	6262	350.599
22	6943	350.488
21	7626	350.397
20	8386	350.243

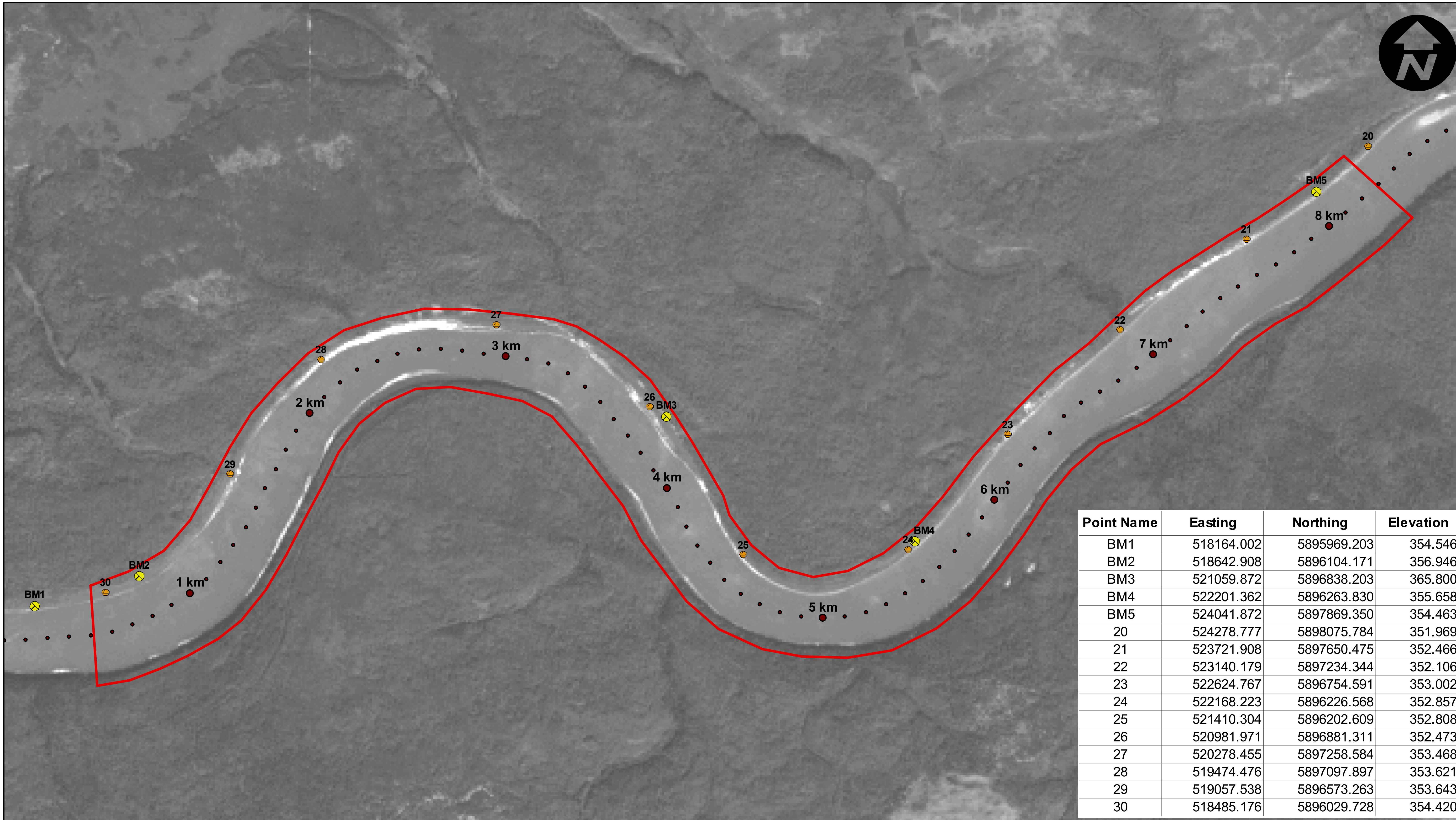
3.4 Discharge Measurements

A SonTek 3 MHz Acoustic Doppler Current Profiler (ADCP) (see **Figure 3.3**) was used to measure detailed depth-integrated velocity profile data and the total discharge across several sections in the reach. These sections include the upstream and downstream limits of the survey in order to provide estimates on inflow and outflow discharge. Additional velocity measurements were obtained in the vicinity of the proposed outfall structure to facilitate model validation. These three sites can be seen in **Figure 3.2**. The ADCP sensor was mounted in a trimaran and then deployed from the front of a boat. An RTK-GPS rover was attached to the system in order to aid bottom tracking and deal with moving bed conditions that were anticipated during the time of the survey. The distance from the endpoints to the edge of water was measured in each case and input into the ADCP software so that the entire section was accounted for when measuring the discharge. A minimum of 4 passes were made across each section. Instrument and quality control monitoring was done via computer from within the boat and the bottom tracking output was used to help guide the boat operator in a straight path between the endpoints.

At the first site (Section A-A), located near the upstream boundary of the survey, 5 passes were conducted on 24 June 2010. The resulting discharges ranged from 996 to 1182 m^3/s, with the average value of these measurements being 1118 m^3/s. There were some difficulties experienced in obtaining measurements at the site as bottom tracking and signal return was intermittent in some parts of the section due to sediment loads. Using the real-time data available from Water Survey of Canada for the North Saskatchewan River at Prince Albert and the South Saskatchewan River at Saskatoon, it is estimated that the discharge at the site on 24 June 2010 was approximately 1017 m^3/s. Taking into account the additional discharge from smaller tributaries downstream of the monitoring stations, the true discharge at the site is

expected to be higher. The ADCP average measurement value of 1118 m^3/s is indeed higher, but still within 10% of the estimated value of 1017 m^3/s.
Four passes were conducted on 24 June 2010 at the second site (Section B-B) which is located in the vicinity of the proposed outfall structure. The resulting discharges ranged from 1071 to 1192 m^3/s, with the average value of these measurements being 1122 m^3/s. Again, there were some difficulties in taking measurements due to sediment movement along the bottom of the channel. However, there is confidence in the measurements taken as the values at this site and the previous site were very similar and are within 10% of the estimated discharge value using data from the North Saskatchewan River at Prince Albert and the South Saskatchewan River at Saskatoon.

On 28 June 2010, 8 passes were conducted near the downstream boundary of the survey (Section C-C). The resulting discharges ranged from 1097 to 1236 m^3/s, with the average value of these measurements being 1168 m^3/s. The mean velocities at this site were lower than at the two sites previously measured, which resulted in less sediment movement along the channel bottom and improved signal return. Using the available real-time data, it is estimated that the discharge at the site on 28 June 2010 was approximately 1079 m^3/s. Again, the ADCP average measurement value is slightly higher, as expected, but also within 10% of the estimated data. **Appendix A** provides detailed records for each of the velocity and discharge measurements obtained across the section.



Point Name	Easting	Northing	Elevation
BM1	518164.002	5895969.203	354.546
BM2	518642.908	5896104.171	356.946
BM3	521059.872	5896838.203	365.800
BM4	522201.362	5896263.830	355.658
BM5	524041.872	5897869.350	354.463
20	524278.777	5898075.784	351.969
21	523721.908	5897650.475	352.466
22	523140.179	5897234.344	352.106
23	522624.767	5896754.591	353.002
24	522168.223	5896226.568	352.857
25	521410.304	5896202.609	352.808
26	520981.971	5896881.311	352.473
27	520278.455	5897258.584	353.468
28	519474.476	5897097.897	353.621
29	519057.538	5896573.263	353.643
30	518485.176	5896029.728	354.420

Study Area Boundary
Centreline River Station
Control Point
Temporary Bench Mark

0 500 1,000 2,000 metres

Background Data from GeoBase®
Horizontal Datum: NAD27 (13N)

CLIENT: Shore Gold Inc.

AMEC Project No.: SX0373305

DWN BY: RBA CHK'D BY: RBA

SHORE GOLD RIVER SURVEY

Benchmark and Control Point Locations

November 2010

FIGURE 3.1

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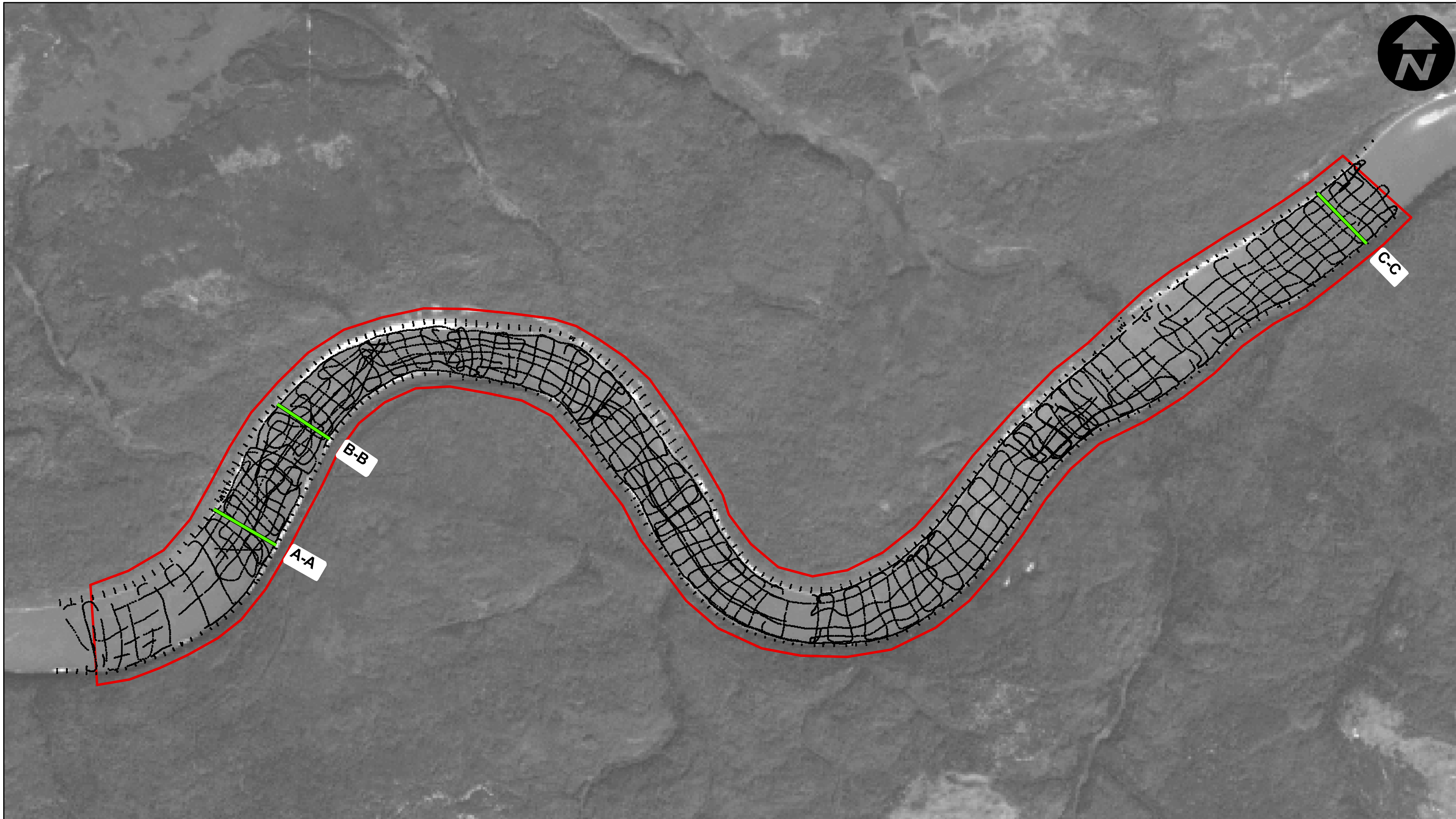
Map Path: L:\Water\PROJECT\EW...



Study Area Boundary

Discharge Measurement Transect

Bathymetry Point

Background Data from GeoBase®

0 500 1,000 2,000 metres

CLIENT: Shore Gold Inc.

AMEC Project No.: SX0373305

DWN BY: RBA CHK'D BY: RBA

SHORE GOLD RIVER SURVEY

Bank and Depth Sounding Survey Point Locations

November 2010

FIGURE 3.2

amec

Map Path: L:\Water\PROJECT\EW\...

Figure 3.3 Photograph of Acoustic Doppler Current Profiler

4.0 HYDROLOGY

4.1 Introduction

Hydrological analyses of recorded river discharges upstream and downstream of the site were undertaken to estimate river discharge parameters in support of dispersion modeling at the proposed effluent discharge location in the Saskatchewan River. The proposed outfall or diffuser structure is located approximately 40 km downstream of the confluence of the North Saskatchewan and the South Saskatchewan Rivers (see **Figure 1.1**). The flow in these rivers is regulated by hydroelectric dams upstream of the site. There are also two reservoirs for hydroelectric power generation located approximately 60 km and 130 km downstream of the site. Data from the closest Water Survey of Canada (WSC) hydrometric stations were used to assemble a suitable streamflow record for the proposed site. Analyses were conducted on the hydrological data to provide representative flows at the study site for river hydraulics and diffuser design modeling.

Using a synthetic streamflow record, average-flow conditions were determined and presented in the form of mean annual discharge and mean monthly discharges. Monthly flow duration curves were also created to determine the probability that various discharges would be equalled or exceeded. When considering mixing in a channel, of particular importance are the periods of low flow, as such cases represent times when water available for dispersion and mixing will be the lowest. To address this critical period, the 1-in-10-year average 7-day low flow, or 7Q10, was determined for both the annual (January–December) and the open water (May–October) cases. In addition, a flood frequency analysis was conducted in order to establish the peak discharges that may be experienced.

An ice data analysis was also undertaken to address critical design parameters affecting the proposed outfall structure that involve winter ice-covered conditions. The analysis was conducted using archived historic ice thickness information data from the WSC. This information was applied in a two-dimensional depth-averaged hydrodynamic model with a continuous intact ice cover in order to evaluate flow and velocity patterns and to a one-dimensional river ice jam model to provide estimates of velocities under potential ice jam conditions. These analyses are described in Sections 5.0 and 7.0, respectively.

4.2 Methodology

The WSC operates and maintains a hydrometric network that measures and records river water levels and flows. Surface water flow data are available at the locations shown in **Table 4.1**. Of these gauges, only three are currently active: North Saskatchewan River at Prince Albert (05GG001); South Saskatchewan River at Saskatoon (05HG001); and, Saskatchewan River below Lake Tobin (05KD003). The latter is the closest active WSC gauge to the site along the Saskatchewan River (located approximately 130 km downstream). However, data from this gauge are not representative of discharges at the site since the gauge is affected by the E.B. Campbell Hydroelectric Station. There are no active WSC gauges located along this reach that can be directly used to represent hydrological conditions at the site.

Table 4.1
List of Water Survey of Canada Hydrometric Gauges Near the Study Site

WSC Station	Name	Period of Record	Drainage Area (km^2)	Comments
05GG001	North Saskatchewan River at Prince Albert	1910 to 2009	131 000	Upstream of confluence; regulated since 1962
05HG001	South Saskatchewan River at Saskatoon	1911 to 2009	141 000	Upstream of confluence; regulated since 1968
05HH001	South Saskatchewan River at St. Louis	1958 to 1997	148 000	Upstream of confluence; regulated since 1968
05KD001	Saskatchewan River at Nipawin	1945 to 1948; 1951 to 1962	287 000	Downstream of confluence; regulated since 1968
05KD003	Saskatchewan River below Lake Tobin	1962 to 2009	289 000	Downstream of confluence; regulated since 1963

To construct a streamflow record for the site, mean daily discharges for the two closest upstream gauges along both the North Saskatchewan River (05GG001 at Prince Albert) and South Saskatchewan River (05HG001 at Saskatoon) were added. These gauges are located approximately 100 km and 260 km upstream of the site, respectively. No gauged or major ungauged tributaries flow into the river between the upstream gauge sites and the study site. Due to the relatively small increase in contributing area between these gauges and the site, no adjustment was made to account for the small incremental increase in discharge downstream of the gauges; this provides slightly conservatively (approximately 5%) low estimates for discharge at the site. The lag time between the upstream gauges and the site was estimated for 2 cases, representing "long" and "short" travel times. Assuming a high average channel velocity, it was determined that the travel time from Prince Albert to the proposed site was 1 day and the travel time from Saskatoon to the proposed site was 2 days. Using a lower average channel velocity to better represent the low-flow period, a travel time of 2 days was used from Prince Albert to the proposed site and a travel time of 6 days was used from Saskatoon to the proposed site. These scenarios are presented in **Table 4.2**. Analysing the data using both sets of travel times allowed both the critical high- and low-flow scenarios to be simulated. These lag times were incorporated in the determination of mean daily discharge at the proposed site. As discussed below, both sets of travel times yielded similar computed river discharge parameter estimates at the proposed outfall site, which indicated that the results were not sensitive to the lag time.

Table 4.2
Estimated Travel Time from WSC Gauged Sites to Study Site

Scenario	Travel Time (days)	
	Prince Albert to Study Site	Saskatoon to Study Site
Short Travel Time	1	2
Long Travel Time	2	6

The effects of regulation were taken into account when building the dataset for the proposed site. There are several large dams that influence the flow patterns of the North Saskatchewan and South Saskatchewan Rivers. The Brazeau Dam, affecting the North Saskatchewan River, was built in 1962 and has an effective drainage area of 5660 km^2. The Bighorn Dam, also affecting the North Saskatchewan River, was created in the early 1970s and has an effective drainage area of 3890 km^2. The flows from these two dams represent a small percentage of the flow of the North Saskatchewan River at Prince Albert, which has a drainage area of 131 000 km^2, and thus are not likely to dramatically influence the flow patterns at the proposed site. The Gardiner Dam, affecting the South Saskatchewan River, was operational in 1968. It has an effective drainage area of 136 000 km^2, which is a large proportion of the effective drainage area of the South Saskatchewan River near Saskatoon of 141 000 km^2. Since regulation from this dam would have a great impact on the flow patterns at the proposed site, it was most appropriate to start the dataset in 1969, after the creation of the dam.

The effects of missing data were also considered in the creation of a dataset for the proposed site. From 1987 through 1991, winter flows were not monitored at the gauge located at South Saskatchewan River at Saskatoon (05HG001). Since excluding such a large period of flow would falsely alter any statistics conducted using daily discharge values, the entire flow record for these years were deleted from the mean daily discharge dataset. When analysing mean monthly data, the values for each month are considered independently of the rest of the year. Thus, for such analysis, it was suitable to incorporate the remaining available monthly data for the years 1987 through 1991.

4.3 Design Discharge Determination

4.3.1 Mean Annual Discharge

The mean annual discharge at the proposed site was calculated using mean daily data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001) for the combined periods of 1969–1986 and 1992–2009. This average was calculated for two annual periods. The first was based on a water year from November 1 to October 31. Using this method allowed for the water data to be split during a relatively steady flow period immediately prior to freeze-up during which discharge does not change significantly. The mean annual discharge was also calculated using a water year from July 1 to June 30. Using this method, the flows were split in the middle of the wet season in order to capture one low-flow period per water year. The value for the mean annual discharge using both methods of water year selection, as well as both sets of travel times, was determined to be 439 m^3/s.

4.3.2 Mean Monthly Discharge

The mean monthly discharge at the proposed site was calculated using mean monthly data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001) throughout the period of 1969–2009. Within that data set, some of the winter months for the years 1987 through 1991 were excluded due to lack of recorded data at the gauge. The results of this analysis are shown in **Figure 4.1**, which indicates that higher mean flows are experienced from April through August. Due to regulation by hydroelectric dams located upstream of the site, it can be seen that mean flows do not vary dramatically from the summer to the winter months. Monthly flow duration curves created using the mean monthly flow data are illustrated in **Figure 4.2**. These graphs show the probability that a given discharge will be equalled or exceeded in each month. **Figure 4.2** indicates that high flows are most likely to occur during the months April through August and that low flows commonly occur during November to March. Individual monthly flow duration curves are contained in **Appendix B**.

4.3.3 Low Flows

The design low-flow values are represented by a calculated consecutive 7-day low average discharge with a 10-year average recurrence interval, or 7Q10. The 7Q10 value was determined for both the open water and the annual case using mean daily data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001) for the combined periods of 1969–1986 and 1992–2009. For the open water case, this value was calculated using the open water period from May 1 to October 31. The open water 7Q10, calculated using both sets of travel times, was determined to be 188 m^3/s. For the annual case, this value was first calculated using a water year from November 1 to October 31 and also using a water year from July 1 to June 30, as in the case of the mean annual flow. The values of the annual 7Q10 using each method and incorporating both sets of travel times were determined to be 168 m^3/s and 170 m^3/s, respectively. Thus, 169 m^3/s was taken as the design value for the annual 7Q10. These results are summarized in **Table 4.3**.

Table 4.3
Calculated 7Q10 Values for the Open Water and Annual Case

7Q10 Values	Discharge (m^3/s)
Open Water Design Value	**188**
Nov 1 to Oct 31 annual value	168
July 1 to June 30 annual value	170
Annual Design Value	**169**

4.3.4 Flood Frequency Estimates

Flood frequency analyses were conducted for the proposed site using mean daily data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001) throughout the periods of 1969–2009. Information from the period between 1987 and 1991 was re-introduced in this case, as missing data in the winter months do not affect the peak flow analysis. Maximum mean daily discharges were used for the analysis since it was found that the ratios of instantaneous peak values to maximum mean daily discharges at Saskatchewan River at Nipawin (05KD001), North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001) were near unity. The Log Pearson Type III distribution was found to best fit the collected data. The results of the analysis are summarized in **Table 4.4**. Note that the discharge measured at the time of the field program in late June 2010 corresponds closely with the 1:2-year flood discharge.

Table 4.4
Flood Frequency Analysis for the Proposed Site Using a Log Pearson Type-III Distribution

Return Period (years)	Discharge (m^3/s)
100	4770
50	3980
20	3080
10	2470
5	1930
3	1550
2	1250

4.4 Ice Data

No ice thickness data were available at the study site. However, in addition to measurements of river water levels and flows, the WSC also records ice thickness measurement data near certain hydrometric stations. Data were available for the North Saskatchewan River at Prince Albert (05GG001) in Saskatchewan and the Saskatchewan River at the Pas (05KJ001) in Manitoba. Data from the North Saskatchewan River at Prince Albert were chosen as representative and analysed since the station is closest to the study site and it lies along similar latitude.

Eleven years of ice thickness data were available from the WSC station at North Saskatchewan River at Prince Albert (05GG001), from 2000 to 2010. This data is shown in **Figure 4.3**. This figure shows ice thickness measurement values from the bottom of the ice to the top of the phreatic water surface. This thickness varies from the actual ice thickness, depending on the density of the ice and the depth of snow on top of the ice. Information on ice density and snow depths are not available; however, these reported thicknesses are considered to be within approximately 10% of the actual ice thickness.

Upon studying the ice thickness data presented, 70 cm was chosen as a representative value for a fully-developed, competent ice cover.

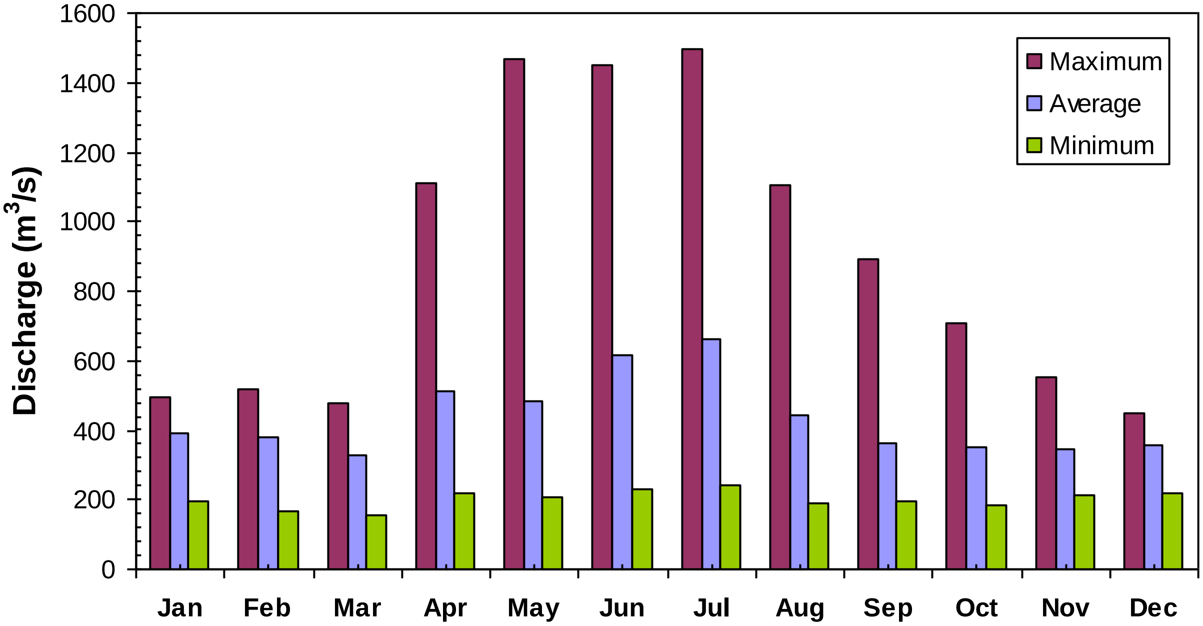


Figure 4.1 Mean Monthly Flows for the Proposed Site Based on Data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009

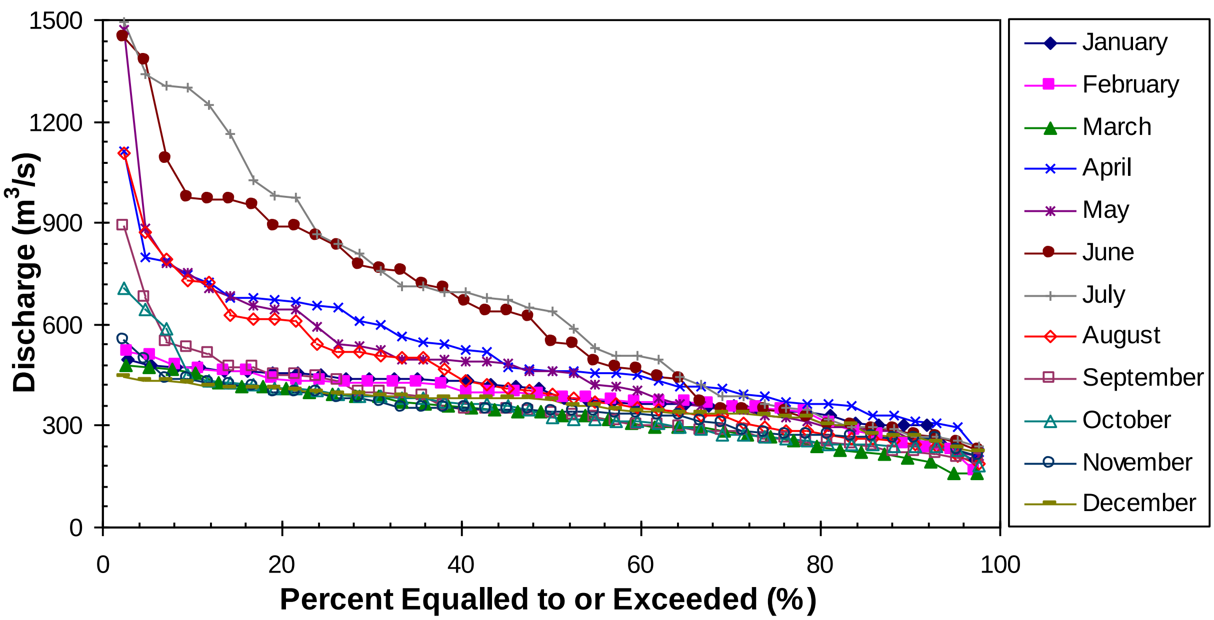


Figure 4.2 Monthly Flow Duration Curves for the Proposed Site Based on Data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–1986; 1992–2009

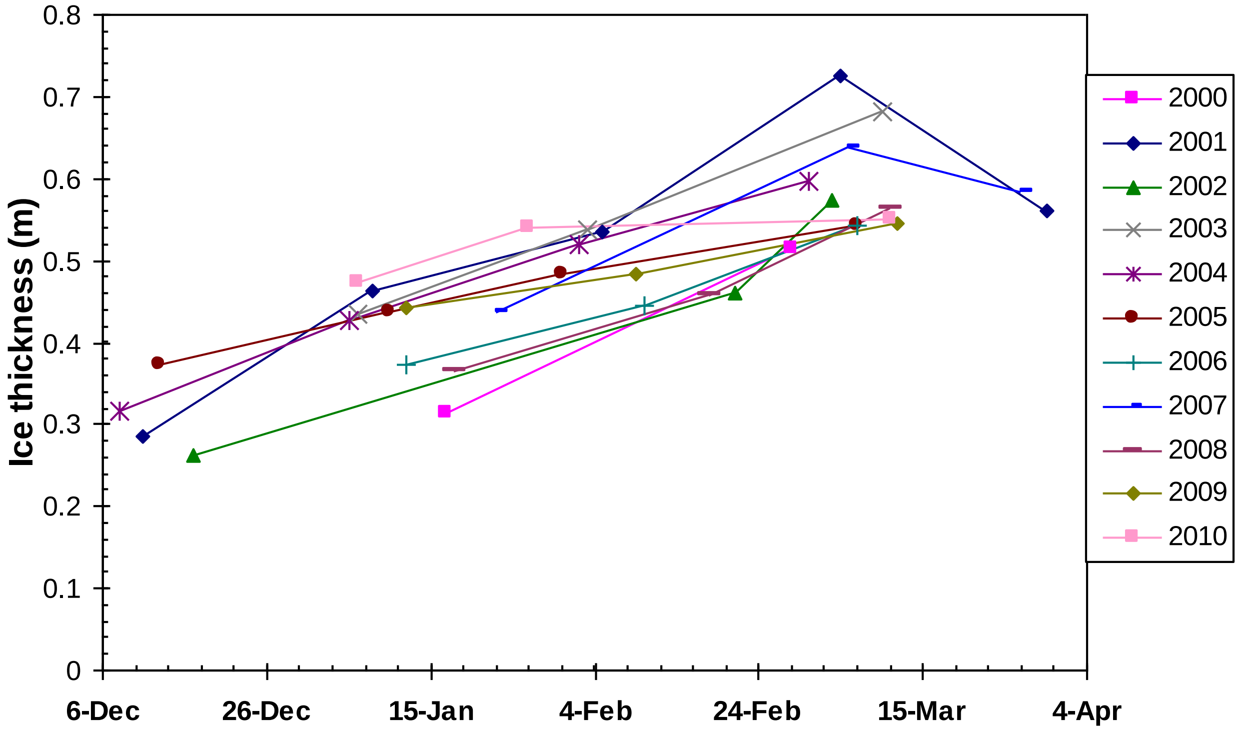


Figure 4.3 Summary of Ice Thickness Measurements on the North Saskatchewan River at Prince Albert (05GG001)

5.0 RIVER HYDRAULIC MODELING

5.1 Introduction

The use of a two-dimensional hydrodynamic model was necessary to assess flow patterns and to provide hydraulic information that is essential for sediment analysis and dispersion modeling. In order to address critical design conditions affecting the proposed outfall structure involving low river discharge and ice-covered conditions in winter, it was necessary that this model be capable of simulating ice-covered hydraulics. The University of Alberta's *River2D* suite of software, which uses a finite element scheme to solve depth-averaged hydrodynamic equations for both open water and continuous intact ice cover conditions, was selected for this study.

Using the data collected during the bathymetric survey, a two-dimensional model was constructed, calibrated to the conditions at the time of the field survey, and used to simulate depth and velocity throughout the study reach during design low- and high-flow conditions. The results of modeling provide an illustration of the water surface elevation, flow depth, velocity magnitude, shear bed velocity magnitude, and cumulative discharge over a range of flow conditions.

5.2 Model Construction

The model geometry bed file and computational mesh were constructed using the detailed bathymetric data that was collected during the field program carried out in June 2010. The bed geometry file was created, covering a reach approximately 7.5 km long in the vicinity of the proposed effluent discharge location. Break lines were inserted in the longitudinal direction to link common river features (e.g., top of bank, bottom of bank, edge of water, channel bed) and allow for correct interpolation of field data points. **Figure 5.1** displays all of the break lines added to the model (as dotted lines). An outer boundary was created around the data points, with delineated upstream and downstream boundaries.

A computational mesh was created using a constant node spacing of 15 m. The Triangulated Irregular Network (TIN) methodology was used to distribute data points, or nodes, and arrange them in a network of non-overlapping triangles (Steffler and Blackburn, 2002). **Figure 5.2** displays a detail of the mesh created in the vicinity of the proposed outfall structure. Special care was taken to distribute a sufficient number of nodes throughout the model, such that the low-flow scenarios would not have issues with the wet-dry transition of elements. The resulting bed geometry, displayed as colour contour elevation information, is shown in **Figure 5.3**. From this figure, it is evident that there is a relatively large change in elevation over the upstream half of the reach, while the downstream half of the reach is flatter. It can also be seen that at the channel bends the river becomes narrower and deeper.

5.3 Model Calibration

In order to be able to apply the model to various flow scenarios, it must first be adequately calibrated. Through the knowledge of the water surface elevation at the downstream boundary and a corresponding discharge at the upstream boundary, channel roughness is calibrated to achieve modeled values corresponding to the observed water surface elevation. For the purpose of this study, a calibration was performed using the data collected on 28 June 2010. This data consists of an inflow discharge of 1168 m^3/s and a water surface profile which includes the downstream boundary condition.

5.3.1 Calibration Results

In *River2D*, channel roughness is represented by a roughness height, k_s. This parameter theoretically represents the median diameter of an equivalent "particle" fixed to the bed that resists movement due to water flowing above. Several constant values of k_s were tested during model calibration, as well as a variation with two different k_s values used together. It was evident that two distinct reaches existed throughout the study site: a steeper upper reach and a wider, more mildly-sloping lower reach. The results of the water surface profile generated by each run are displayed in **Figure 5.4**. It can be seen that the profile which best represents the observed data is one with a value of $k_s = 0.25$ m used for the upstream half of the reach and $k_s = 0.15$ m used for the downstream half. **Figure 5.5** shows the simulated water surface elevation (in metres) along the entire study reach.

5.3.2 Velocity Comparisons

Observed depths and velocities were available for comparison at each of the three discharge measurement cross sections. In order to make an adequate comparison, the depth and velocity data collected by the ADCP was orthogonally projected onto the modeled cross section. The scalar projection of the ADCP velocity was also determined.

Figure 5.6 presents all of the observed and simulated distributions of depth and velocity at Section A-A, located as shown on **Figure 3.2**. This cross section, which is located near the upstream boundary of the reach, is in a steep area of the reach where relatively high mean velocities were experienced at the time of survey. From the velocity graph, it can be seen that there was a section where the ADCP had difficulty in measuring velocity. However, the overall correlation between the depth and velocity data collected with the ADCP and the output from the model was good.

Figure 5.7 shows all of the observed and simulated distributions of depth and velocity at Section B-B, located near the proposed effluent discharge site. This cross section also exists in a steep area where relatively high mean velocities were experienced at the time of survey. Very good correlation was found when comparing the depth and velocity data observed to the output from the model.

All of the observed and simulated distributions of depth and velocity at Section C-C, located near the downstream boundary, are shown in **Figure 5.8**. This cross section exists in a more mildly sloping, lower velocity area, as compared with the first two cross sections. Again, very good correlation was found when comparing the observed data to the output from the model. The results of the comparisons of modeled and observed velocities and depths at the three sites indicated that the calibration was successful and provides some validation for the modeling results. **Figure 5.9** shows the simulated velocity magnitude throughout the entire study reach for the calibration discharge of 1168 m^3/s. As can be seen in the figure, depth-averaged velocities are highest (1.6 m/s) at the upstream end of the reach and decrease towards 1 m/s at the downstream end of the reach. This is a result of the steeper bed slope that exists in the upstream part of the reach (see **Figure 5.3**). **Figure 5.10** displays the bed shear velocity magnitude throughout the reach for the calibration discharge. Again, values of bed shear velocity decrease in the downstream direction along the reach (from approximately 0.14 to 0.07 m/s). **Figure 5.11** presents the flow depth throughout the channel for the calibration discharge. Average values of depth tend to increase towards the downstream end of the channel (ranging from 2.2 to 3.8 m/s), with the deepest areas located at the channel bends. **Figure 5.12** exhibits the cumulative discharge, sometimes referred to as stream function, along the channel for the calibration flow. It can be seen that the flow is very evenly distributed throughout the channel.

5.3.3 Boundary Conditions

A depth-unit discharge relationship downstream boundary condition was used for all the scenarios modeled using *River2D*. This approach allowed for some consideration of the backwater effects at the time of the field program due to the reservoir located downstream of the study site. Based on experience, this type of boundary condition tends to result both in a more stable model than a fixed outflow boundary elevation and in a more natural shape to the water surface at the outflow boundary. This boundary condition takes the form of the following equation:

$$q = Kh^m$$

where q = unit discharge

h = flow depth

K, m = constants

By modifying the value for K from the default of unity to a value of 0.43 and using the default value of 1.666 for the exponent *m*, a good match between the shape of the observed and simulated water surfaces was obtained. These values were used to describe the downstream boundary condition for the simulation of low- and high-flow scenarios. In the case of a continuous intact ice cover, the value of K was decreased to reflect the additional wetted perimeter and roughness introduced by the presence of the ice cover. These values are summarized in **Table 5.1**.

Table 5.1
Modeling Constants for the Open Water and Annual Case

	Default Value	Calibrated Open Channel Flow	Ice-covered Flow
K	1.00	0.43	0.27
m	1.666	1.666	1.666

5.4 Model Simulation Results

Model simulations were carried out for four different flow scenarios. Open water low-flow modeling was conducted on an open water 7Q10 value of 188 m^3/s, as determined through low-flow analysis. Annual low-flow modeling with a 0.7 m thick continuous intact ice cover was conducted for a 7Q10 value of 169 m^3/s, as determined through low-flow analysis. Model results for the mean annual-flow conditions of 439 m^3/s were considered. A high-flow scenario model was run using a 1:100-year flow value of 4770 m^3/s, as determined through flood frequency analysis. These values were chosen to represent a broad range of conditions that may be experienced throughout the reach. Assessments of the water surface elevations, velocity magnitude, shear velocity magnitude, flow depth and cumulative discharge across the channel were made at the proposed outfall site for each flow scenario, with special consideration placed on the velocity and flow depth as parameters most relevant to design. **Figures 5.13 through 5.32** show the various modeled results under each flow scenario. These results were compared with the calibration flow of 1168 m^3/s, where relevant.

5.4.1 Water Surface Elevation

Figures 5.13 through 5.16 display the simulated water surface elevations throughout the study reach. **Table 5.2** presents the modeled water surface elevations at the upstream and downstream boundary for each flow scenario. For the low-flow ice-covered scenario, the water surface elevation represents the top of the phreatic water surface assuming that the specific density of ice is 0.92. Due to the channel geometry, the slope of the water surface flattens significantly in the downstream half of the reach under each flow scenario.

Table 5.2
Modeled Water Surface Elevations at the Upstream and Downstream Boundary

Scenario (Discharge)	Upstream Water Surface Elevation (m)	Downstream Water Surface Elevation (m)
Open Water Low Flow (188 m^3/s)	351.59	347.66
Annual Low Flow (169 m^3/s)	352.35	348.66
Mean Annual Flow (439 m^3/s)	352.02	348.53
High Flow (4770 m^3/s)	357.15	354.95

5.4.2 Flow Depth

Figures 5.17 through 5.20 present the simulated flow depth throughout the study reach under each of the flow conditions. The figures show that flow depths generally increase in the downstream direction. Again, this is due to the change in slope of the channel and the backwater effects from the reservoirs located downstream of the study site. Flow depths are lowest under low-flow scenarios. Under open water low-flow conditions, the site of the proposed outfall structure is located in an area where the flow transitions from a shallow to a deep section, with depths ranging from 0.9 to 2.0 m along the section. Under ice-covered annual low-flow conditions, depths near the proposed outfall site range from 1.1 to 2.2 m along the section. It can be seen that there is a slight increase in flow depth from the open water to the ice-covered case. This is due to the increased resistance in the channel from the roughness of the bottom of the ice cover. The open water low-flow case should be considered in the placement of the outfall or diffuser structure during detailed design. **Table 5.3** displays the flow depth at the proposed outfall structure site as well as at the upstream and downstream boundary for each flow scenario.

Table 5.3
Modeled Flow Depth at the Site and Upstream and Downstream boundary

Scenario (Discharge)	Proposed Site Flow Depth Range (m)	Upstream Average Flow Depth (m)	Downstream Average Flow Depth (m)
Open Water Low Flow (188 m^3/s)	0.9 to 2.0	0.8	1.3
Annual Low Flow (169 m^3/s)	1.1 to 2.2	0.9	1.6
Mean Annual Flow (439 m^3/s)	1.7 to 2.8	1.2	2.1
High Flow (4770 m^3/s)	7.9 to 9.0	6.1	8.4

5.4.3 Velocity Magnitude

Figures 5.21 through 5.24 show the simulated velocity magnitude throughout the study reach under each of the flow conditions. From the figures, it can be seen that there are higher average velocities at the upstream half of the reach as compared to the downstream portion. This is due to the change in slope of the channel and the backwater effects from the reservoirs located downstream of the study site. For the low-flow scenarios, the highest velocities exist in the regions located just upstream of each of the main bends in the channel. For the mean annual-flow scenario, the increases in velocities experienced in the areas just upstream of the main channel bends are less pronounced than at the low-flow scenario and there is more of a consistent gradual decrease in velocity throughout the reach. For the high-flow scenario, the greatest velocities are experienced inside the main channel bends, with values reaching 3 m/s. The high-flow scenario represents the critical case in terms of velocities experienced at the site and must be considered in the final design. **Table 5.4** reports the velocity magnitude at the proposed outfall structure site as well as at the upstream and downstream boundary for each flow scenario. It can be seen that the modeled velocities at the site are very similar to the velocities at the upstream boundary of the reach.

Table 5.4
Modeled Velocity Magnitude at the Site and Upstream and Downstream Boundary

Scenario (Discharge)	Proposed Site Velocity (m/s)	Upstream Average Velocity (m/s)	Downstream Average Velocity (m/s)
Open Water Low Flow (188 m^3/s)	0.8	0.8	0.5
Annual Low Flow (169 m^3/s)	0.6	0.6	0.4
Mean Annual Flow (439 m^3/s)	1.1	1.1	0.7
High Flow (4770 m^3/s)	2.2	2.2	1.7

5.4.4 Bed Shear Velocity Magnitude

Figures 5.25 through 5.28 display the simulated bed shear velocity magnitude throughout the study reach under each of the flow conditions. The figures show that the trends followed by the bed shear velocity magnitude are very similar to that of the velocity magnitude, where the maximum bed shear velocities are experienced in the areas just upstream of, or in the main channel bends, depending on the flow. As with the velocity magnitude, the highest values of the bed shear velocity magnitude result from the high-flow scenario. This case should be considered in the final design. **Table 5.5** presents the bed shear velocity magnitude at the proposed outfall structure site as well as at the upstream and downstream boundary for each flow scenario.

Table 5.5
Modeled Bed Shear Velocity Magnitude at the Site and Upstream and Downstream Boundary

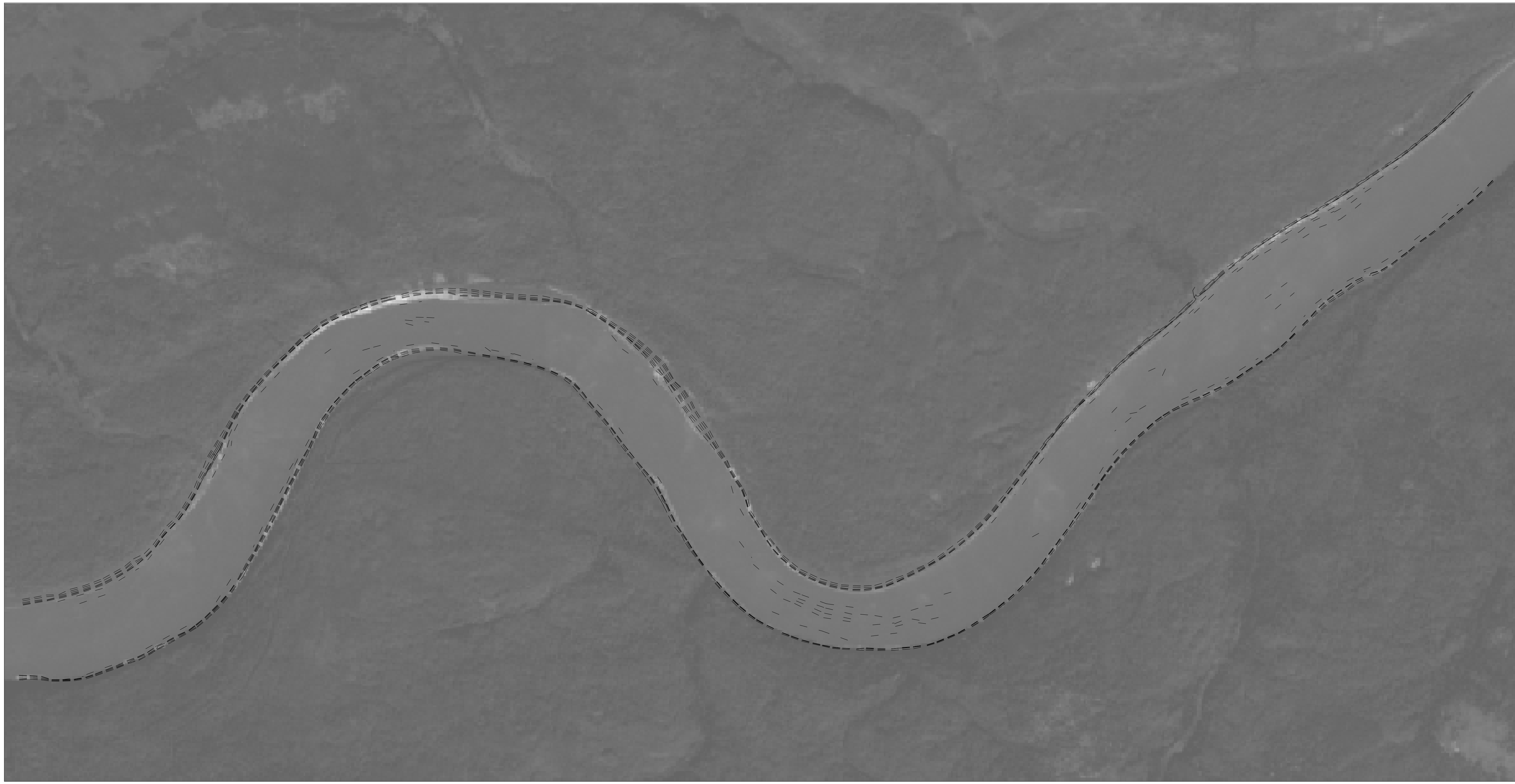
Scenario (Discharge)	Proposed Site Shear Velocity (m/s)	Upstream Average Shear Velocity (m/s)	Downstream Average Shear Velocity (m/s)
Open Water Low Flow (188 m^3/s)	0.07	0.08	0.04
Annual Low Flow (169 m^3/s)	0.06	0.07	0.03
Mean Annual Flow (439 m^3/s)	0.09	0.10	0.05
High Flow (4770 m^3/s)	0.15	0.15	0.10

5.4.5 Cumulative Discharge

Figures 5.29 through 5.32 exhibit the cumulative discharge along the channel under each of the flow conditions. For higher flow conditions, the flow is very evenly distributed throughout the channel. As flows decrease, the distribution becomes less uniform. For low-flow scenarios, flow spreading begins to develop as the channel widens towards to the downstream boundary.

Scale

500.0 m



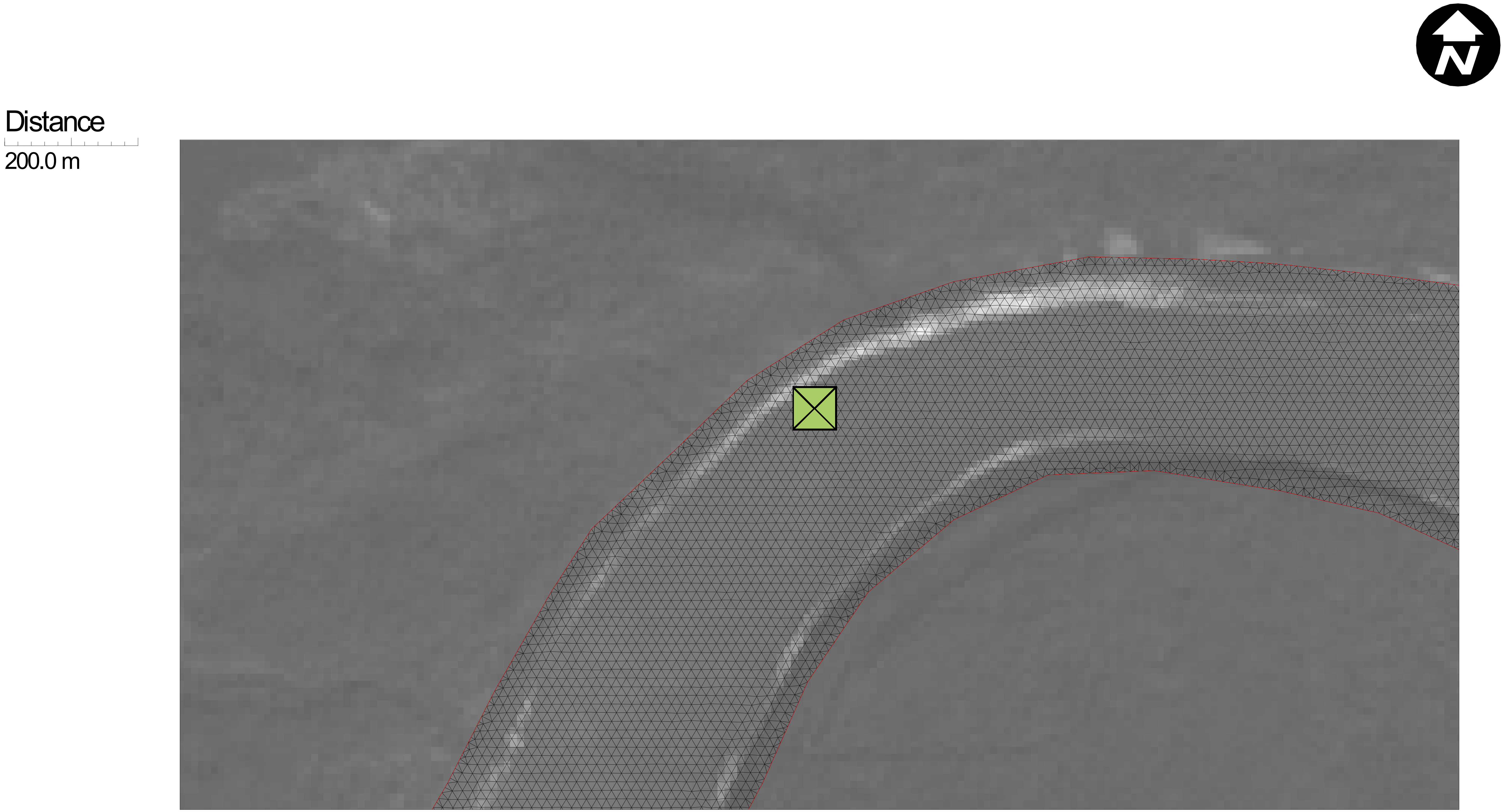
- - - - River2D Breaklines

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.1
DWN BY: RBA	CHK'D BY: RBA	Breakline Locations	amec

Map Path: L:\Water\PROJECT\EW\...



Model Boundary

Outfall/Diffuser (Approximate)

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
			FIGURE 5.2
AMEC Project No.: SX0373305		River2D Computational Mesh Near Proposed Outfall/Diffuser	amec
DWN BY: RBA	CHK'D BY: RBA		

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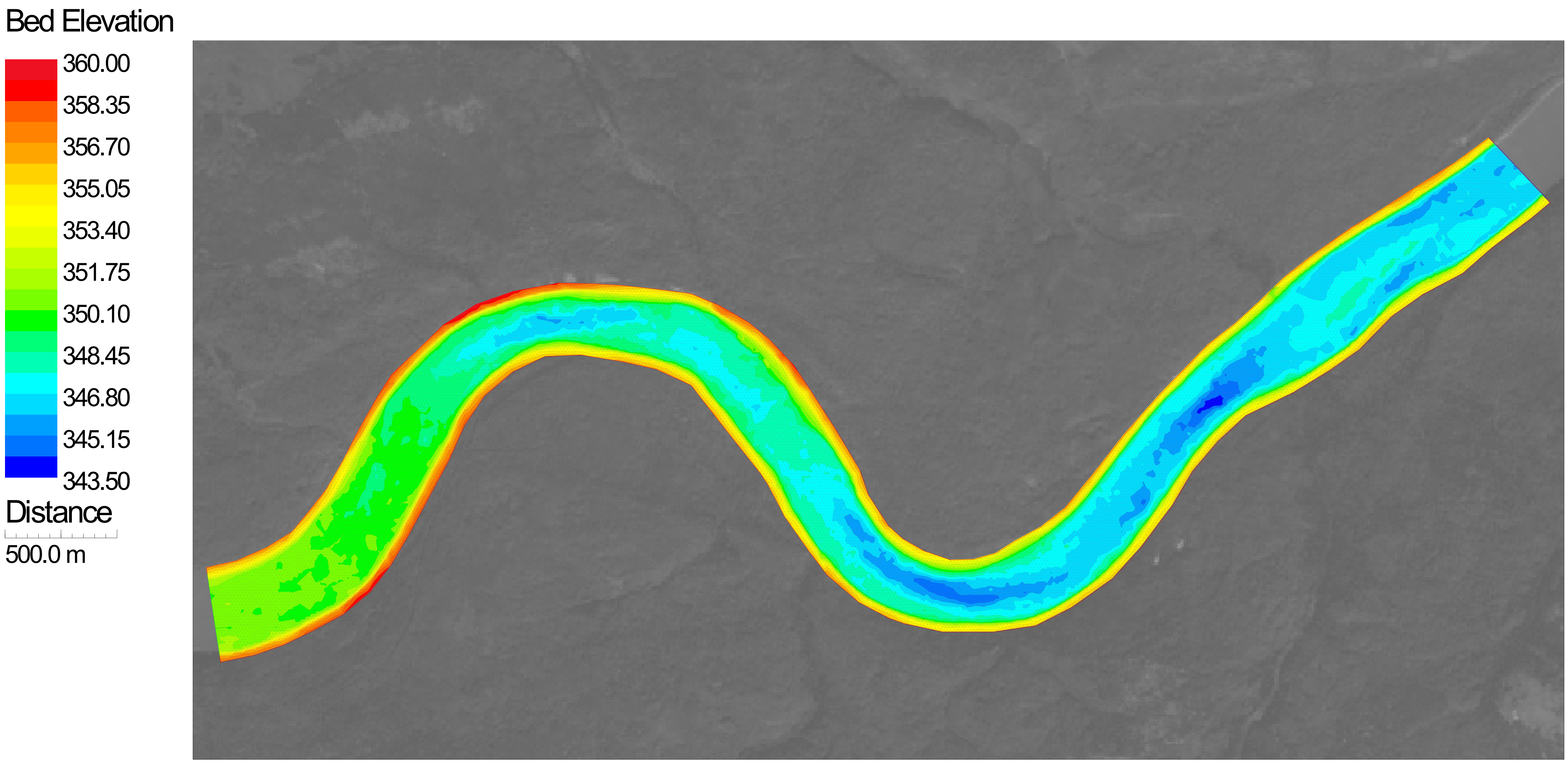


Model Boundary

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.3
DWN BY: RBA	CHK'D BY: RBA	Bed Elevation Contour Map	amec

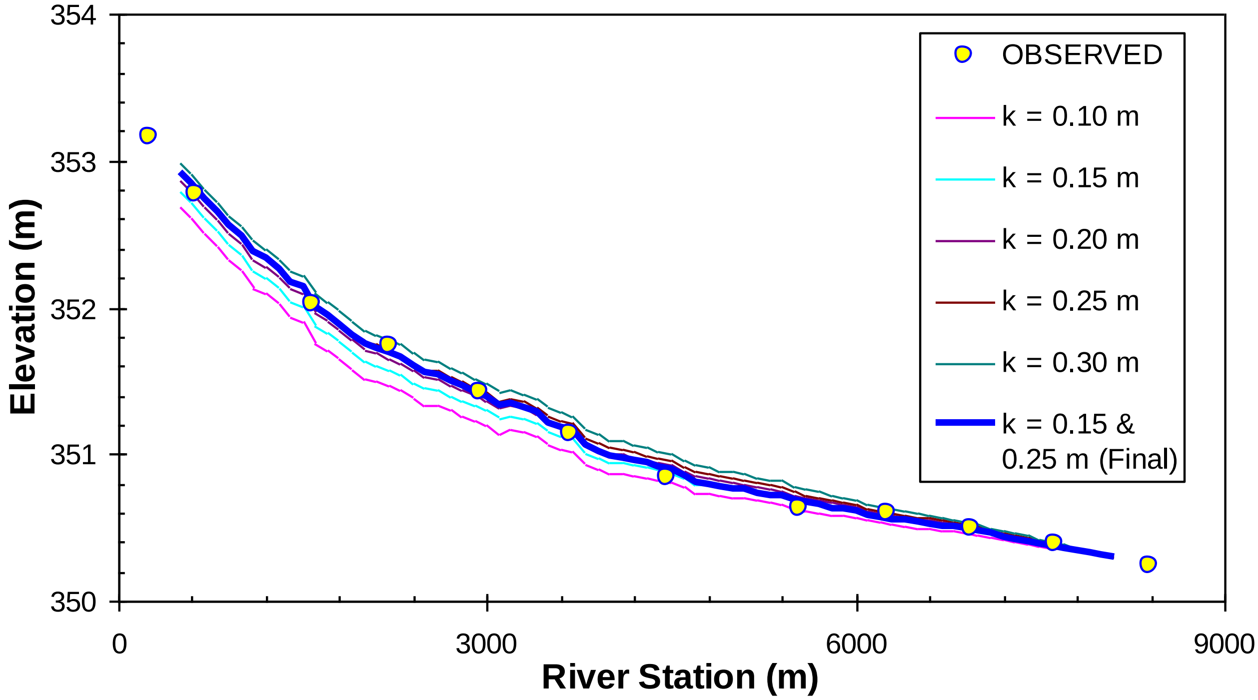
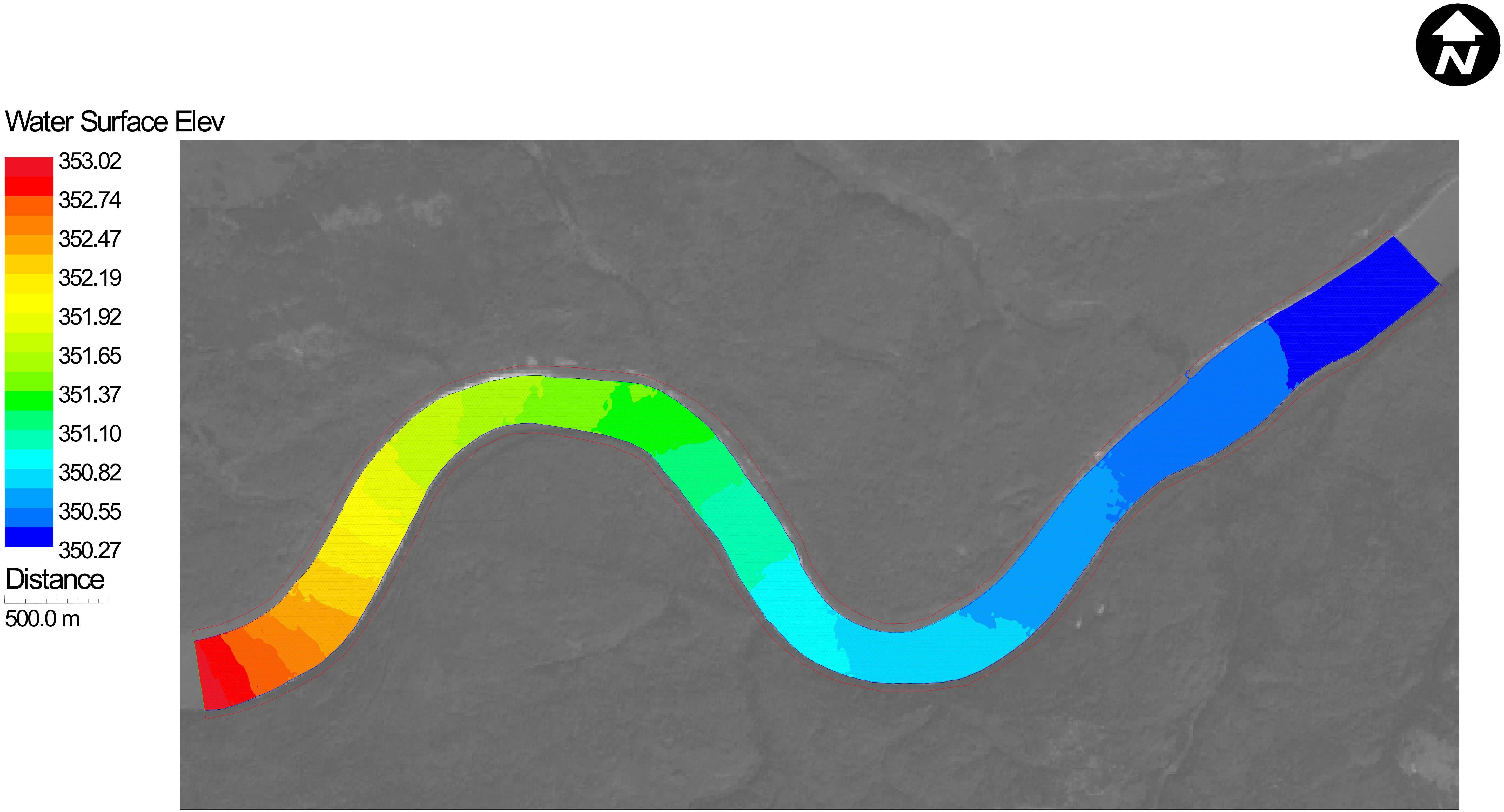


Figure 5.4 Water Surface Profile Calibration (River2D)



Model Boundary

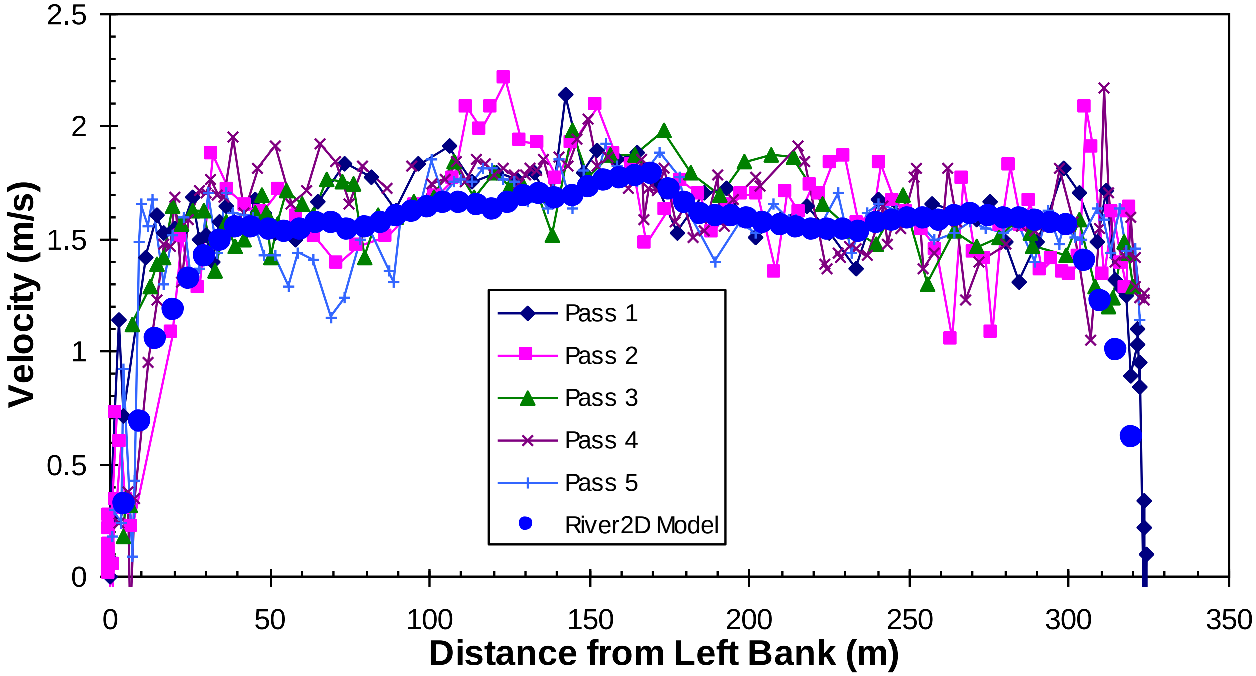
Background Data from GeoBase®

DISCHARGE
1168 m^3/s

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.5
DWN BY: RBA	CHK'D BY: RBA	Simulated Water Surface Elevation (Calibrated Model)	amec

a)



b)

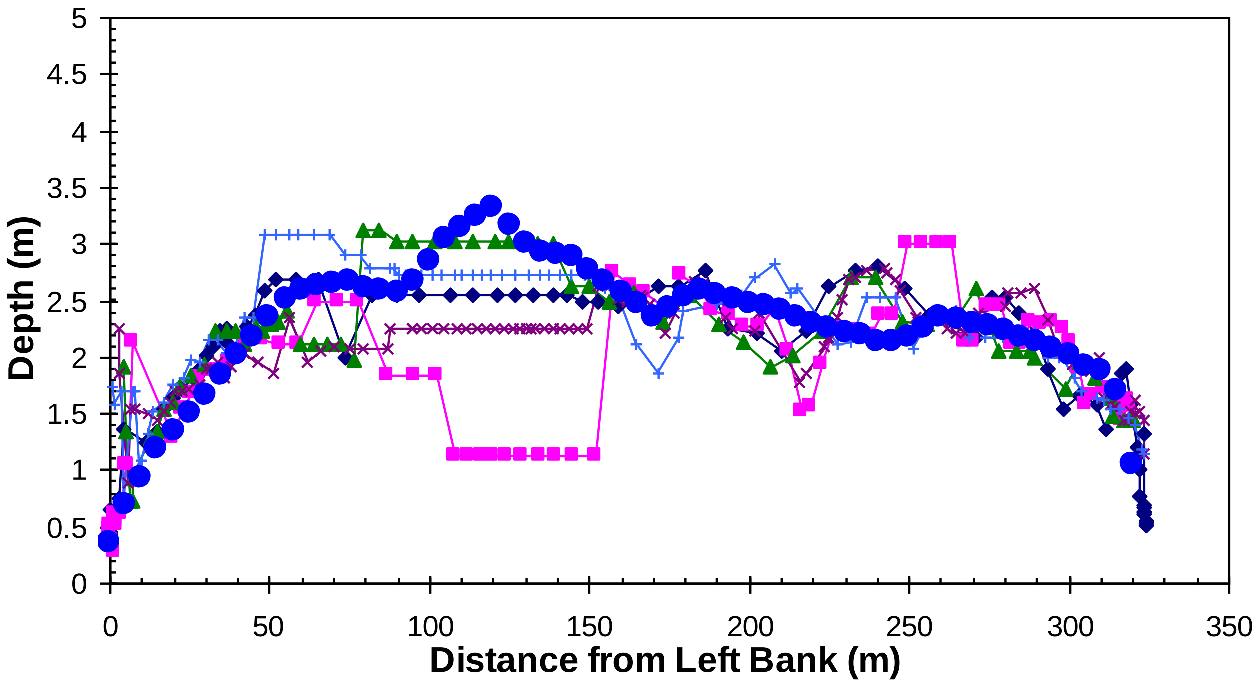
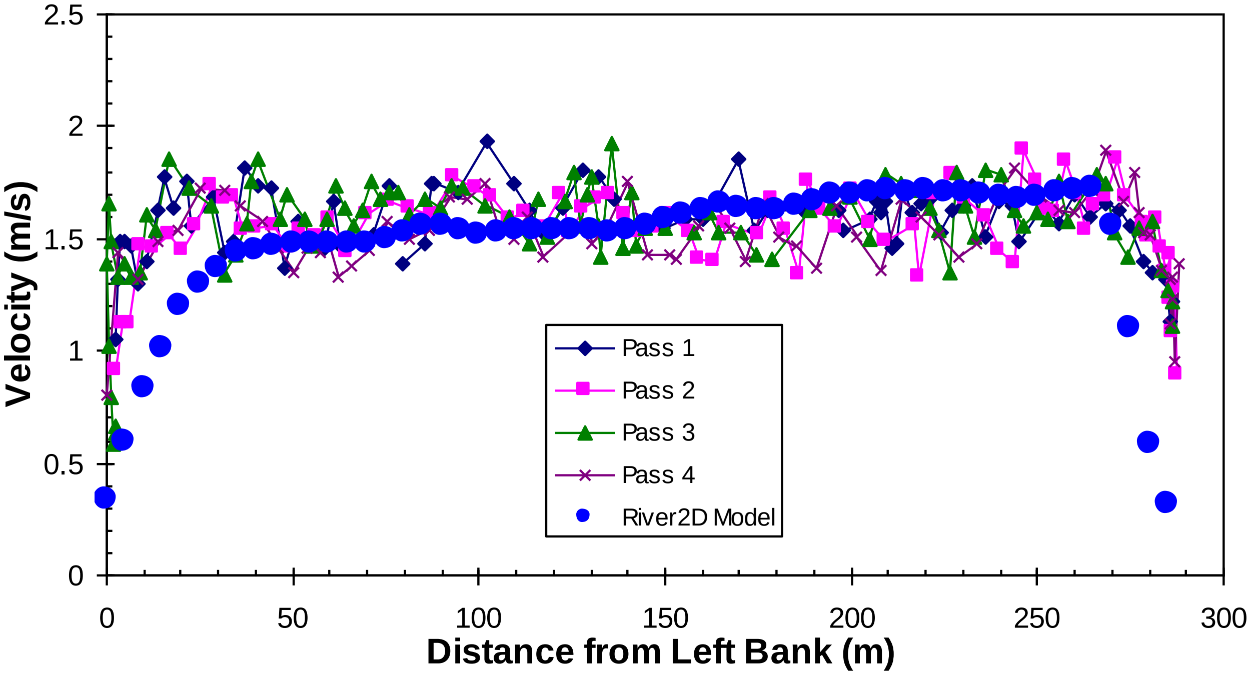


Figure 5.6 Comparisons Between Observed and Modeled (a) Velocity and (b) Depth at Section A-A.

a)



b)

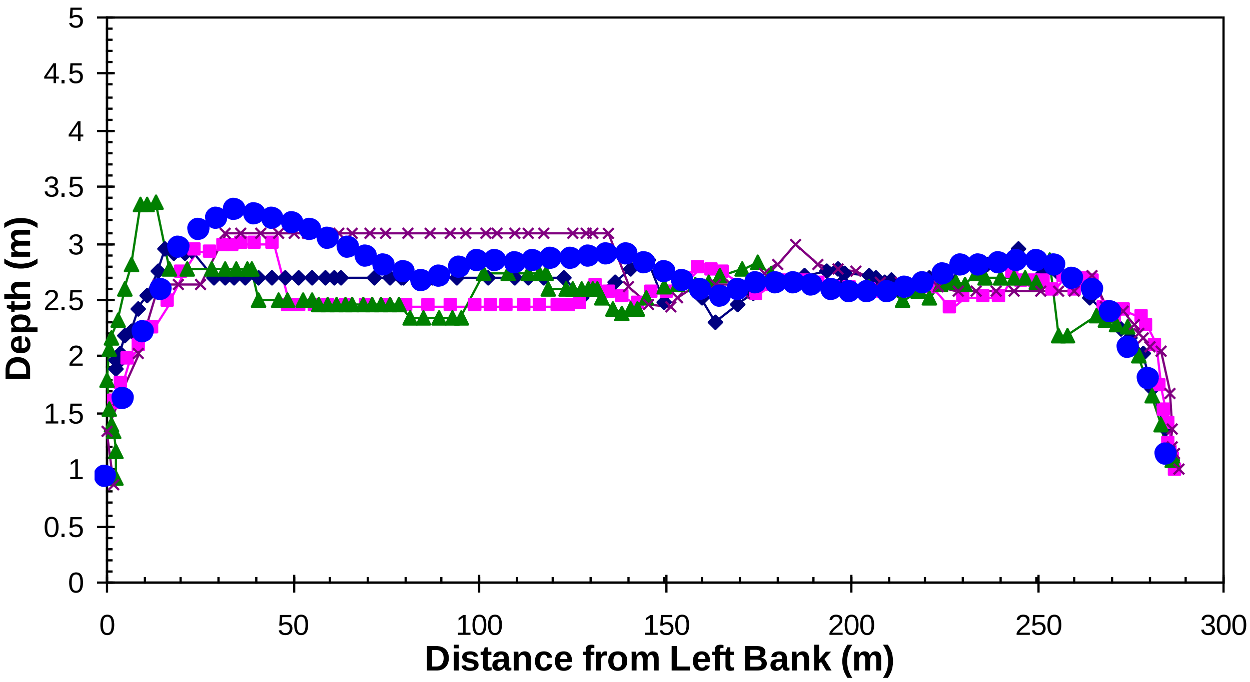
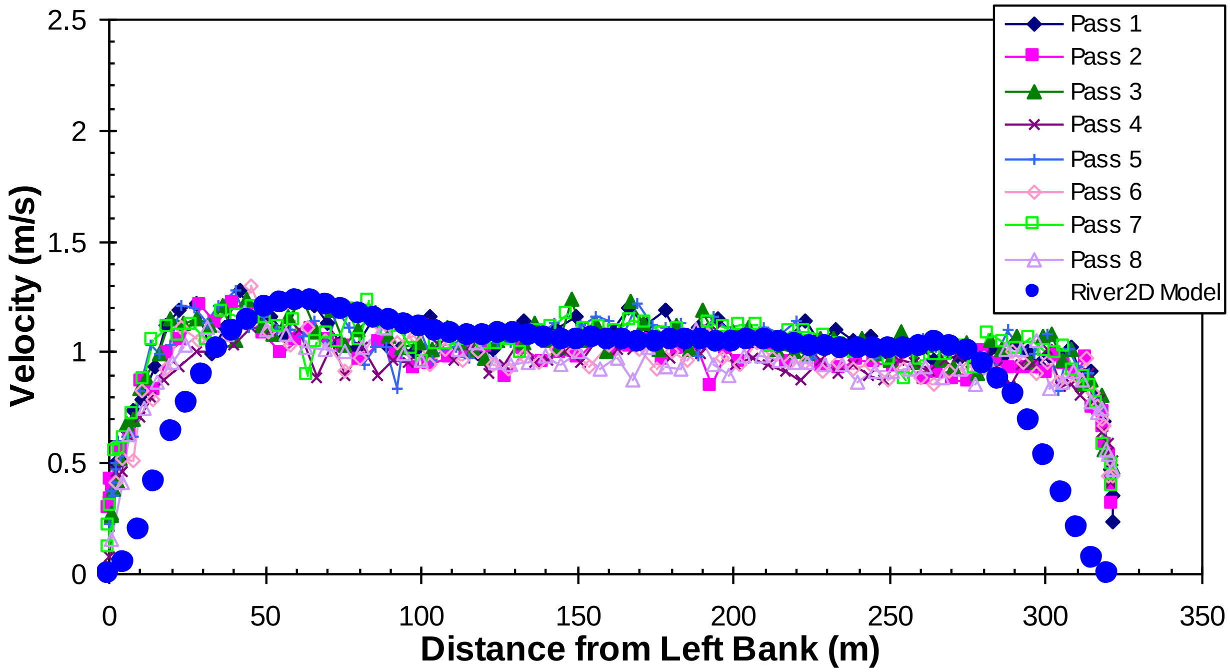


Figure 5.7 Comparisons Between Observed and Modeled (a) Velocity and (b) Depth at Section B-B

a)



b)

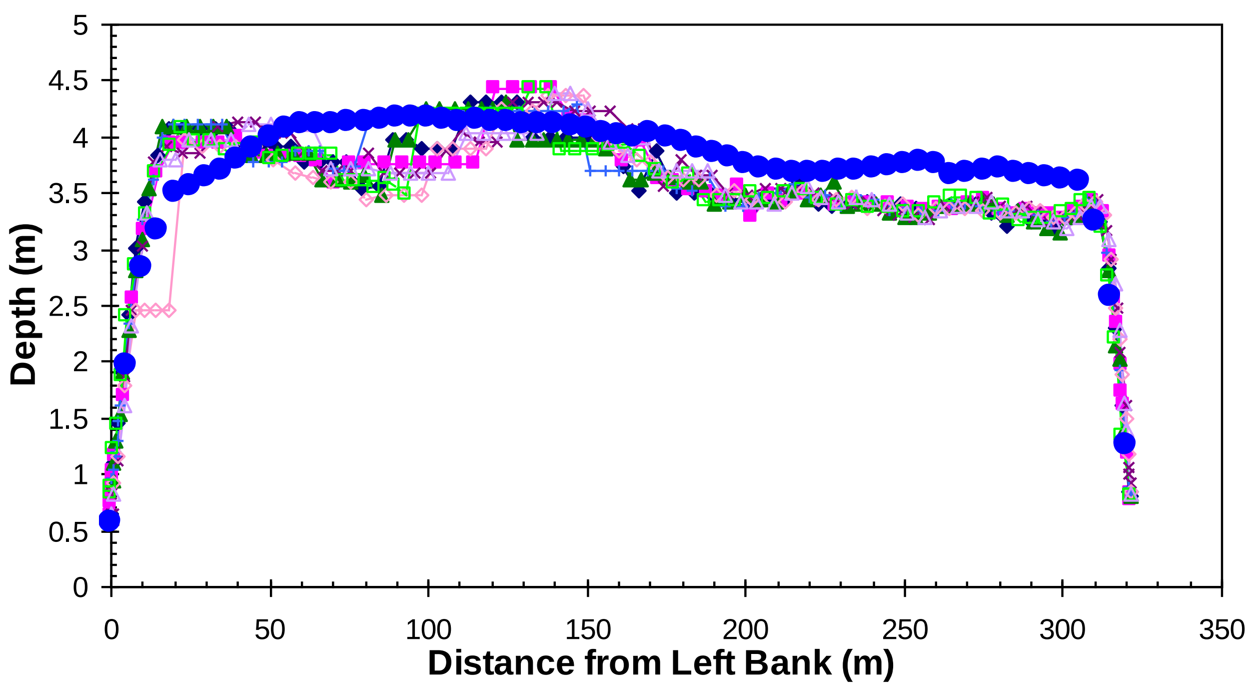
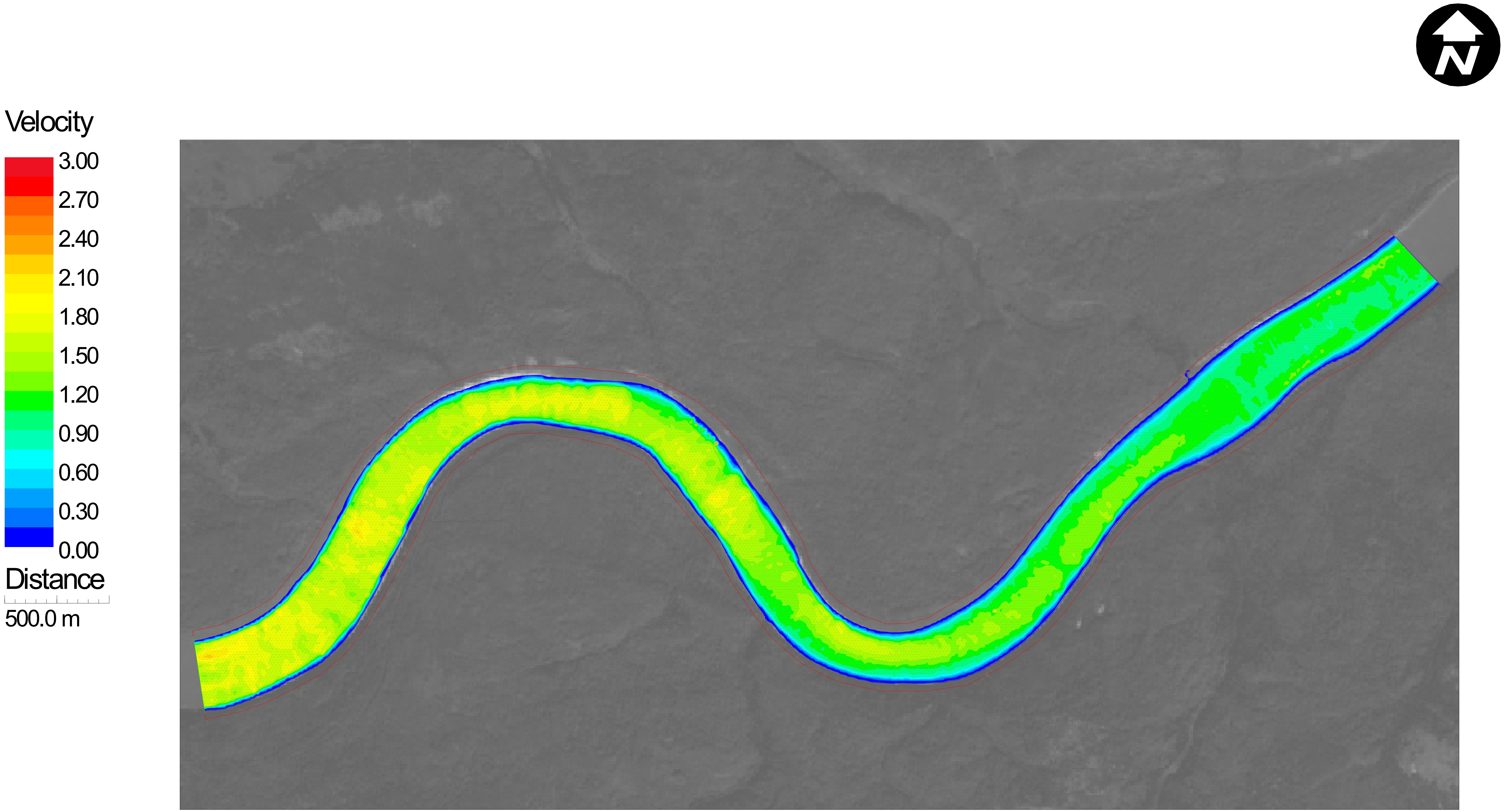


Figure 5.8 Comparisons Between Observed and Modeled (a) Velocity and (b) Depth at Section C-C



Model Boundary

DISCHARGE
1168 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
			FIGURE 5.9
AMEC Project No.: SX0373305		Simulated Velocity Magnitude (Calibrated Model)	amec
DWN BY: RBA	CHK'D BY: RBA		

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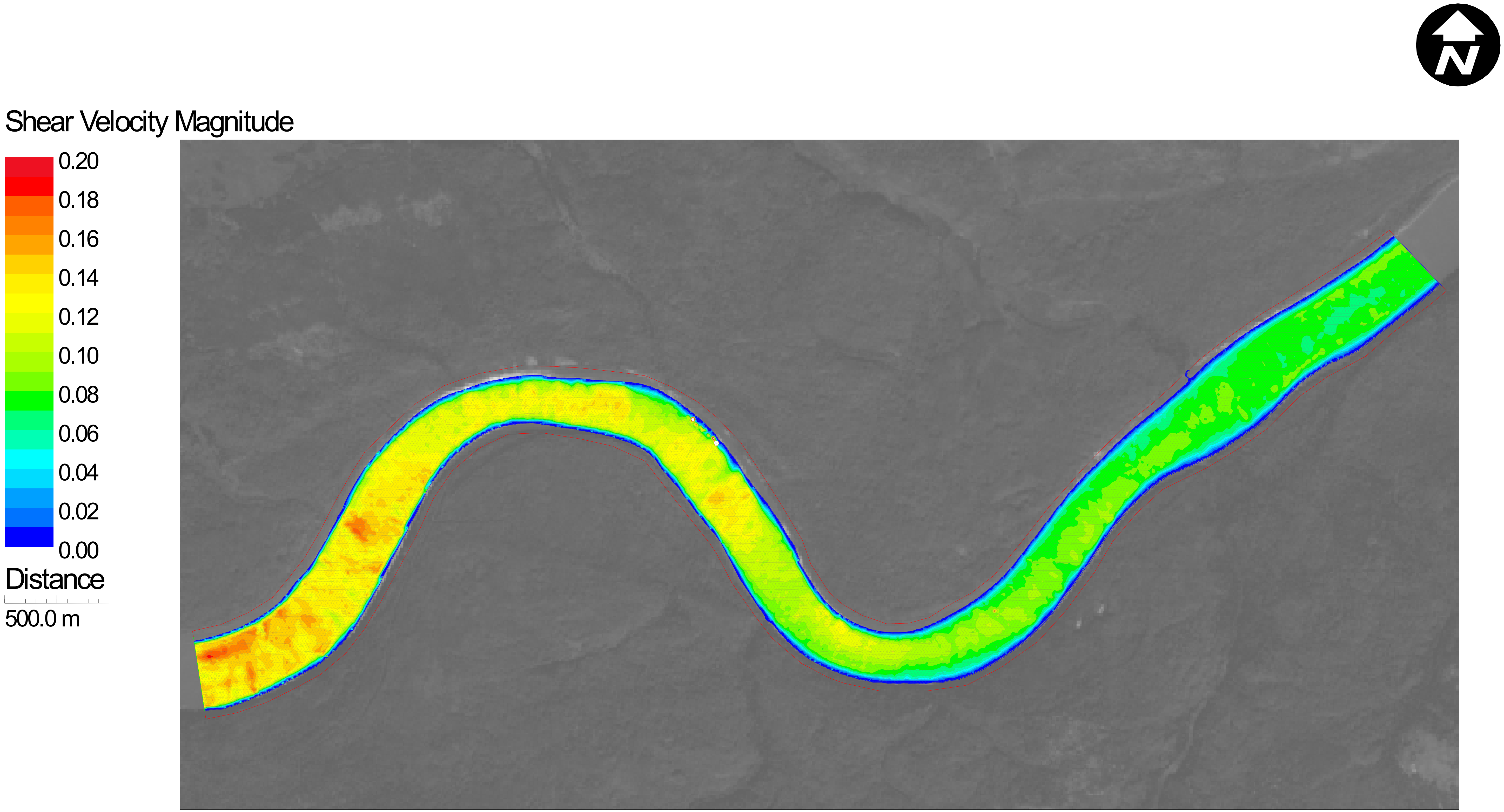

Model Boundary	**DISCHARGE** **1168 m^3/s**	Background Data from GeoBase® **SCALE AS SHOWN**	CLIENT: Shore Gold Inc.	SHORE GOLD RIVER SURVEY	November 2010 FIGURE 5.10
			AMEC Project No.: SX0373305 DWN BY: RBA CHK'D BY: RBA	Simulated Shear Velocity Magnitude (Calibrated Model)	amec

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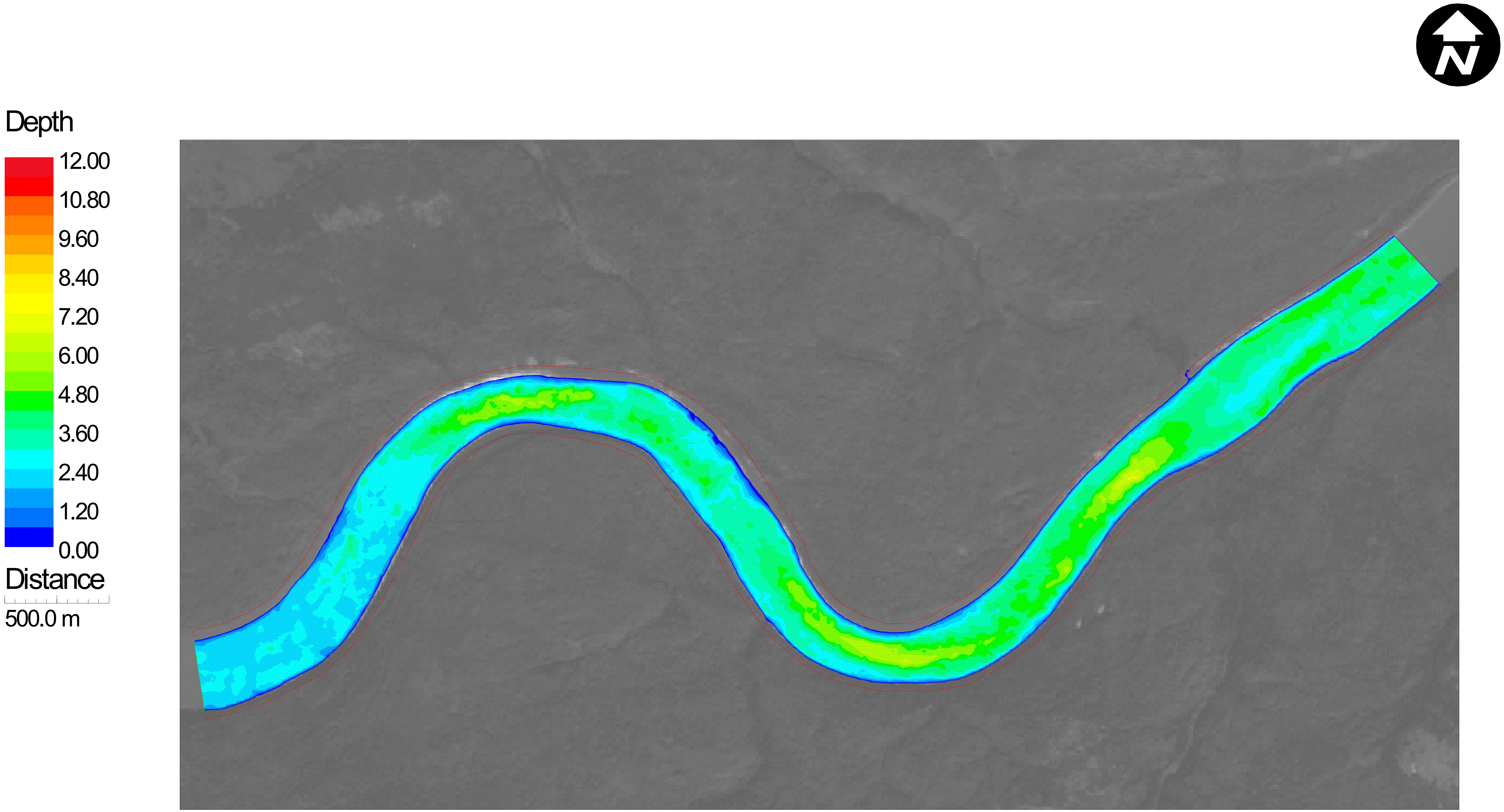

Model Boundary	**DISCHARGE** **1168 m³/s**	Background Data from GeoBase® **SCALE AS SHOWN**	CLIENT: Shore Gold Inc. AMEC Project No.: SX0373305 DWN BY: RBA CHK'D BY: RBA	SHORE GOLD RIVER SURVEY Simulated Depth (Calibrated Model)	November 2010 FIGURE 5.11 amec

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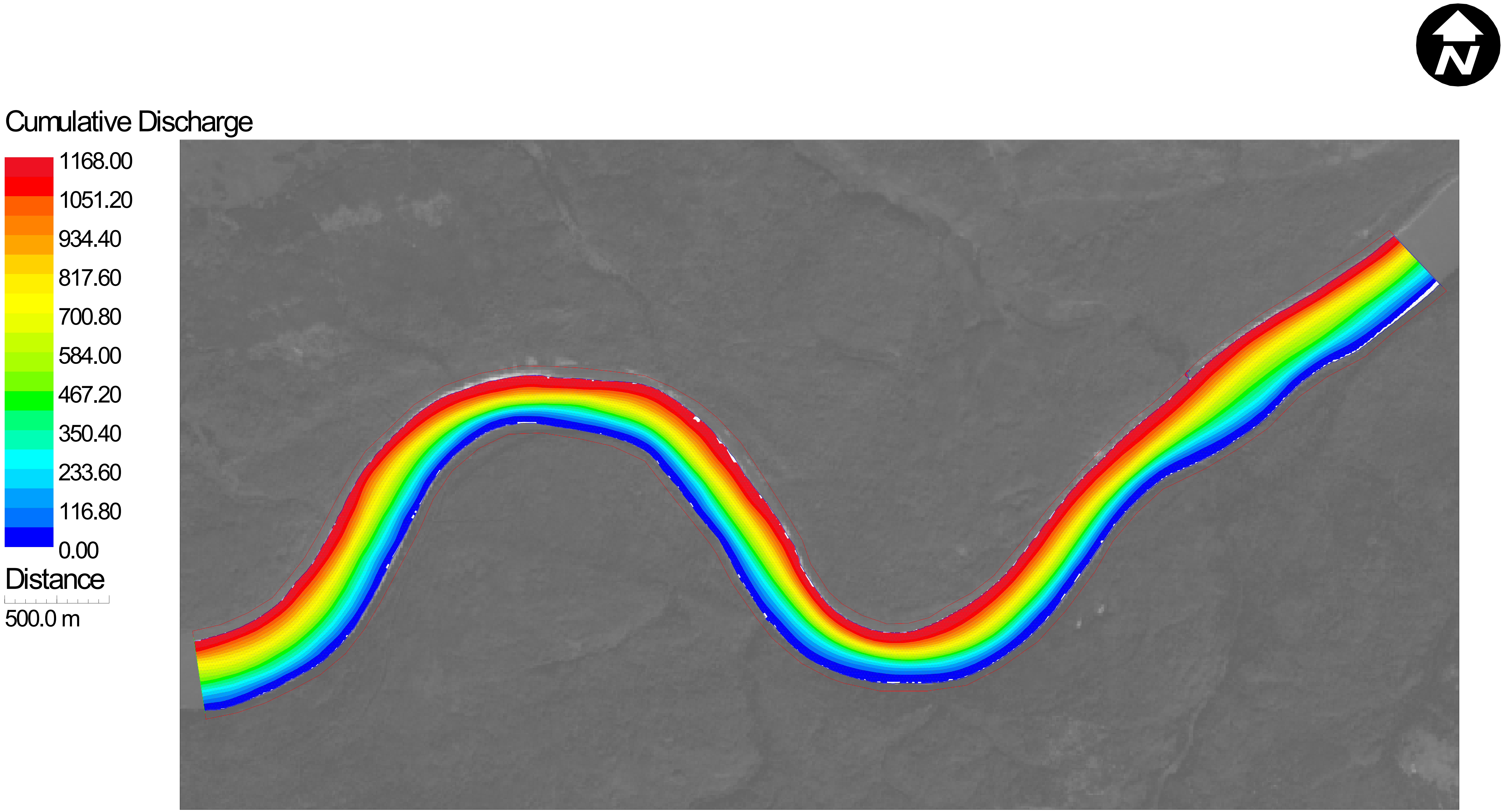

Model Boundary	DISCHARGE 1168 m^3/s	Background Data from GeoBase® SCALE AS SHOWN	CLIENT: Shore Gold Inc. AMEC Project No.: SX0373305 DWN BY: RBA CHK'D BY: RBA	SHORE GOLD RIVER SURVEY Simulated Cumulative Discharge (Calibrated Model)	November 2010 FIGURE 5.12 amec

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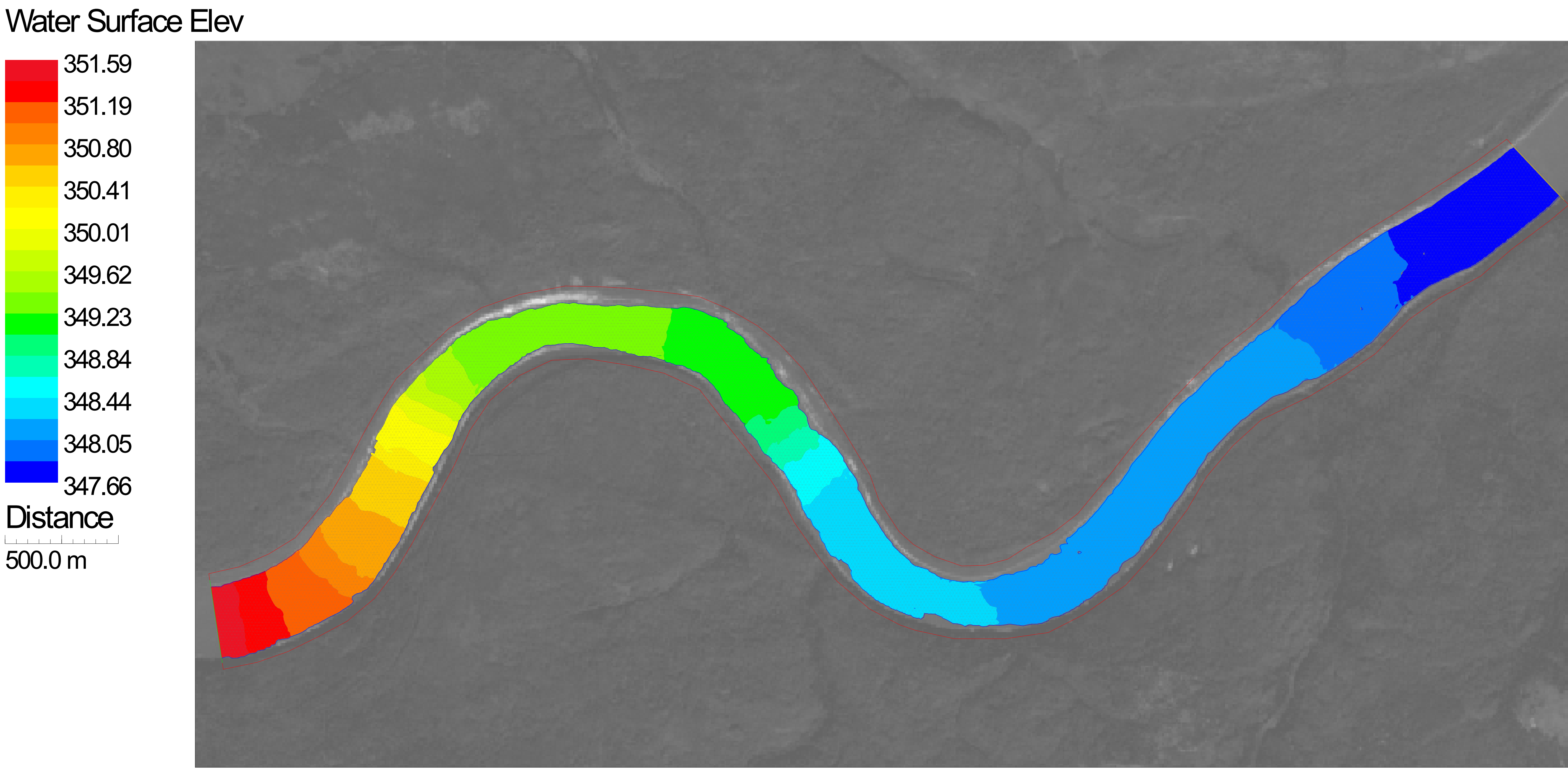

Model Boundary

DISCHARGE
188 m³/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.

AMEC Project No.: SX0373305

DWN BY: RBA

CHK'D BY: RBA

SHORE GOLD RIVER SURVEY

Simulated Water Surface Elevation
(7Q10 Open Water Low Flow)

November 2010

FIGURE 5.13

amec

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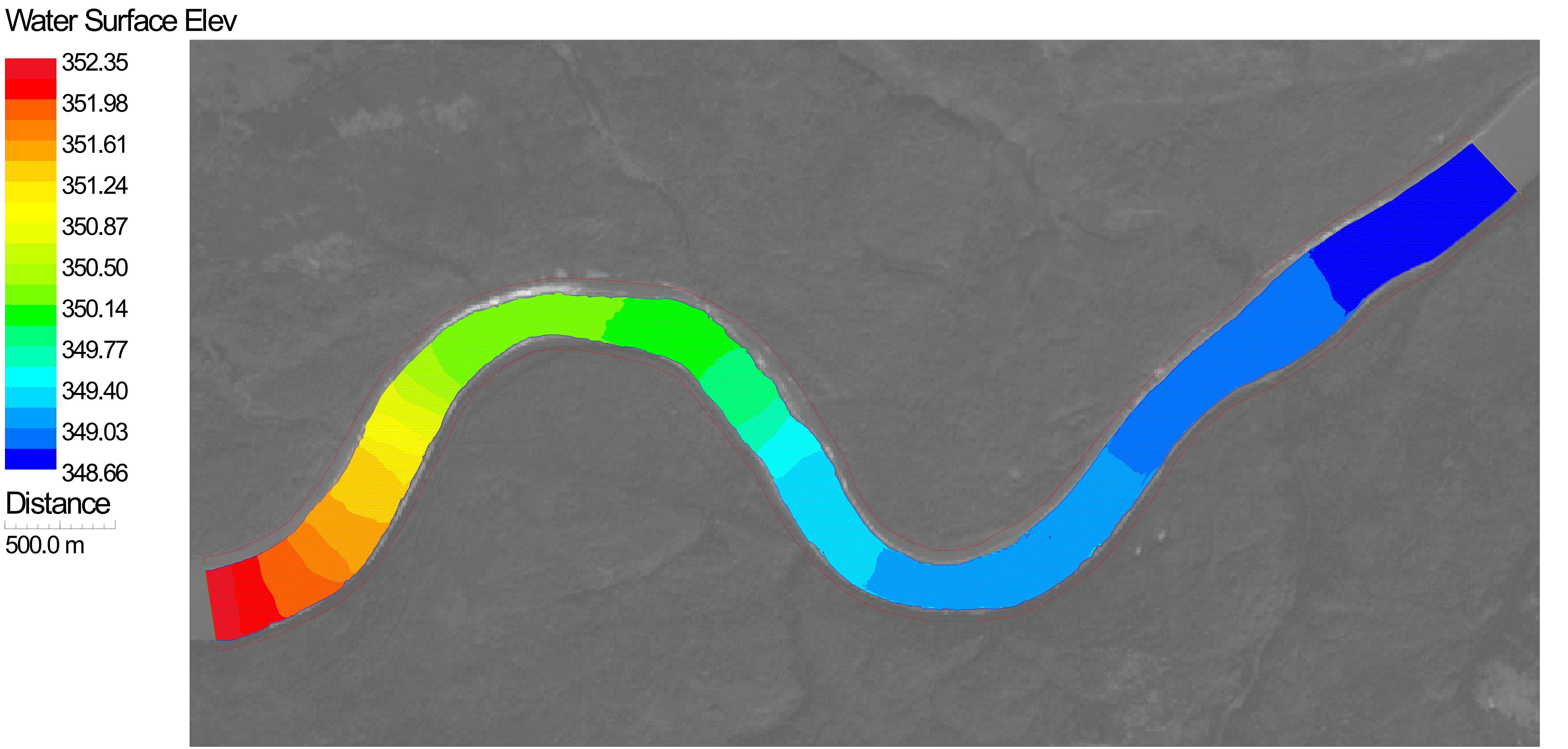
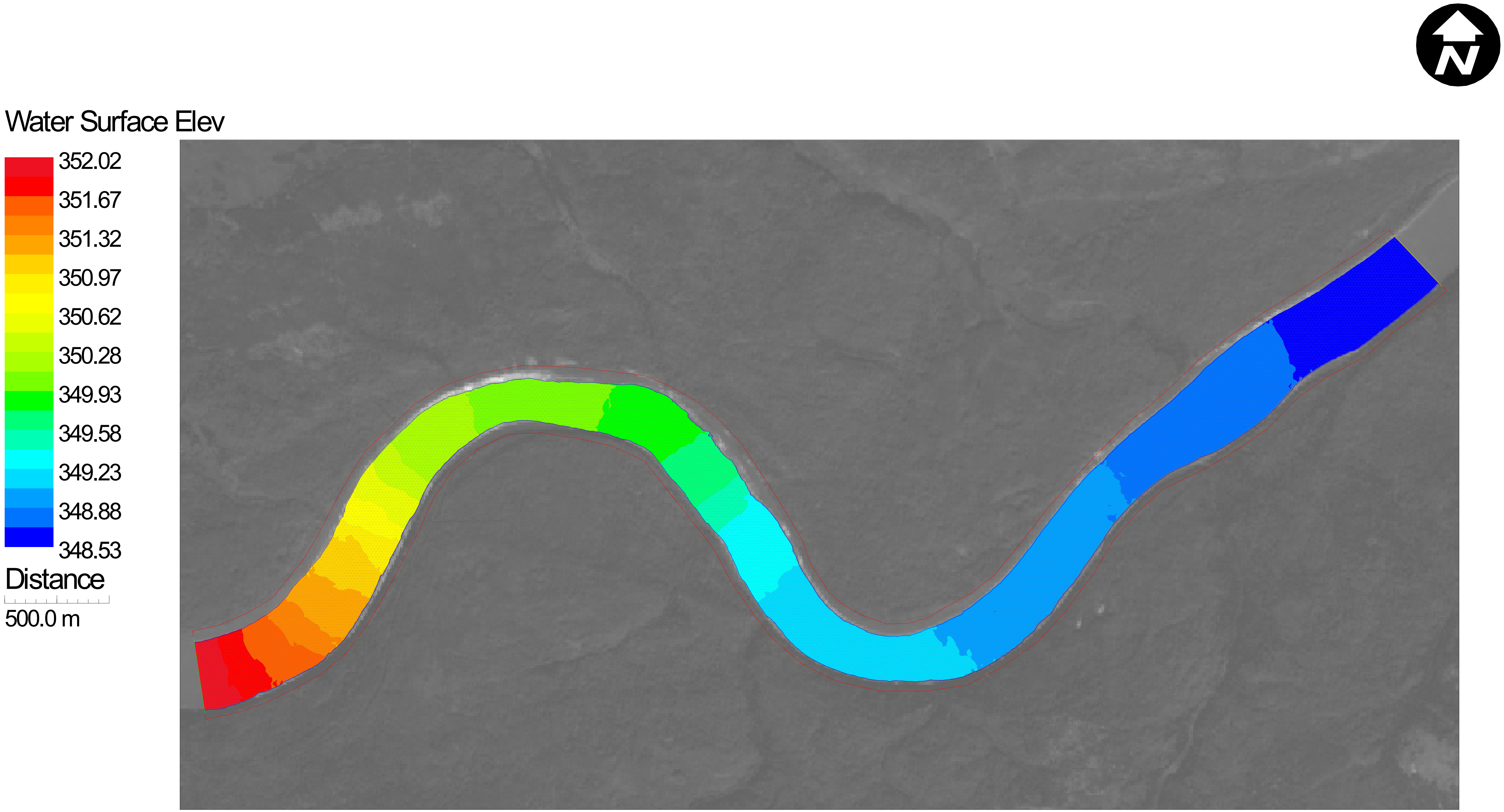


Model Boundary	DISCHARGE 169 m^3/s	Background Data from GeoBase® SCALE AS SHOWN	CLIENT: Shore Gold Inc. AMEC Project No.: SX0373305 DWN BY: RBA CHK'D BY: RBA	SHORE GOLD RIVER SURVEY Simulated Water Surface Elevation (7Q10 Ice-Covered Low Flow)	November 2010 FIGURE 5.14 amec



Model Boundary

DISCHARGE
439 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
			FIGURE 5.15
AMEC Project No.: SX0373305		Simulated Water Surface Elevation (Mean Annual Discharge)	amec
DWN BY: RBA	CHK'D BY: RBA		

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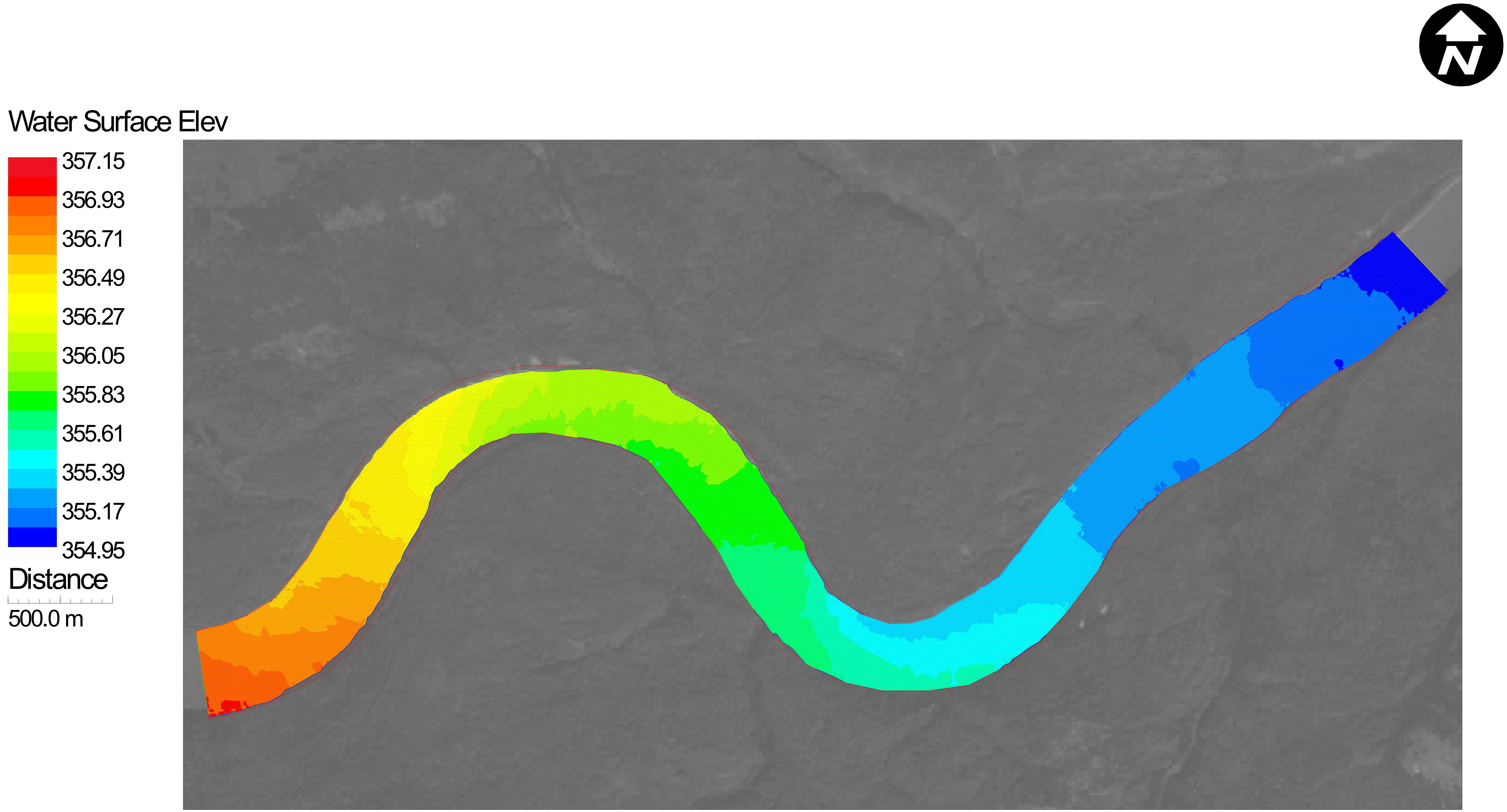

Model Boundary

DISCHARGE
4770 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
			FIGURE 5.16
AMEC Project No.: SX0373305		River2D Simulated Water Surface Elevation (1:100 Year Discharge)	amec
DWN BY: RBA	CHK'D BY: RBA		

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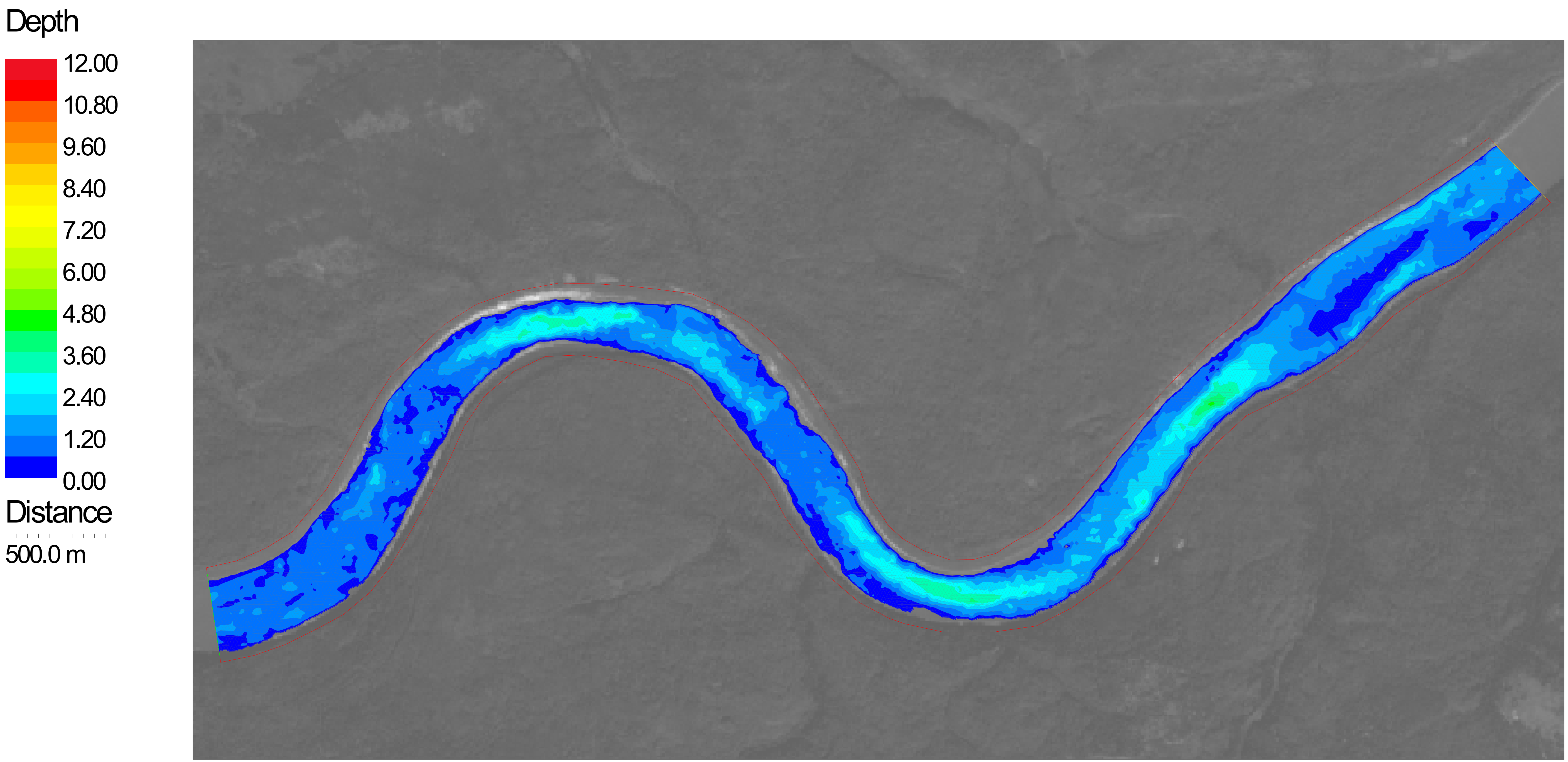

Model Boundary

DISCHARGE
188 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.17
DWN BY: RBA	CHK'D BY: RBA	Simulated Depth (7Q10 Open-Water Low Flow)	amec

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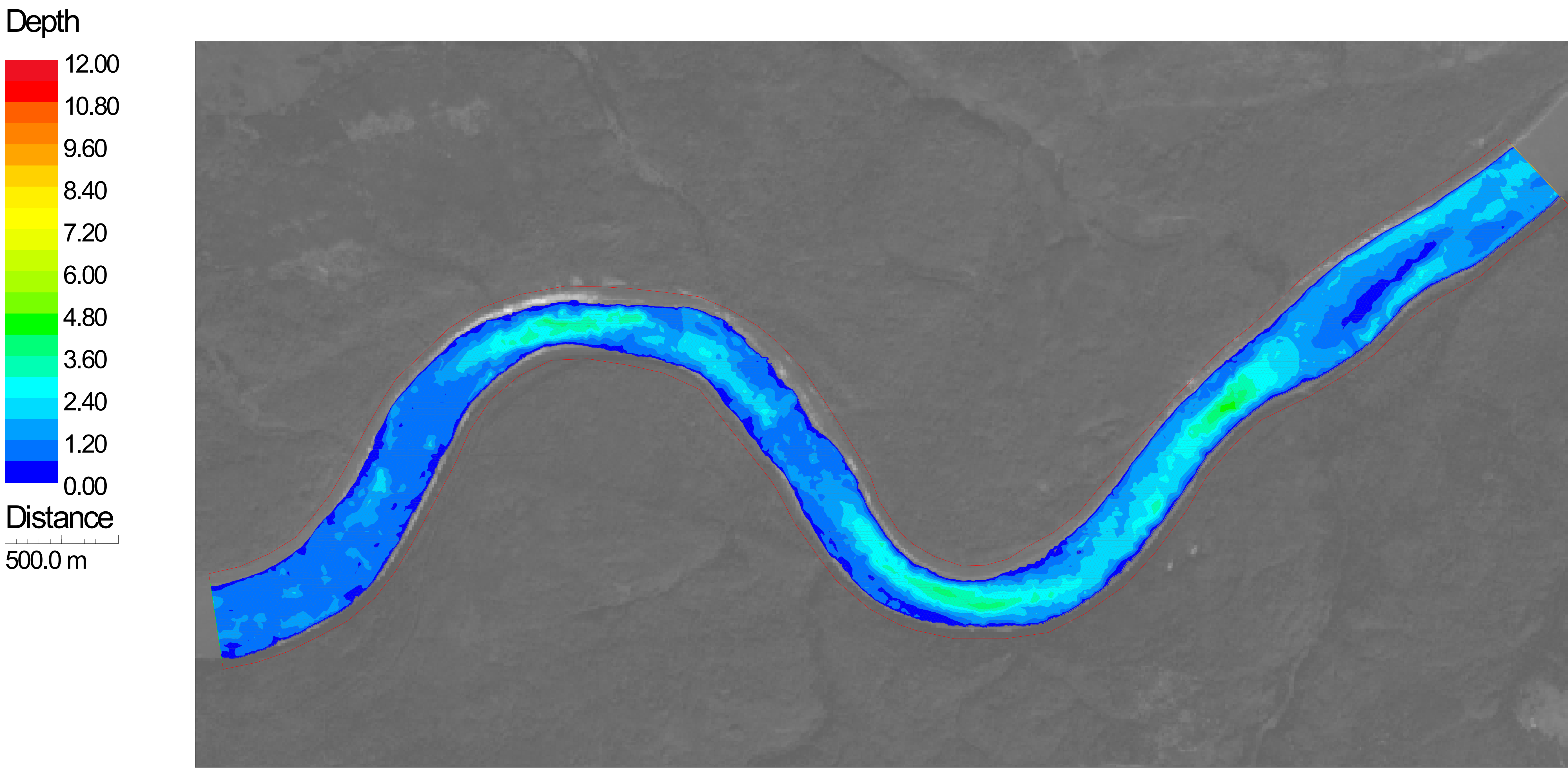


Model Boundary

DISCHARGE
169 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.

AMEC Project No.: SX0373305

DWN BY: RBA | CHK'D BY: RBA

SHORE GOLD RIVER SURVEY

Simulated Depth
(7Q10 Ice-Covered Low Flow)

November 2010

FIGURE 5.18

amec

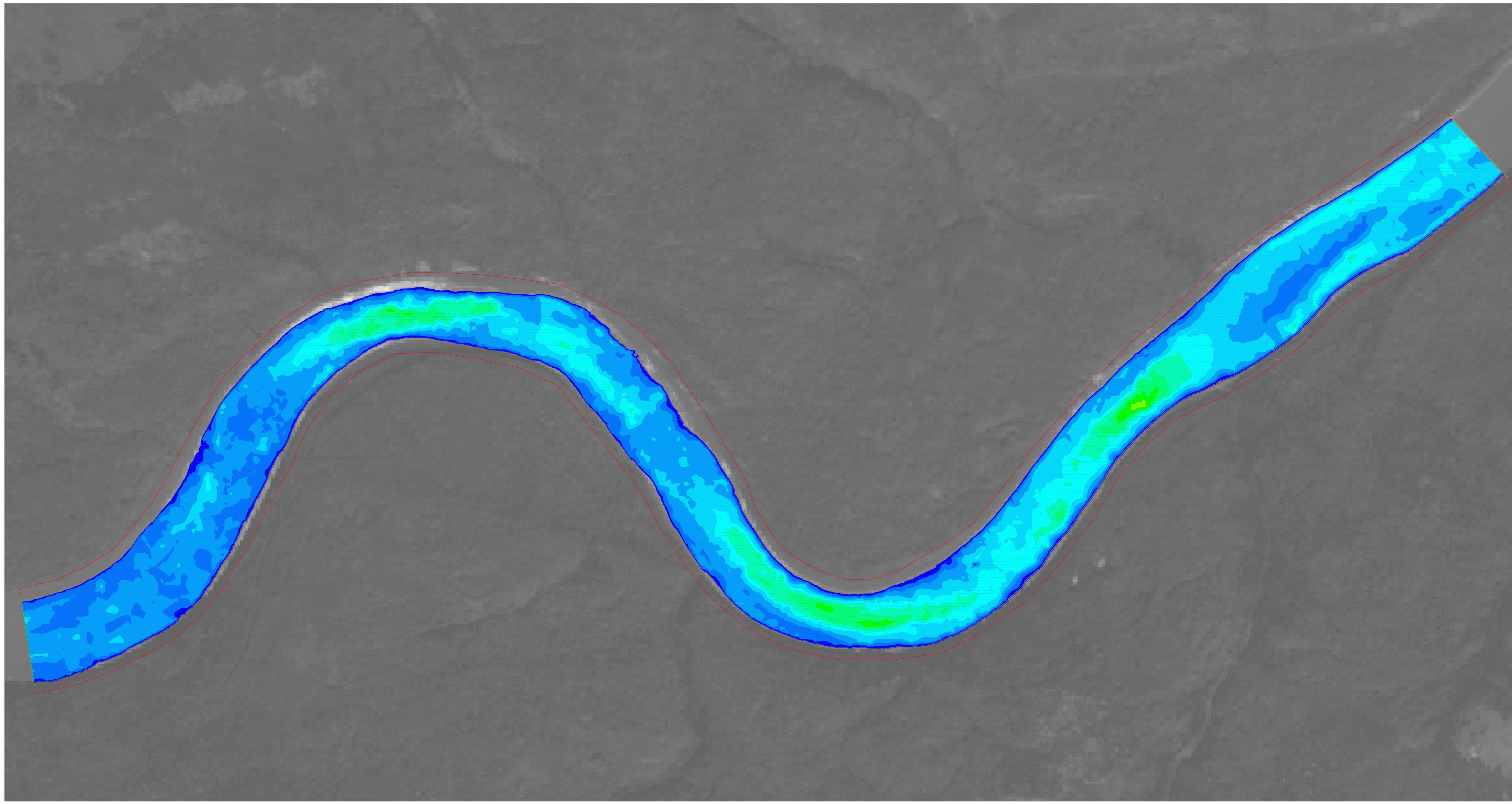
Map Path: L:\Water\PROJECT\EW...

Depth

12.00
10.80
9.60
8.40
7.20
6.00
4.80
3.60
2.40
1.20
0.00

Distance

500.0 m



Model Boundary

DISCHARGE
439 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
			FIGURE 5.19
AMEC Project No.: SX0373305		Simulated Depth (Mean Annual Discharge)	amec
DWN BY: RBA	CHK'D BY: RBA		

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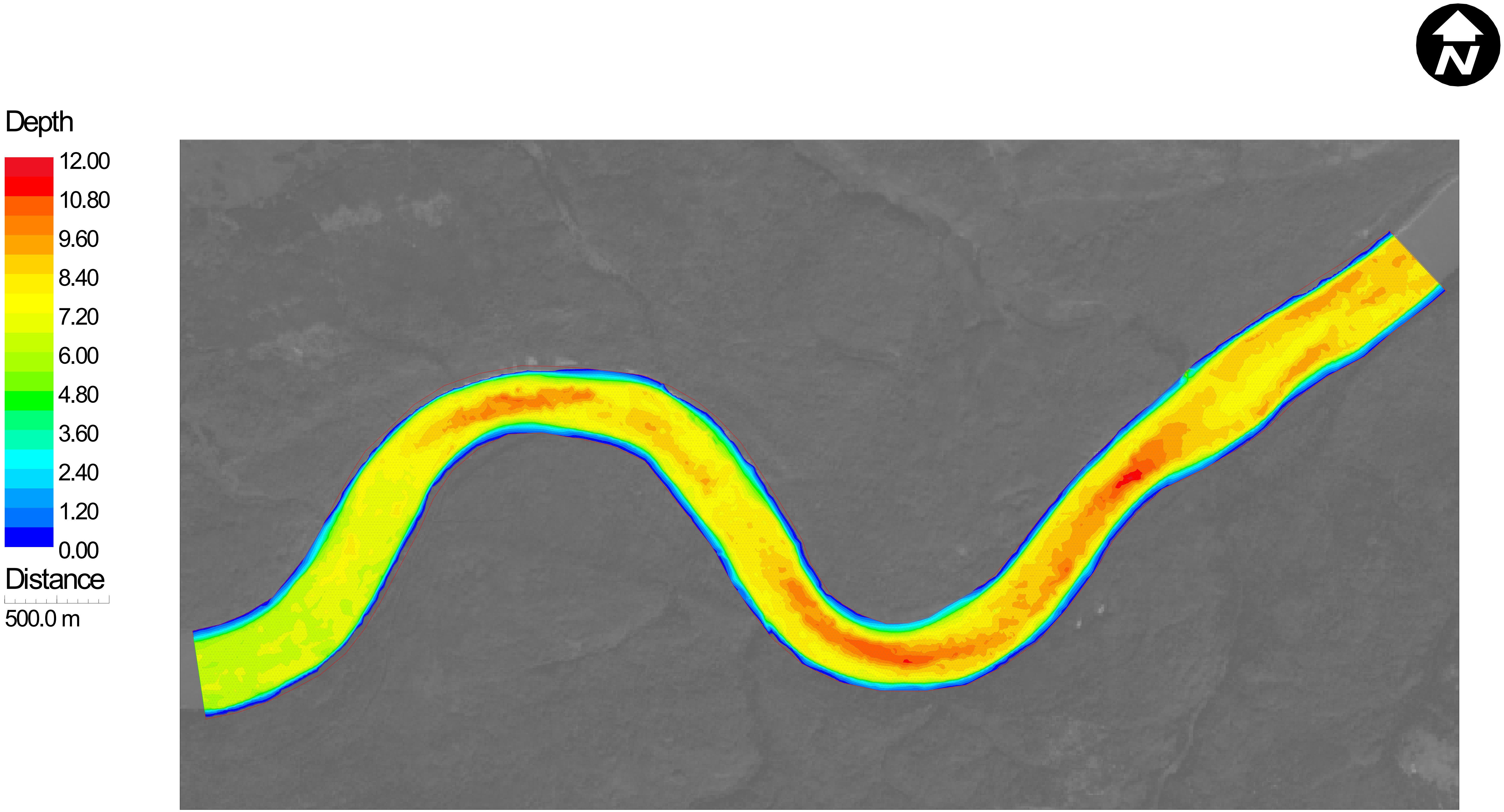

Model Boundary

DISCHARGE
4770 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
			FIGURE 5.20
AMEC Project No.: SX0373305		Simulated Depth (1:100 Year Discharge)	amec
DWN BY: RBA	CHK'D BY: RBA		

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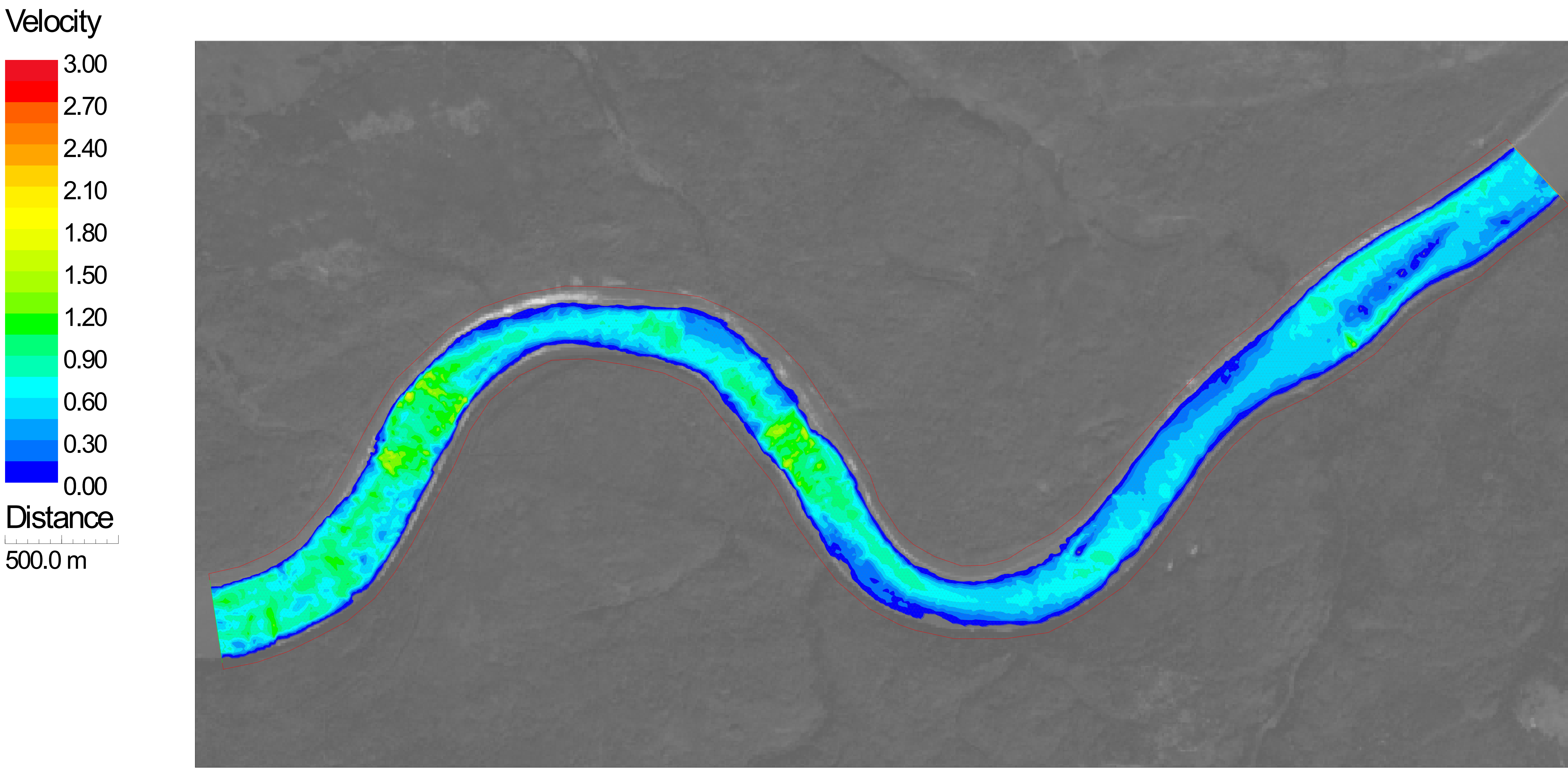

Model Boundary

DISCHARGE
188 m³/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
			FIGURE 5.21
AMEC Project No.: SX0373305		Simulated Velocity Magnitude (7Q10 Open-Water Low Flow)	amec
DWN BY: RBA	CHK'D BY: RBA		

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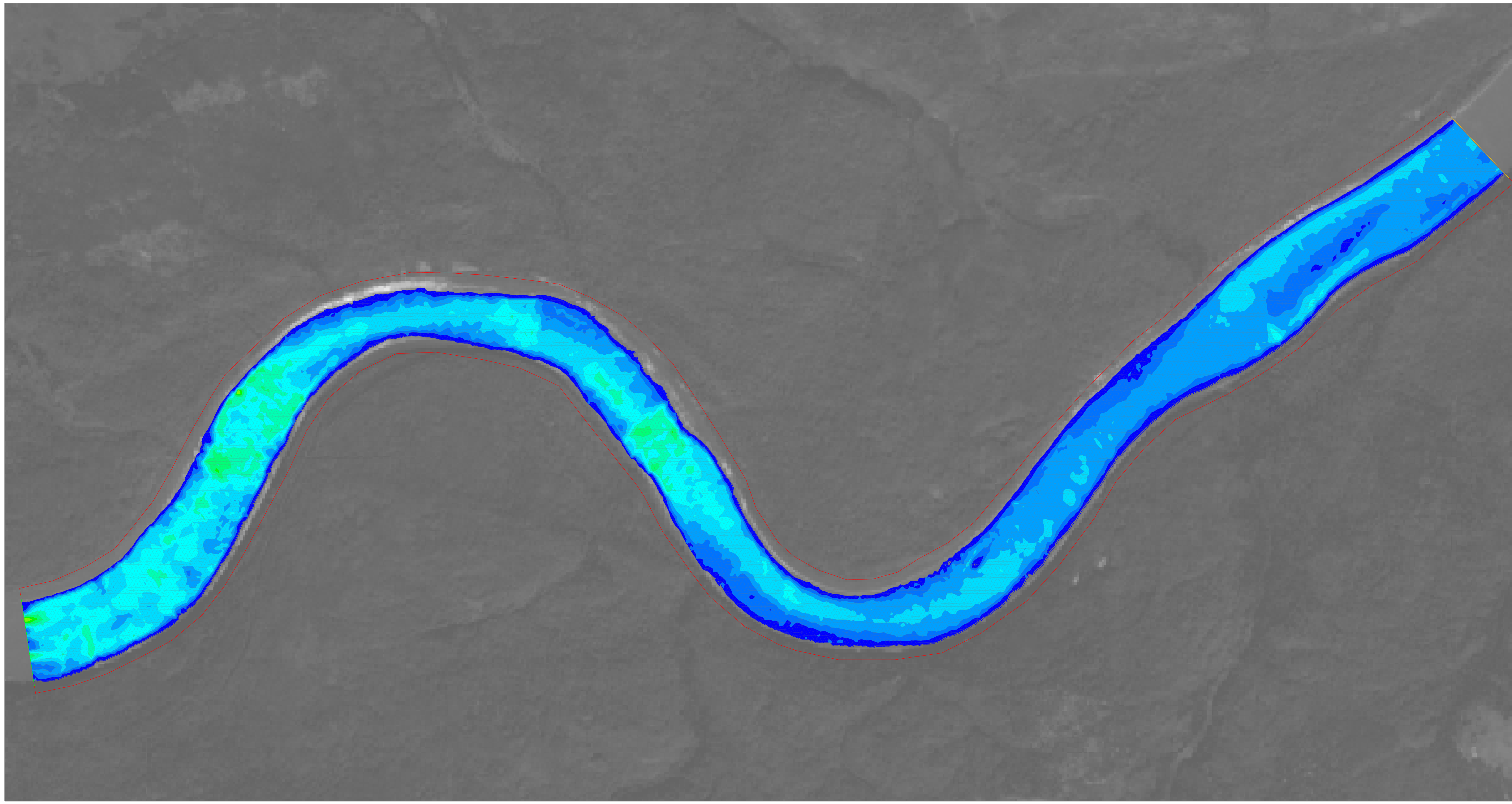
Velocity

3.00
2.70
2.40
2.10
1.80
1.50
1.20
0.90
0.60
0.30
0.00

Distance

500.0 m



Model Boundary

DISCHARGE
169 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.22
DWN BY: RBA	CHK'D BY: RBA	Simulated Velocity Magnitude (7Q10 Ice-Covered Low Flow)	amec

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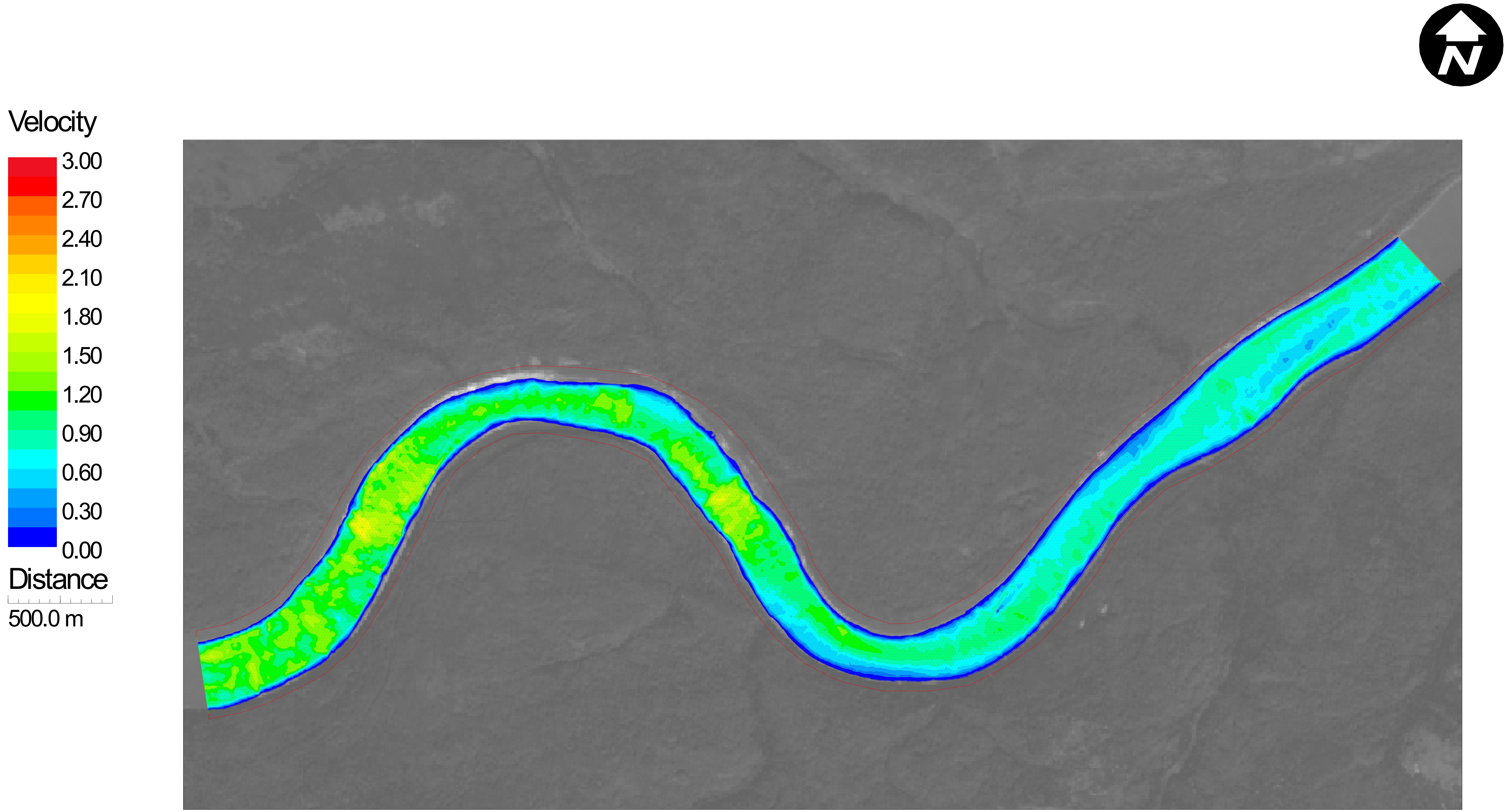

Model Boundary

DISCHARGE
439 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.	SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305		FIGURE 5.23
DWN BY: RBA / CHK'D BY: RBA	Simulated Velocity Magnitude (Mean Annual Discharge)	amec

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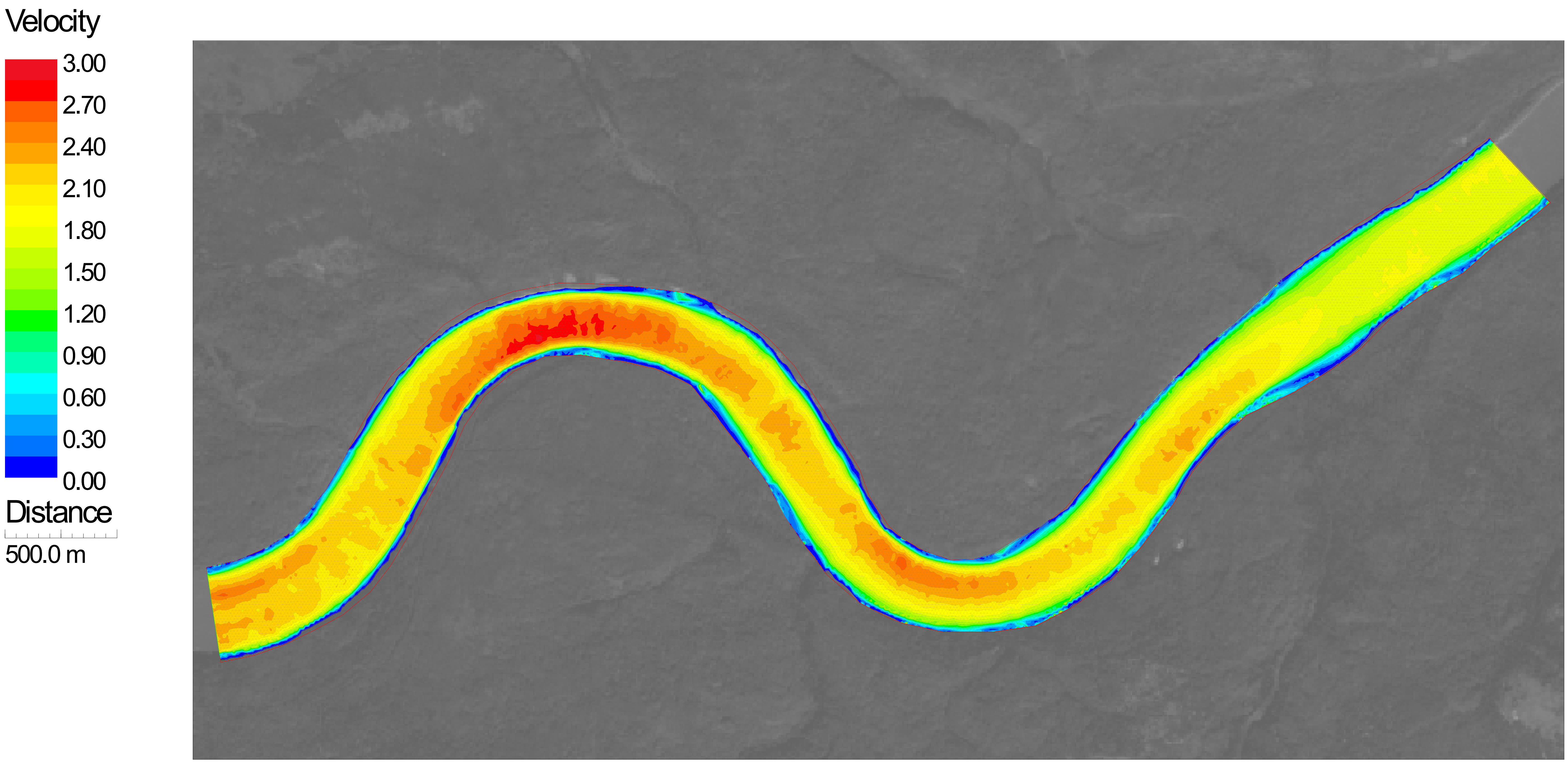

Model Boundary

DISCHARGE
4770 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.24
DWN BY: RBA	CHK'D BY: RBA	Simulated Velocity Magnitude (1:100 Year Discharge)	amec

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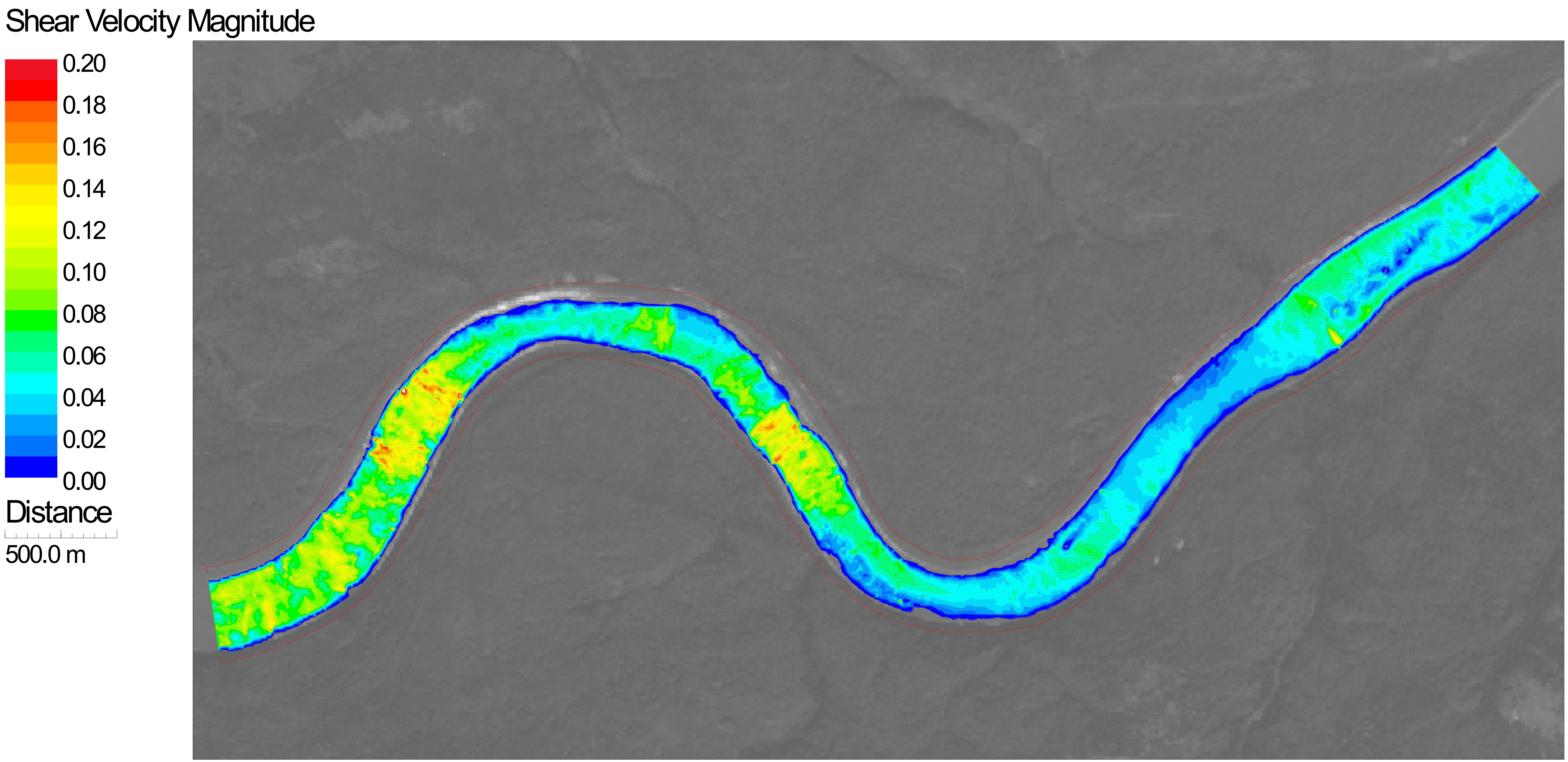

Model Boundary

DISCHARGE
188 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.25
DWN BY: RBA	CHK'D BY: RBA	Simulated Shear Velocity Magnitude (7Q10 Open-Water Low Flow)	amec

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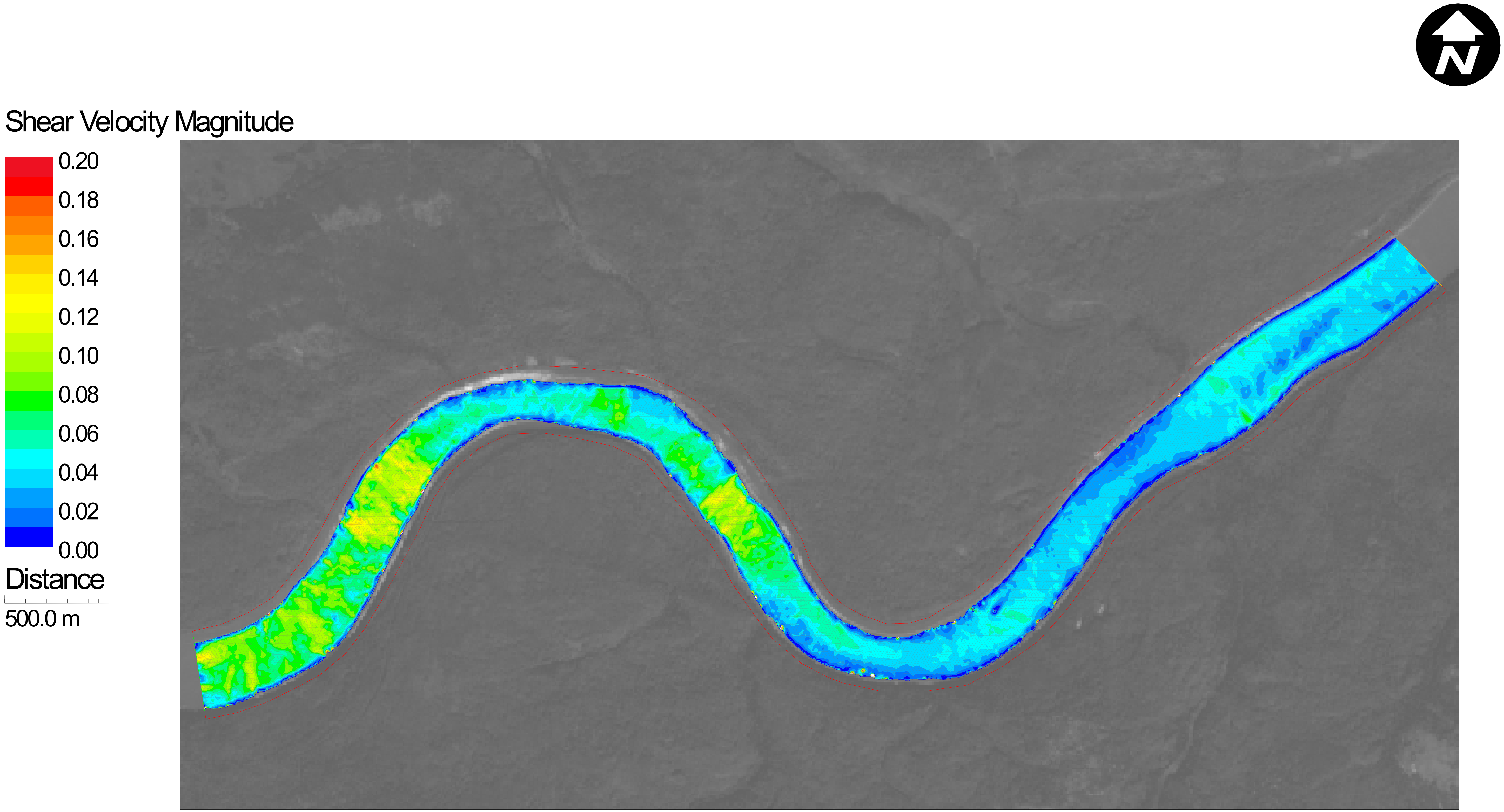

Model Boundary

DISCHARGE
169 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
			FIGURE 5.26
AMEC Project No.: SX0373305		Simulated Shear Velocity Magnitude (7Q10 Ice-Covered Low Flow)	amec
DWN BY: RBA	CHK'D BY: RBA		

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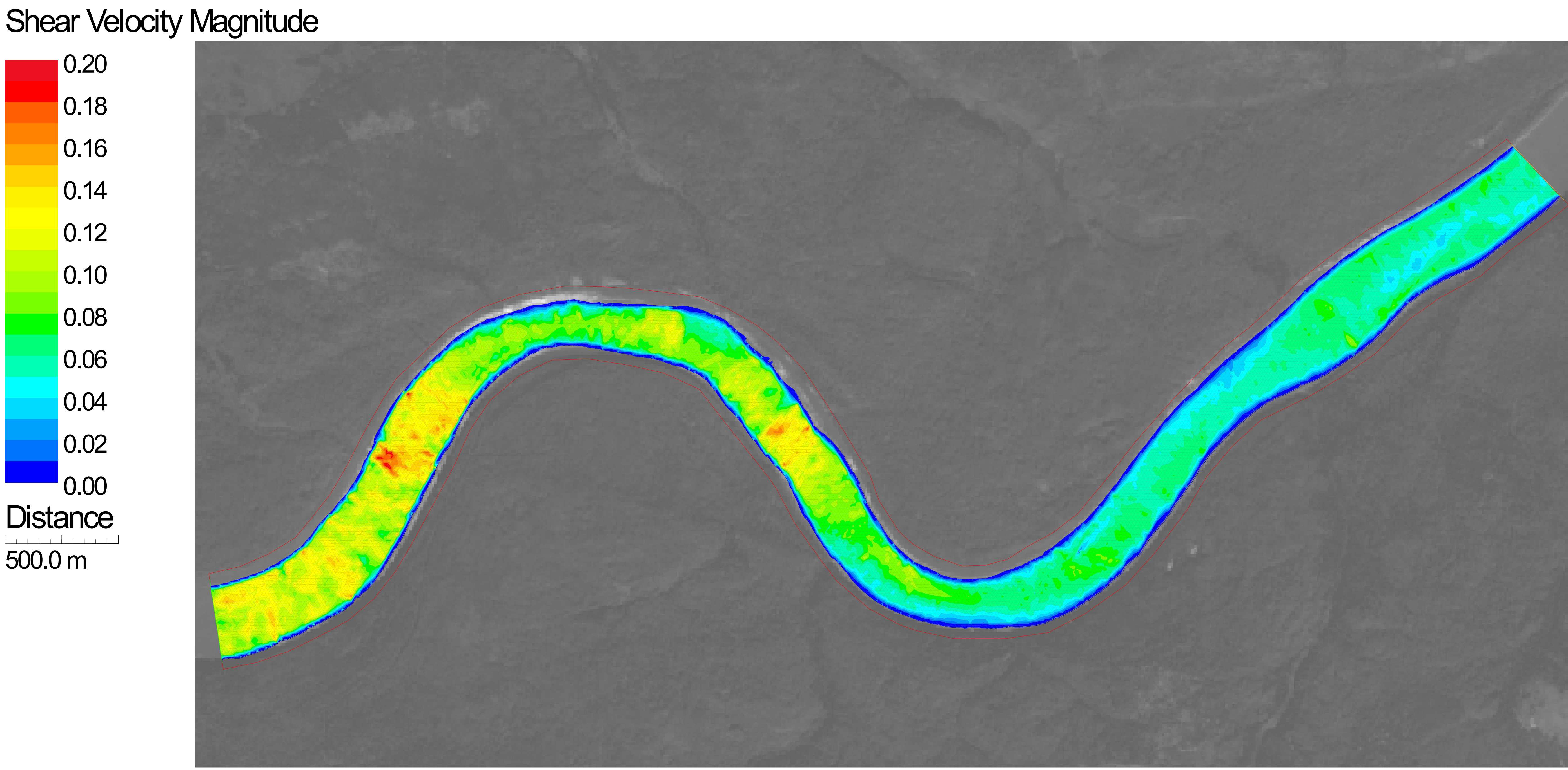

Model Boundary

DISCHARGE
439 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.27
DWN BY: RBA	CHK'D BY: RBA	Simulated Shear Velocity Magnitude (Mean Annual Discharge)	amec

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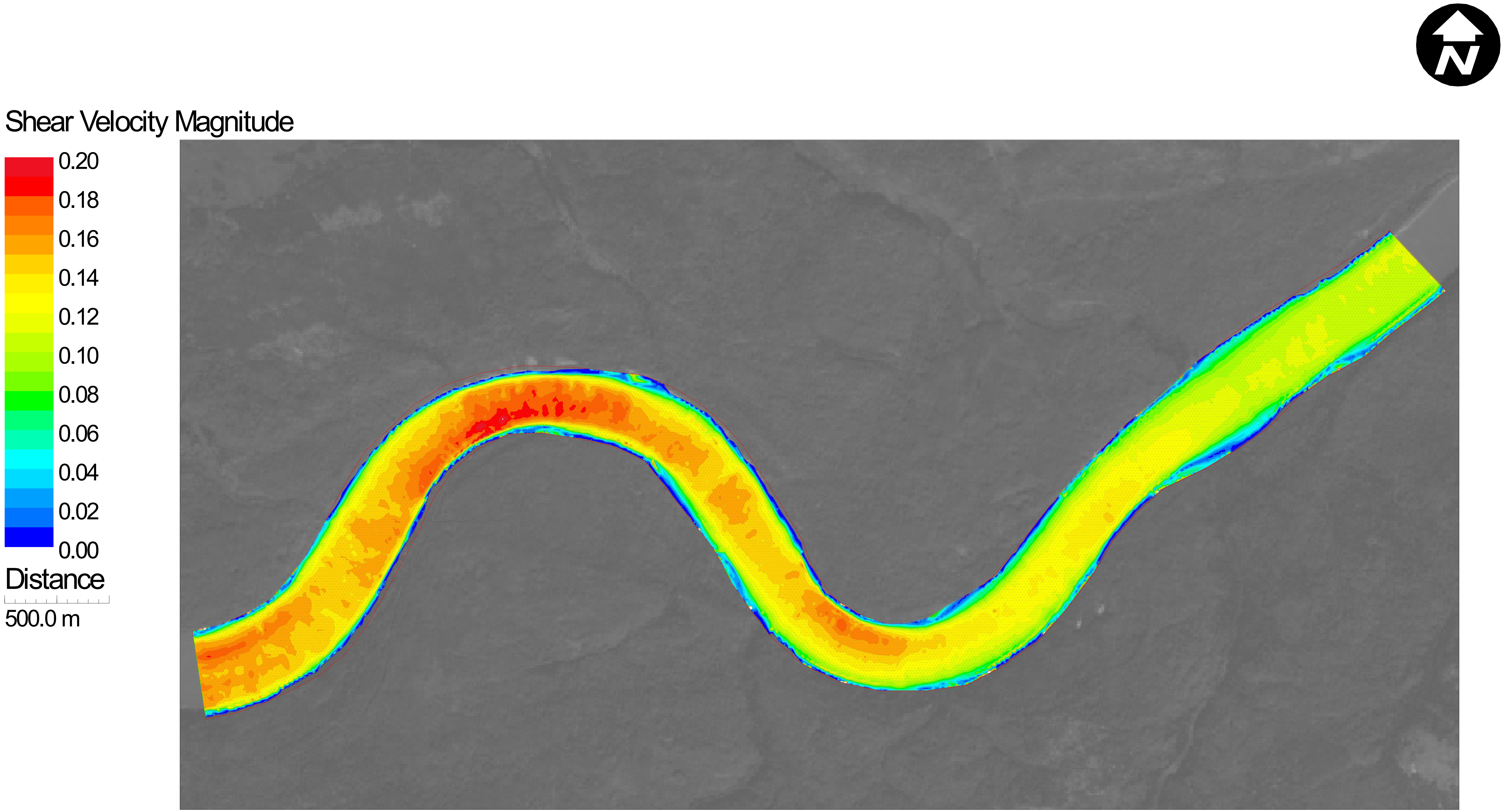


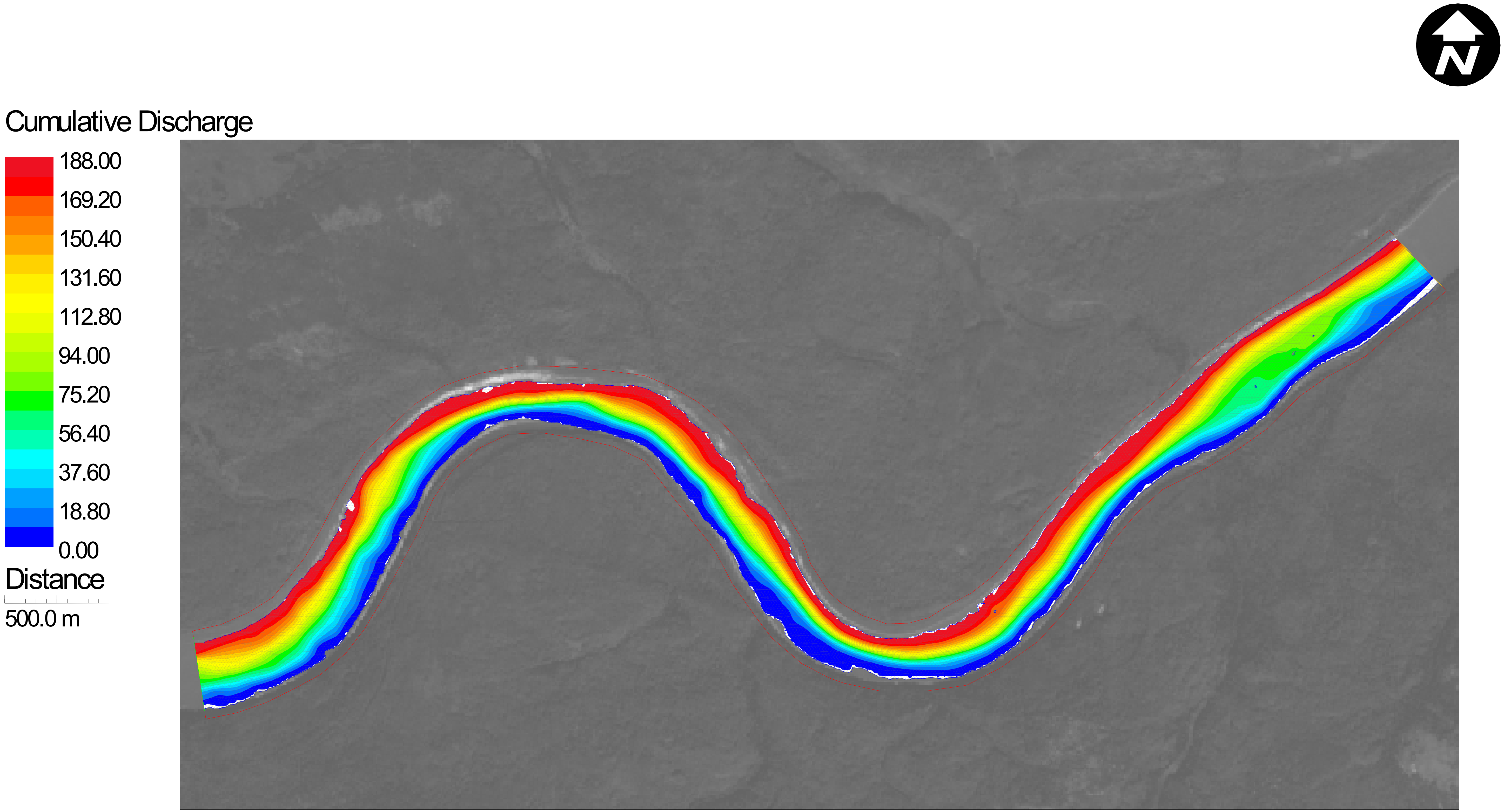
Model Boundary

DISCHARGE
4770 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.28
DWN BY: RBA	CHK'D BY: RBA	Simulated Shear Velocity Magnitude (1:100 Year Discharge)	amec



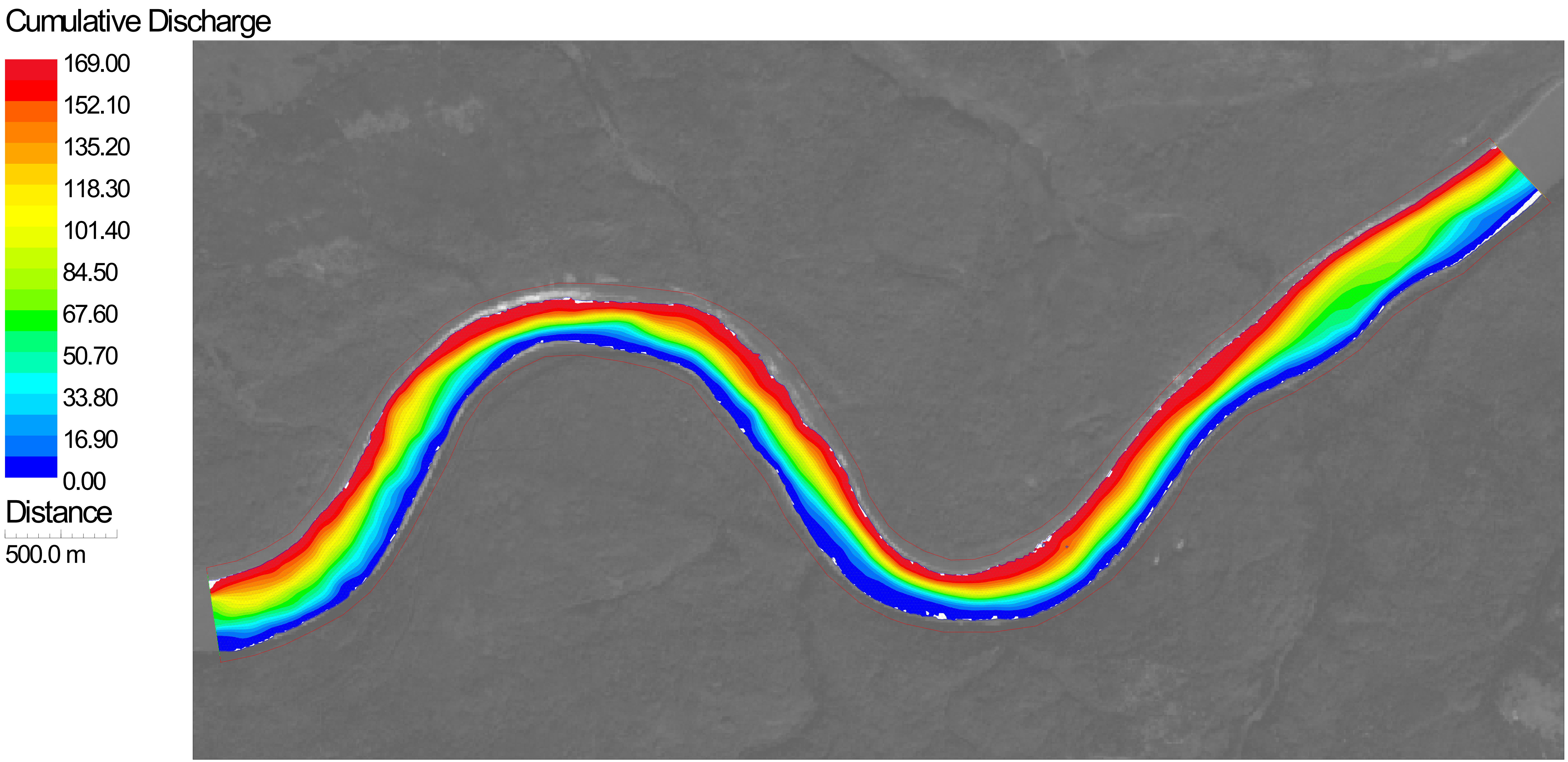
Model Boundary

DISCHARGE
188 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.29
DWN BY: RBA	CHK'D BY: RBA	Simulated Cumulative Discharge (7Q10 Open-Water Low Flow)	amec



Model Boundary

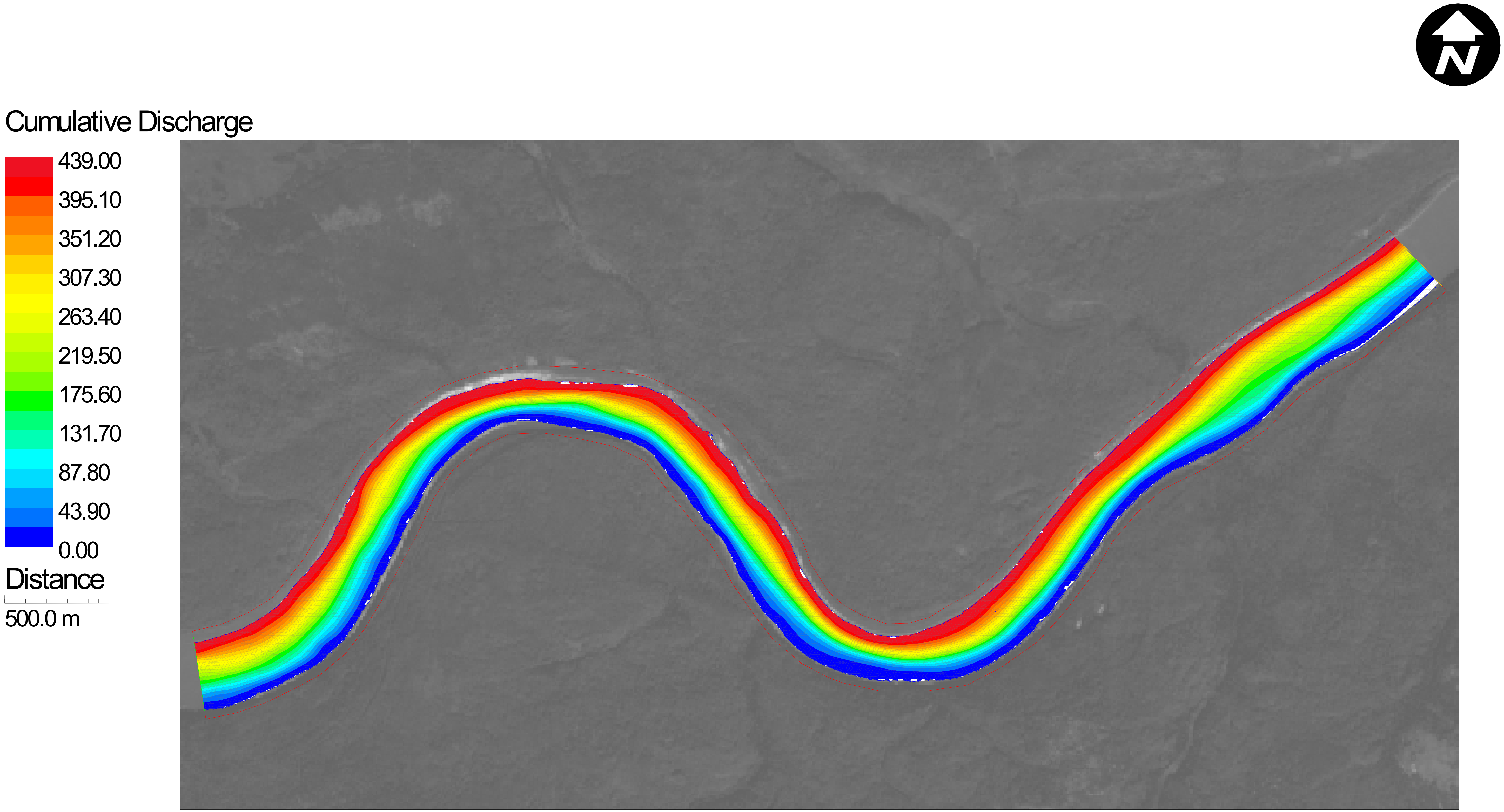
DISCHARGE
169 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305			FIGURE 5.30
DWN BY: RBA	CHK'D BY: RBA	Simulated Cumulative Discharge (7Q10 Ice-Covered Low Flow)	amec

Map Path: L:\Water\PROJECT\EW...



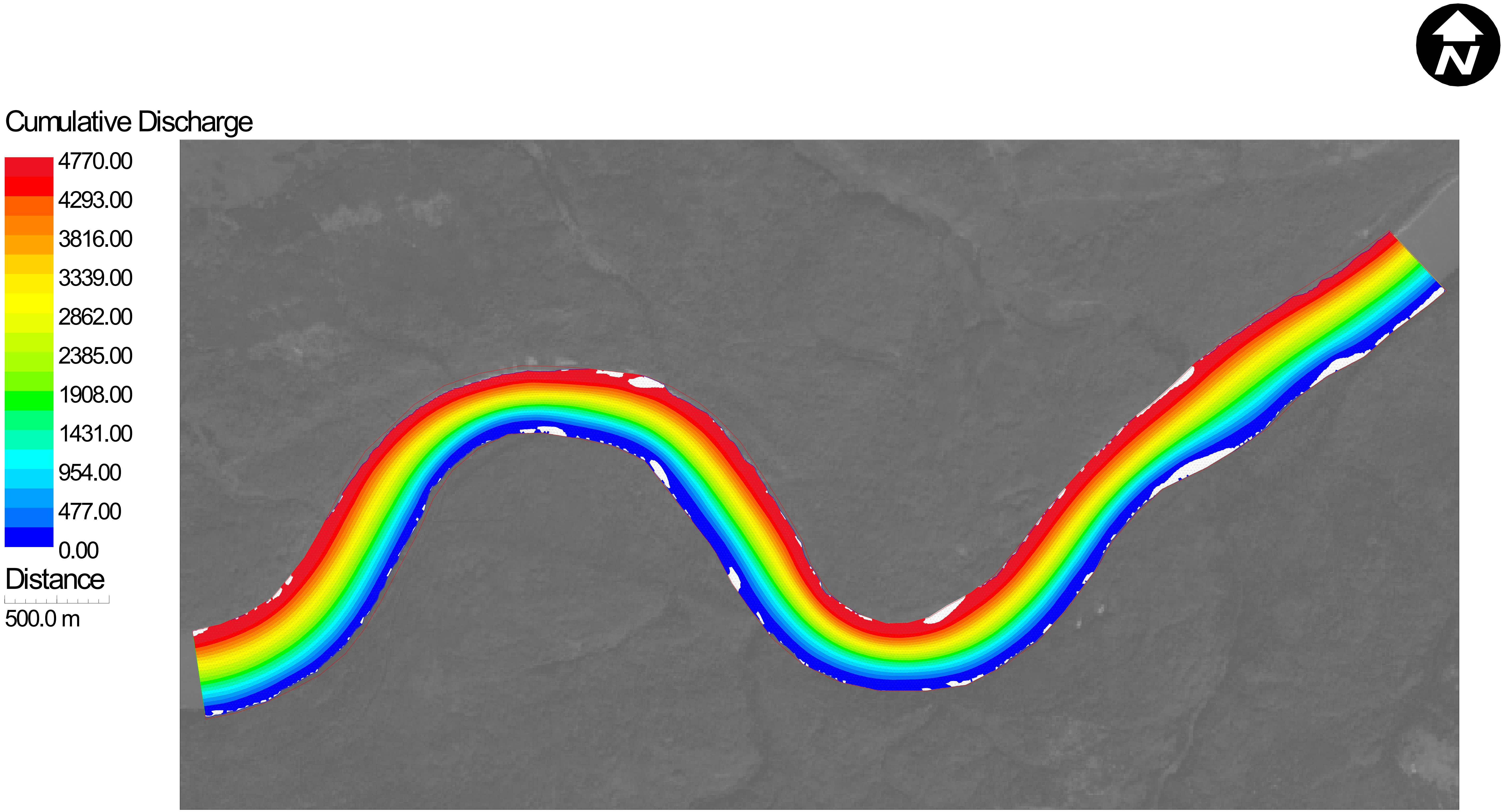
Model Boundary

DISCHARGE
439 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305		Simulated Cumulative Discharge (Mean Annual Discharge)	FIGURE 5.31
DWN BY: RBA	CHK'D BY: RBA		amec



Model Boundary

DISCHARGE
4770 m^3/s

Background Data from GeoBase®

SCALE AS SHOWN

CLIENT: Shore Gold Inc.		SHORE GOLD RIVER SURVEY	November 2010
AMEC Project No.: SX0373305		Simulated Cumulative Discharge (1:100 Year Discharge)	FIGURE 5.32
DWN BY: RBA	CHK'D BY: RBA		amec

Map Path: L:\Water\PROJECT\EW...

6.0 SEDIMENT CHARACTERISTICS

6.1 Introduction

It is necessary to characterize the size of bed sediment as its movement can create bed forms such as ripples and dunes on the stream bed that have varying degrees of impact on a river outfall or diffuser. Archived sediment data was retrieved for several WSC stations along the North Saskatchewan, South Saskatchewan, and Saskatchewan Rivers near the proposed site. Bed material data obtained from each of the station records were analysed to estimate the bed sediment characteristics at the proposed site. This information was subsequently used to determine the estimated dimensions of bed forms that may develop as a result of hydraulic action on the river bed material.

6.2 Methodology

Bed material data are available at the WSC hydrometric gauge locations summarized in **Table 6.1**.

Table 6.1
List of Water Survey of Canada Sediment Gauges Near the Study Site

Station	Name	Period of Record	Location Relative to Study Site	Number of Samples
05GG001	North Saskatchewan River at Prince Albert	1958 to 1995	Upstream	331
05HG001	South Saskatchewan River at Saskatoon	1961 to 1995	Upstream	26
05KD005	Saskatchewan River above Sipanok Channel	1954 to 1956	Downstream	2

Since no representative data are available in the proximity of the proposed site, bed material data from all three of the sites listed in **Table 6.1** were examined and the data gathered from the two upstream stations were compared with the data for the downstream station. The average value of all of the measurements available at each site was computed and assumed to be representative of the bed material at that station.

6.3 Bed Material Data

The average values of the bed material measurements available at each of the three WSC sites are shown graphically on a semi-log plot in **Figure 6.1**. This figure indicates that the data from the three stations are very consistent. The bed material is predominantly sand and contains less than 10% gravel and less than 20% silt. Estimated values of median grain size (D_{50}) for the three sites range from 0.24 mm to 0.32 mm.

While the available bed material data indicate that the bed material is predominantly sand, it should be noted that coarse fractions have been measured along the North Saskatchewan River at Prince Albert (05GG001). During the field program discussed in Section 3.0, most of the shoreline was submerged and the channel bed was not visible due to high amounts of suspended sediment in the water. As a result, a visual assessment of the bed material composition was not possible. Photographs supplied by CanNorth (see **Figure 6.2**) from May 2008 indicate that there is some coarser material present on the bed and banks. It is uncertain whether that is widespread and representative of the channel bed. Since there have not been any bed material samples taken at the study site, it is recommended that confirmation of bed material gradation be conducted at time of detailed design so that the bed form analysis can be refined. For the purpose of this analysis, sand with a D_{50} of 0.28 mm was taken as the representative bed material.

6.4 Analysis of Bed Forms

Bed forms are depositional features created by the movement of bed materials due to the flow of a river. In a lower-flow regime, bed form types include ripples, dunes and washed-out dunes, depending on the flow conditions. This regime, which is initiated with the beginning of motion, has a large resistance to flow and a small capacity to transport sediment. In the area of transition from a low- to a high-flow regime, a plane bed is formed. In an upper-flow regime, bed form types include anti-dunes, as well as chutes and pools, depending on the flow. The upper-flow regime is characteristic of a small resistance to flow and large amounts of sediment transport (Simons and Senturek, 1992). It important to predict the type and size of bed form that may be produced since bed forms may affect the performance of an in-river structure. If the effluent discharge does not exit the structure with enough energy to move through the bed form, deposition may cause blockage of the outfall and require constant maintenance. Outfall or diffuser design must therefore take into consideration the size of bed forms that may be created so that they can be constructed above the crest of any likely bed forms.

Predictions of the type of bed forms that may be created as a result of hydraulic action on the river channel perimeter were made through graphical analysis of several important variables. This empirical approach is commonly used as there is an absence of universally acceptable analytical solutions for such predictions. The variables which have been determined to affect the creation of bed forms in alluvial channels require the knowledge of local water depth, velocity, bed shear velocity, slope of the water surface, cross-section area, and top width. These values were obtained from the *River2D* model in the vicinity of the proposed diffuser for three different discharges: the mean annual flow (439 m^3/s), the calibration flow (1168 m^3/s) and the 1:100-year flood (4770 m^3/s, as determined through flood frequency analysis). Low-flow conditions were not considered to be important for bed formation processes. After determining the type of bed form that could be expected, the sizes of the bed form were estimated using a set of empirical relationships.

6.4.1 Prediction of Bed Form Type

There are several variables which have been found to affect the creation of bed forms in alluvial channels. The variables calculated from information received from River2D for the proposed effluent discharge location include stream power, Froude number, shear intensity factor, dimensionless particle parameter, particle settling velocity, shear velocity Reynolds number, transport stage parameter, grain Froude number, and mobility parameter. Using the data obtained from the model, the parameters described above were calculated and then plotted onto the relevant graphs allowing predictions of bed form types to be made. The details of this analysis are contained in **Appendix C**. **Table 6.2** summarizes the predictions of all of the relationships examined.

Table 6.2
Results of Predictions of Bed Form Type Based on Flow Scenario

Variables Used	Flow Scenario/Discharge (m^3/s)			Applicability
	Mean Annual	Calibration	1:100-Year	
	439	1168	4770	
Stream Power; Mean Particle Diameter	upper regime	upper regime	upper regime	Relationship not proven for large rivers.
Froude Number; Hydraulic Depth	lower regime	lower regime	off the chart	Use in combination with relationship below.
Slope; Shear Intensity Factor	transition	transition	transition	Use in combination with relationship above.
Ratio of Bed Shear Velocity to Settling Velocity; Shear Velocity Reynolds Number	plane bed/ anti-dunes	plane bed/ anti-dunes	n/a	Based on data for flow depth up to 3 m.
Dimensionless Particle Diameter; Shear Velocity Reynolds Number	flat bed	anti-dunes	anti-dunes	Based on data from alpine rivers.
Froude Number; Ratio of Depth to Mean Particle Diameter	dunes	dunes	n/a	Based on data for flow depth up to 3 m.
Transport Stage Parameter; Mean Particle Diameter	plane bed / anti-dunes	plane bed/ anti-dunes	plane bed/ anti-dunes	Relationship not proven for large rivers.
Grain Froude Number; Slope	lower regime	lower regime	upper regime	
Mobility Parameter; Dimensionless Particle Diameter	ripples	ripples	ripples	

The results demonstrate definite variability in the predictions of bed form types based on the relationship used. For example, using the Froude number versus ratio of depth to mean particle diameter relationship, it is predicted that the resulting bed forms are dunes. However, this relationship is limited as it is based on data with a flow depth of up to 3 m. The plot of transport stage parameter versus mean particle diameter results in a plane bed/anti-dunes prediction; however, this relationship has not been proven for large rivers. Since these inconsistencies make a conclusive analysis difficult, the Froude number, which is an indicator of the flow regime, was used to draw conclusions about the bed form type. In the study area, this value is low (in the order of 0.2) for the range of flow conditions analysed. Therefore it is anticipated that a lower regime dominates. Since many of the other investigated parameters plotted in the higher regions on their corresponding charts, it is concluded that dunes would be the most likely bed form to appear. With the prediction of the bed type, the height and length of dunes were estimated.

6.4.2 Prediction of Bed Form Size

Several equations exist in literature that aid in predicting the size of dunes on a sand river bed. Using hydraulic characteristics obtained from the model, these equations were used to make predictions of bed form sizes at the calibration and 1:100-year flow conditions. The results of each equation were analysed and the average value of each of the equations was chosen as representative of the dune height and length. The details of this analysis are contained in **Appendix D**. A comparison was also made with observations from the field at calibration flow conditions.

Table 6.3 presents a summary of all predicted dune heights and dune lengths resulting from each proposed equation. The results in this table indicate that there is some variation in each of the sets of results.

Table 6.3
Summary of Dune Height and Length for Each Proposed Method

Method	Flow Scenario	Height (m)	Wavelength (m)
Allen (1963)	Calibration	0.36	7.20
	1:100-year	0.88	24.08
Yalin (1964)	Calibration	0.56	16.75
	1:100-year	1.17	35.00
Goswami (1967)	Calibration	0.34	7.98
	1:100-year	0.70	18.61
Julien and Klaassen (1995)	Calibration	0.50	20.94
	1:100-year	0.84	43.75
Karim (1999)	Calibration	0.81	20.94
	1:100-year	1.73	43.75

Table 6.4 presents the estimated dune height and wave length, based on the average value of the calculations used above. In order for the outfall to maintain satisfactory operation, it is advised that it be placed at a height above any possible dune formation; i.e., above the estimated dune crest elevation obtained by adding half the estimated dune height to the mean bed elevation.

Table 6.4
Predicted Sizes (height and length) of Dune Formations Based on Flow Scenario

	Dune Size	
Flow Scenario	**Height (m)**	**Wave Length (m)**
1:100-year (4770 m^3/s)	1.0	33.0
Calibration (1168 m^3/s)	0.5	14.8

Several straight longitudinal paths of surveyed depth sounding data were plotted to see if any bed forms could be identified; however, the data was inconclusive in terms of any well defined bed forms. A site survey during low-flow conditions would be required to more accurately assess bed form type and size.

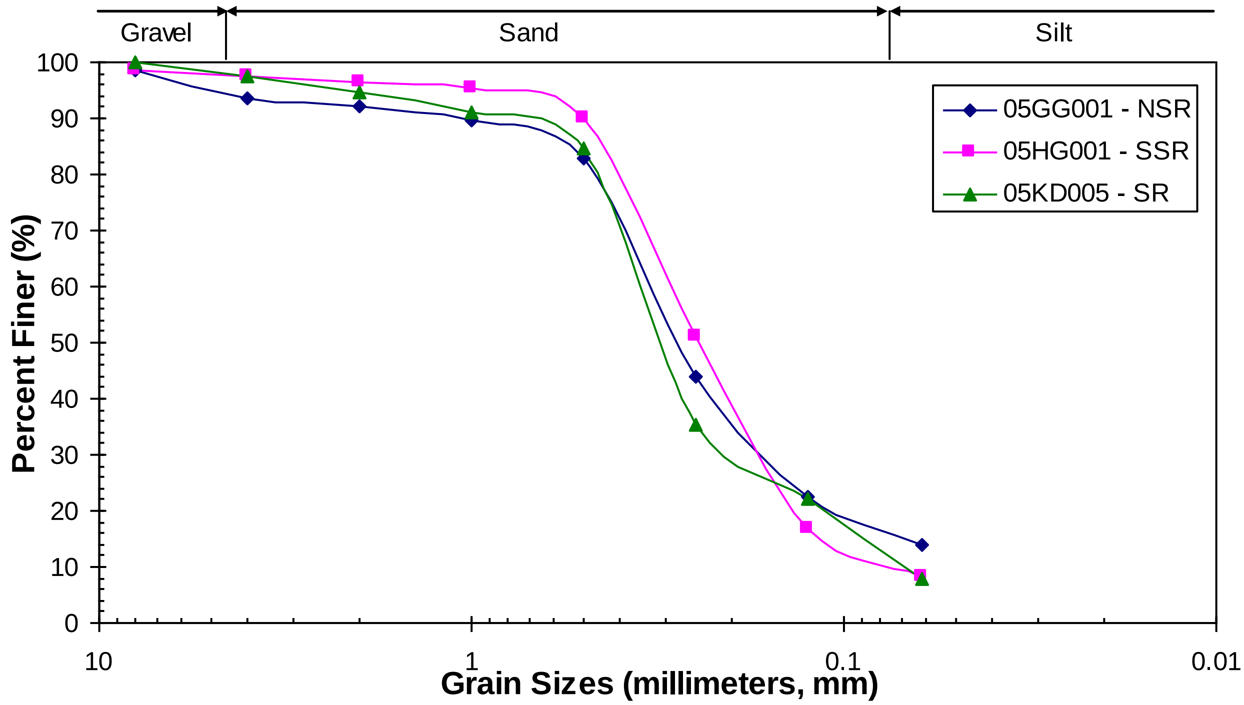


Figure 6.1 Comparison of Average Bed Material Size Distribution for the Available Data

Figure 6.2 Photograph of the Study Site Showing Bed Material, May 2008 (provided by CanNorth)

7.0 ICE JAM CONSIDERATIONS

7.1 Introduction

Ice action is an important consideration for all types of river engineering structures in Canada. The Saskatchewan River at the study site is expected to be affected by ice conditions for approximately 5 months of every year. Under many circumstances, a solid and intact winter ice-cover forms on rivers that increases hydraulic resistance to flow and raises water levels. This scenario generally does not create a problem for a river outfall or diffuser that is properly designed. However, some sites can be prone to ice jam formation either during freeze-up (in the fall or early winter), break-up (in the spring), or both. An ice jam is a highly dynamic, unstable event where ice becomes broken into many pieces that accumulate into a much thicker, rougher ice-cover. The formation and release of ice jams can exert extremely high forces capable of damaging or destroying structures in or along rivers and result in flow velocities, discharges, and water levels that can far exceed typical design flood values.

The frequency and severity of ice jams is difficult to predict and highly site-specific. Unfortunately, there are no observations or documentation of ice jam activity within the study area. Nonetheless, the relative impacts of ice jam occurrence should be addressed as they pertain to the proposed effluent discharge structure to guide subsequent phases of the design and to provide a basis for recommending an appropriate river ice monitoring program.

Through two-dimensional modeling, both an open water and ice-covered low-flow scenario was analysed in detail to quantify the effect of a solid, intact ice-cover on flow depths and velocities throughout the study reach (Section 5.0). To further meet the objectives outlined above, estimates of velocities and depths at the proposed effluent discharge location under plausible ice jam conditions were considered. This was accomplished using the U.S. Army Corps of Engineers' Hydrologic Engineering Center River Analysis System (HEC-RAS), a one-dimensional hydrodynamic model. In addition to evaluating the potential impacts of a river ice jam, the likelihood of a dynamic break-up mechanism, with the associated potential for bed and bank scour causing damage to the proposed outfall structure, was qualitatively evaluated.

7.2 Ice Jam Model

HEC-RAS model geometry is input in the form of natural channel cross sections with specified spacing. Using the model geometry from the *River2D* bed file, cross sections were extracted every 200 m throughout the 7.5 km study reach. Interpolated sections, spaced 25 m apart, were created between these sections in order to facilitate one-dimensional ice jam modeling.

The same channel roughness heights were used in the HEC-RAS model as were used in the *River2D* model. That is, a value of k_s of 0.25 m was used for the upstream half of the reach and a k_s value of 0.15 m was used for the downstream part of the reach. The model was run with the same conditions used to calibrate the 2-D model: a known downstream boundary condition and an inflow discharge of 1168 m^3/s. In order to ensure that the 1-D model produced the same results as the 2-D model, the corresponding water surface profiles were plotted and compared. **Figure 7.1** demonstrates that the results of the two hydraulic models are the same.

For the case of the continuous intact ice-covered conditions, a normal depth downstream boundary condition with a slope of 0.13 m/km was found to be suitable. The water surface elevation of the ice-covered low-flow scenario (inflow discharge of 169 m^3/s and 0.7 m continuous ice thickness) is compared with that of the 2-D ice-covered simulation in **Figure 7.2**. From this figure it can be seen that both models produce equivalent results. This downstream boundary condition was used for all the subsequent ice jam model simulations.

7.3 Simulation Results

Ice jam simulations were conducted using three different flow scenarios. These include the ice-covered low-flow scenario of 169 m^3/s; the 50% equalled or exceeded discharge value of 450 m^3/s for the month of April; and, the 10% equalled or exceeded discharge value of 750 m^3/s for the month of April (as determined through monthly flow duration curves – **Appendix B**). These values were chosen to represent a range of plausible conditions experienced at the time of spring river ice break-up. A pseudo-steady equilibrium jam formulation was used in the analysis. For each scenario, an ice jam was assumed to cover the entire reach, with the toe of the jam located near the downstream boundary and the head of the jam near the upstream boundary. Typical parameters of ice jam roughness, friction angle, porosity, stress ratio and maximum velocity were evaluated and applied to the model. The default values and the range of values used are included in **Table 7.1**.

Table 7.1
Default and Applied Values of Typical Ice Jam Parameters

Ice Jam Parameter	Default Value	Value(s) Applied
Ice Jam Roughness	n/a	0.06
Friction Angle (°)	45	45 to 55
Porosity	0.4	0.4
Stress Ratio	0.33	0.33
Maximum Velocity (m/s)	1.5	1.2 to 1.5

Using the details described above, an assessment of the velocity and flow depth at the proposed outfall site was conducted for each flow scenario. **Figures 7.3 through 7.5** show the modeled ice jam profiles and the water surface elevation under each flow scenario.

7.3.1 Velocity

The mean velocity of water at the proposed diffuser site was considered for each given flow scenario. Mean velocity was lowest under continuous ice-covered low-flow conditions of 169 m^3/s and the highest for an ice jam occurring at low-flow conditions of 169 m^3/s. Each of the velocities simulated for an ice-covered condition are lower than those for the 1:100-year flood of 4770 m^3/s, thus the 1:100-year flood event would appear to be the most critical for detailed design when considering velocity. However, it is important to note that ice jams are by nature very dynamic and non-uniform features, so the mean channel velocity may not accurately represent the maximum local velocity near the outlet structure. A summary of velocities in the vicinity of the proposed outfall structure is presented in **Table 7.2**.

Table 7.2
Velocities at the Proposed Outfall Structure Under Various Flow and Ice Conditions

Flow Scenario (m^3/s)	Velocity (m/s)
188 (no ice cover)	0.66
169 (continuous ice cover)	0.50
169 (ice jam)	0.77
450 (ice jam)	0.71
750 (ice jam)	0.73
4770 (1:100-year flood)	2.18

7.3.2 Depth

Flow depths and ice thickness were considered for each given flow scenario along a cross-section near the proposed effluent discharge site. A summary of this analysis is presented in **Table 7.3**. Depths were the lowest for an ice jam occurring at a flow of 169 m^3/s, suggesting that a low-flow ice jam scenario may be critical for final placement of the outfall structure. The ice thickness is greatest for the case of 450 m^3/s. As noted in the previous section, ice jams are by nature very dynamic and non-uniform features, so the depth under an ice jam will vary substantially both laterally and transversely throughout its evolution and among individual ice jam events.

Table 7.3
Depths and Ice Thickness at the Proposed Outfall Structure
Under Various Flow and Ice Conditions

Flow Scenario (m^3/s)	Estimated Depth at Outfall (m)	Ice Thickness (m)
188 (no ice cover)	0.9 to 2.0	–
169 (continuous ice cover)	1.1 to 2.2	0.7
169 (ice jam)	0.7 to 1.8	2.2
450 (ice jam)	2.4 to 3.5	2.3
750 (ice jam)	3.8 to 4.9	2.0

7.3.3 Other Considerations

As can be seen from the figures displaying ice jam simulation results, there are several locations where the thickness of the ice jam is increased. These locations correspond with areas that exhibit a decrease in channel slope, a bend, or narrowing of the river. Such areas tend to have a higher potential for an ice jam to develop, but their occurrence is highly site-specific. As the proposed effluent discharge location is near both a bend and change in bed slope, the potential for bed and bank scour causing damage to the proposed outfall structure as a result of ice jams is increased. This must be considered in addition to an assessment of the velocity and depth when proceeding to final design of the effluent discharge structure.

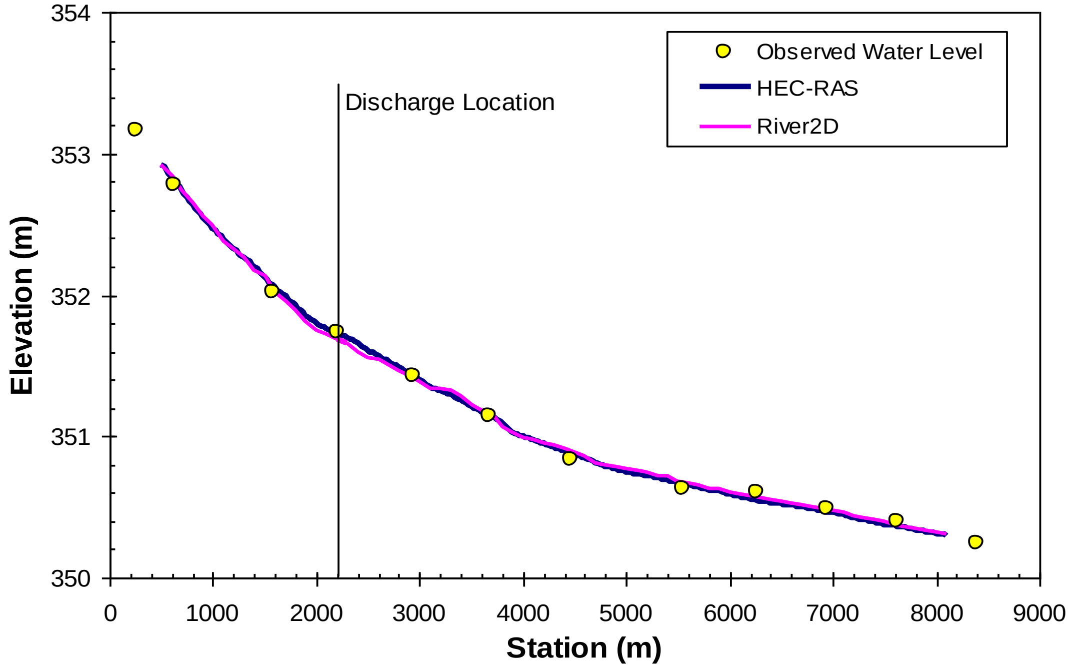


Figure 7.1 Water Surface Profile Calibration (HEC-RAS and River2D)

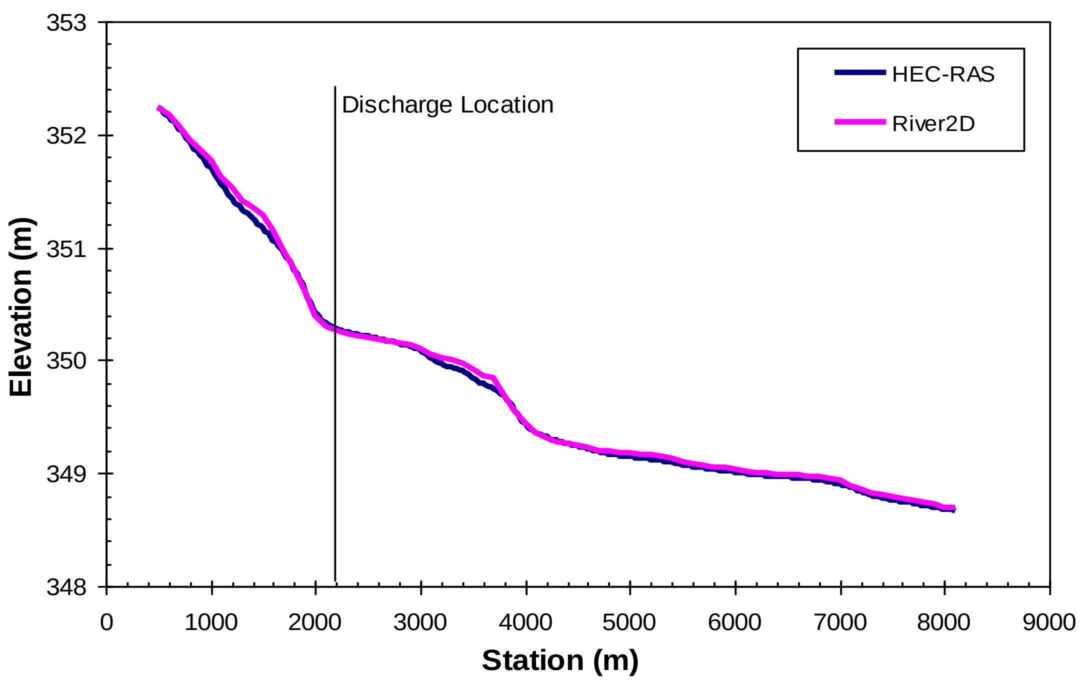


Figure 7.2 Simulated Water Surface Elevation Modeled using HEC-RAS and River2D (7Q10 Ice-covered Low Flow)

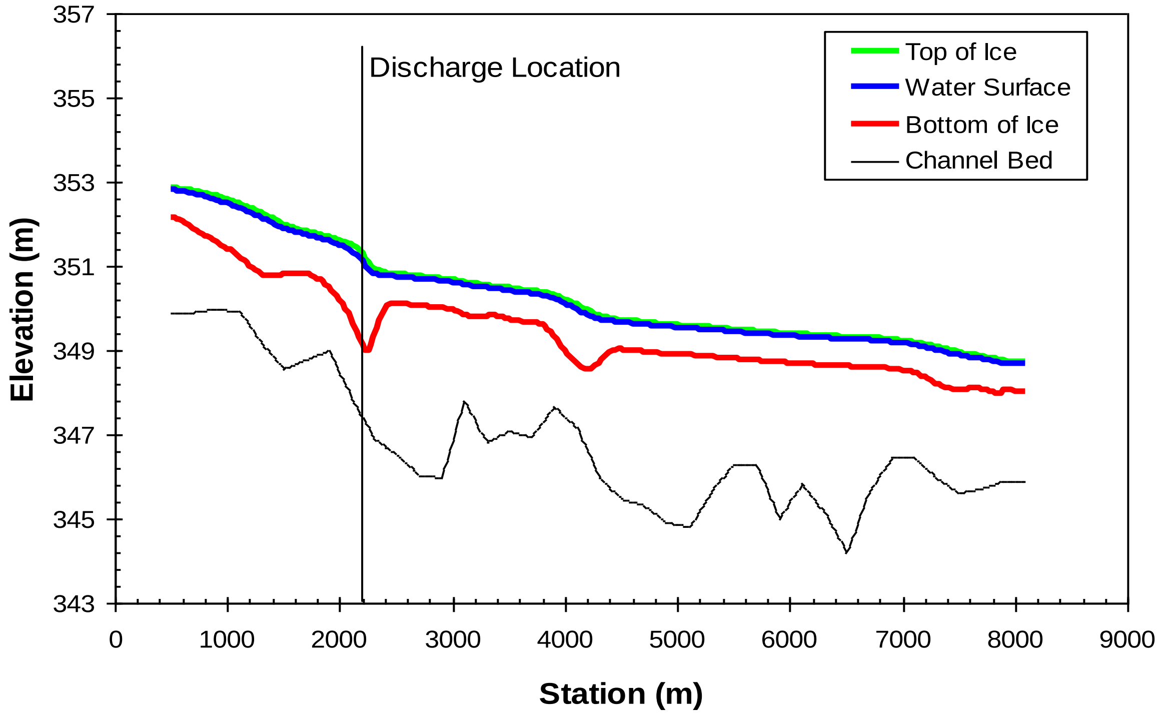


Figure 7.3 Simulated Ice Jam Profile and Water Surface Elevation (7Q10 Open Water Low Flow)

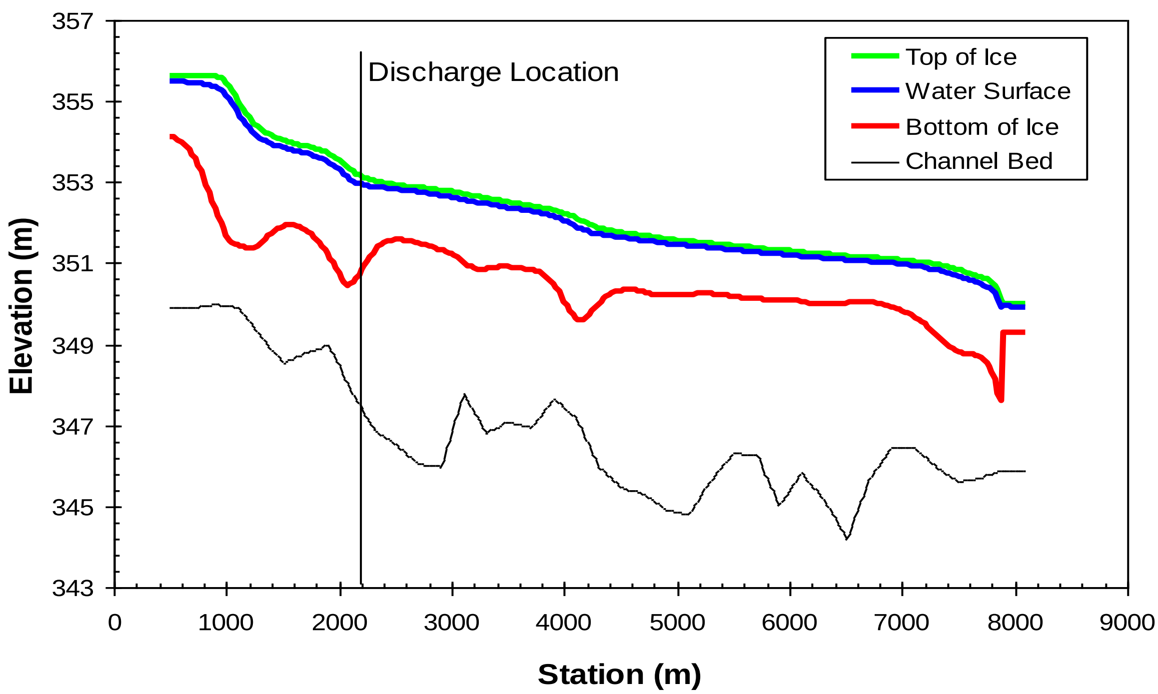


Figure 7.4 Simulated Ice Jam Profile and Water Surface Elevation (April Mean Discharge)

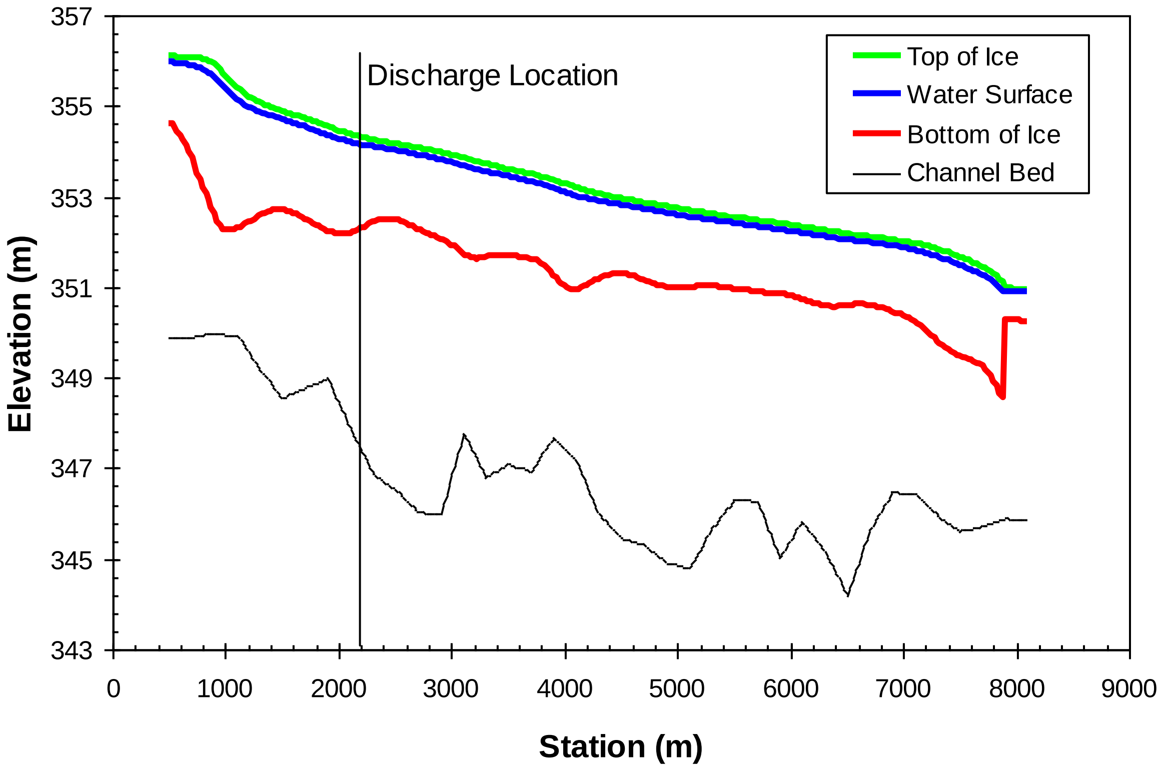


Figure 7.5 Simulated Ice Jam Profile and Water Surface Elevation (Selected April High Discharge)

8.0 SUMMARY AND RECOMMENDATIONS

AMEC Earth & Environmental (AMEC) was retained by **Shore Gold Inc.** to undertake a hydrotechnical modeling study with the goal of evaluating effluent mixing in the Saskatchewan River for the *Star Diamond Project*. This part of the report discussed details of the field program, river hydrology, river hydraulics, sediment characteristics and river ice jam considerations applicable to the design of an outfall or diffuser structure. Instream mixing and details of effluent dispersion are covered in Part 2 of this report.

In June 2010, a comprehensive field program that included measurements of bathymetry, water levels, flow velocities, and discharge were carried out on the Saskatchewan River near the proposed effluent discharge structure location. From data collected in the field program, several key observations could be made about the study reach. The 7.5 km study reach encompasses two distinct sub-reaches: a steeper upper reach, with higher velocities and shallower depths, and a wider, more mildly-sloping lower reach, with lower velocities and greater depths. Throughout the study site, the flow is relatively uniform across the channel under most conditions, and there are no islands or dominant mid-channel bars that divide the flow and promote transverse mixing. Based on the conditions experienced at the site during the time of survey, it was evident that there is some degree of backwater effect from the hydropower reservoir located downstream. A calibrated unit discharge downstream boundary condition was used to account for this in the model. It would be beneficial to survey a water surface profile at a different (lower) discharge to provide additional validation information that can be used later in detailed design.

In order to develop discharges relevant to river engineering design, a hydrological analysis was carried out. Streamflow records from the North Saskatchewan River at Prince Albert and the South Saskatchewan River at Saskatoon were used to construct a historical data series for the site. Using this data, a 7Q10 low-flow analysis was carried out for both the open water and annual case. These values were determined to be 188 m^3/s and 169 m^3/s, respectively. A flood frequency analysis was carried out in order to quantify the 1:100-year design discharge. This value, 4770 m^3/s, was used to assess higher velocities near the proposed outfall site and to predict the type and size of bed forms. From discharge measurements taken at the time of the survey, the calibration flow for modeling was determined to be 1168 m^3/s. This value was validated through hydrological analysis using the available real-time gauge data at the sites mentioned above. Flood frequency analysis also determined that these flow conditions were approximately equivalent to a 1:2-year flow.

A reliable two-dimensional hydrodynamic model was created using the data collected during the survey. This model provided essential supporting information for dispersion modeling and sediment analysis. It was also used to model various flow scenarios to assess depth and velocity design considerations for the proposed outfall structure. It was determined that the open water low-flow scenario resulted in the lowest depths at the site of the diffuser and that the high-flow scenario resulted in the highest plausible velocities that would be experienced at the proposed site over the range of flows considered. Output from the model included mapping of water surface elevation, velocity magnitude, bed shear velocity magnitude, flow depth and cumulative discharge throughout the study reach.

Based on sediment data available from Water Survey of Canada at three sites, an analysis of bed forms was conducted. It was found that due to the limited sediment data at the site and empirically-based relationships available in the literature, bed form type and size were difficult to assess. The best estimate of sediment characteristics given the available data is a sand-bed channel with a mean particle diameter of 0.28 mm. However, from photographs of the site, it can be seen that some gravels and cobbles are present along the channel banks. Assuming a sand bed channel is representative of the study site, an analysis of bed form type was inconclusive based on several different relationships. Although some of the methods examined indicated upper regime bed forms, low Froude numbers in the order of 0.2 throughout the reach support the conclusion that the most likely bed form types are dunes. An estimation of size, based on the average value of several relationships, suggests that dunes formed may be 0.5 to 1.0 m in height, depending on the flood severity considered. Direct observations and sampling of bed material during a low-flow period are recommended as part of the detailed engineering design of the effluent discharge structure following environmental approvals.

Although no ice jam activity has been documented in the study reach, the impacts of a hypothetical ice jam scenario were considered. The site of the proposed discharge structure is located near a river bend and a transition to mild slope, which is a type of area that is known to favour ice jam formation. A pseudo steady-state equilibrium ice jam was modeled throughout the entire study reach at three different flow scenarios: 169 m^3/s, 450 m^3/s and 750 m^3/s. While the model did not highlight any notable effects on water velocities experienced at the site, the smallest depths below the ice cover may be experienced if an ice jam were to occur at low-flow conditions. Simulations indicated a range of flow depths near the structure from 0.7 to 1.8 m under the ice jam conditions considered. Since ice jams are very dynamic and non-uniform by nature, it is important to note that the depth under an ice jam can vary substantially both laterally and transversely throughout its evolution and among individual ice jam events. To proactively gain a better understanding of how the river ice regime might impact the detailed design of an effluent discharge structure, ice observations should be made along the study reach in the winter and/or immediately after break-up.

9.0 CLOSURE

This report has been prepared for the exclusive use of **Shore Gold Inc**. This report is based on, and limited by, the interpretation of data, circumstances, and conditions available at the time of completion of the work as referenced throughout the report. It has been prepared in accordance with generally accepted engineering practices. No other warranty, express or implied, is made.

Respectfully yours,

AMEC Earth & Environmental

Agata Hall, M.Sc., E.I.T.
Water Resources Engineer

2 *December 2010*

Robyn Andrishak, M.Sc., P.Eng.
Senior Water Resources Engineer
Direct Tel.: (780) 377-3682
Direct Fax: (780) 435-8425
E-mail: robyn.andrishak@amec.com

Reviewed by:

Monica Wagner, M.Eng., P.Eng.
Senior Associate Engineer

RA/AH/lb

c: Ian Judd-Henrey

Permit to Practice No. P-4546

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PART 2

DISPERSION MODELING STUDY

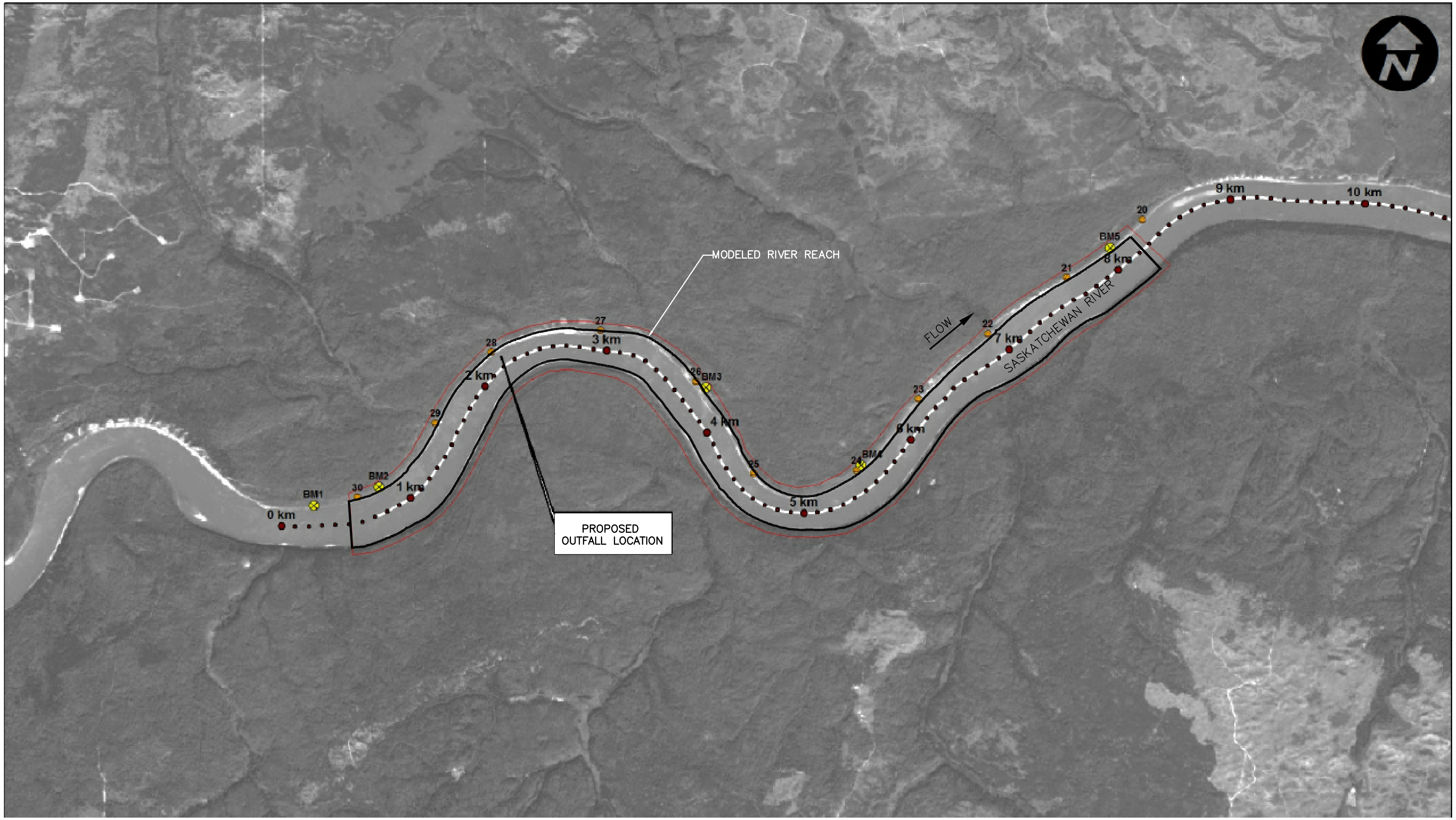
1.0 INTRODUCTION

This dispersion modeling study was undertaken by AMEC Earth & Environmental (AMEC) to assist Shore Gold Inc. with project design of a diffuser/outfall location on the Saskatchewan River and the preparation of permitting for the Star Diamond Project. This study addresses the prediction of chloride concentrations in the Saskatchewan River that are expected to result from the discharge of groundwater to the river. Groundwater will be collected from an open pit perimeter well field system, and will be conveyed to the Saskatchewan River through a pipeline. Groundwater quality analysis indicated that chloride is the primary salinity parameter of interest from drinking water and protection of aquatic life perspectives. There is no current or proposed use of the Saskatchewan River for drinking water within the bounds of the model domain; nor are there any water supply systems immediately downstream on the Saskatchewan River that currently draw drinking water from the river.

The location plan of the Saskatchewan River Outfall and the Modeled River Reach are shown in **Figure 1.1**. The proposed outfall site is located approximately 40 km downstream of the confluence of the North Saskatchewan and South Saskatchewan Rivers. The outfall location is positioned opposite the north bank of the Saskatchewan River between Duke and FALC Ravines at approximate coordinates E519546, N5897068 (NAD27). This site is approximately 600 m downstream of the originally planned discharge site. The location was selected such that sufficient depth of water above the diffuser or outfall would be available for its normal operation during the low-flow conditions.

The total length of the river reach included in the current model is approximately 8.0 km. Background chloride concentrations reported by SRK Consulting (2009) for the Saskatchewan River varied from 5 mg/L to 10 mg/L (average 7.4 mg/L) for various river flow conditions. It was assumed that the 10 mg/L value measured on 18 May 2008 is representative of the background chloride concentration for the 10-year return period, 7-day ice-covered low-flow (7Q10) condition. The average of 7 mg/L was assumed to be representative of the background chloride concentration for the average-flow condition. Saskatchewan River average annual and ice-covered 7Q10 flows at this location have been calculated at 37 930 000 m^3/d (439 m^3/s) and 14 600 000 m^3/d (169 m^3/s), respectively. Open water low flow for the river has been calculated at 16 200 000 m^3/d (188 m^3/s). The well field discharge is expected to achieve a steady-state condition discharge of approximately 199 000 m^3/d (2.3 m^3/s). Well field water salinity values have been estimated at 1725 mg/L for chloride. AMEC's understanding is that the discharge value given by Shore Gold Inc. is a conservative estimate.

The chloride concentration in the Saskatchewan River for the average annual-flow condition, after complete mixing with well field water, was calculated at 16 mg/L (or 9 mg/L above background). The maximum predicted chloride concentration in the Saskatchewan River, after complete mixing, during the extreme low-flow (7Q10) condition, was calculated at 33 mg/L (or 23 mg/L above background).



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CLIENT	PROJECT: SHORE GOLD RIVER SURVEY	08–10
AMEC PROJECT NO: SX03733	TITLE: LOCATION PLAN OF SASKATCHEWAN RIVER OUTFALL AND THE MODELED RIVER REACH	FIGURE 1.1
DWN BY: CG / CHK BY: SS		amec

The purpose of this modeling work is to establish the mixing and dispersion potentials of mine water in the Saskatchewan River downstream of the discharge location, for open water low-flow and average-flow conditions. The results of the modeling exercise will assist in the decision making process for the outfall design, and provide backup data for permitting.

Hydraulic parameters and flow characteristics of the river reach that were used to construct and calibrate the model were determined from water level and discharge measurements along the reach, together with bathymetric and ice thickness data, as well as topographic maps and aerial photographs. The data set of historical discharges at the site was estimated from recorded data combined for the hydrometric stations upstream of confluence: North Saskatchewan River at Prince Albert, South Saskatchewan River at Saskatoon and St. Louis; and downstream of confluence: Saskatchewan River at Nipawin and below Tobin Lake. Detailed descriptions of the field program, hydrology, river hydraulics and sediment characteristics of the reach are provided in the Preliminary Hydrotechnical Evaluation Report.

The following sections summarize the collation and utilization of the river data for the construction and calibration of flow and transport models for the river reach, simulation of mixing of the mine water plume, and the results of the study. <u>Unless otherwise stated, all concentrations mentioned in the report are additive to the river background total chloride concentrations described above</u>. Also the results are directly proportional to the initial concentrations and can be used to assess the concentrations of any other conservative substance dissolved in the effluent. Conservative substances are those that normally do not increase or decrease in the water column as a result of chemical reactions, biological activity, or adsorption onto suspended solids.

2.0 HYDROLOGY AND HYDRAULIC CHARACTERISTICS

A comprehensive field program was undertaken for the 8 km long study reach over the period of June 24th through June 28th, 2010. During this time, a detailed bathymetric survey, water level surveys and discharge measurements were completed. Throughout the survey, horizontal and vertical positioning was achieved using survey-grade Trimble® R6/R8 GNSS RTK-GPS receivers. The channel bed survey was conducted from a boat with an OHMEX SonarLite 2000 depth sounder and RTK-GPS attached. Bathymetric contours were generated from the bed point data using a contouring software program, extrapolated to the river banks by interpolation with the topographic data. Details of the field program are given in the Hydrotechnical Modeling Study (Part 1 of this report).

Characteristic background river flows (discharges) at the site were obtained by statistical analysis of daily discharges computed from the Water Survey of Canada (WSC) station data. Hydraulic parameters (velocity, water depths, etc.) for the outfall reach were obtained from analysis of water level and discharge measurements for the open water condition.

2.1 Saskatchewan River Discharge

Mean annual discharge and low-flow discharges were estimated based on an analysis of historical streamflow data for the Saskatchewan River which are available from the WSC division of Environment Canada stations near the proposed site listed in **Table 2.1**.

Table 2.1
Regional Water Survey Canada Hydrometric Stations

WSC Station	Name	Period of Record	Drainage Area (km^2)	Comments
05GG001	North Saskatchewan River at Prince Albert	1910 to 2009	131 000	Upstream of confluence; regulated since 1962.
05HG001	South Saskatchewan River at Saskatoon	1911 to 2009	141 000	Upstream of confluence; regulated since 1968.
05HH001	South Saskatchewan River at St. Louis	1958 to 1997	148 000	Upstream of confluence; regulated since 1968.
05KD001	Saskatchewan River at Nipawin	1945 to 1948; 1951 to 1962	287 000	Downstream of confluence; regulated since 1968.
05KD003	Saskatchewan River below Lake Tobin	1962 to 2009	289 000	Downstream of confluence; regulated since 1963.

The approach and the methodology followed for this work were described in Sections 1.2 and 1.3 of the Preliminary Hydrotechnical Evaluation Report. The mean annual discharge, design modeling value for the Saskatchewan River near the Shore Gold outfall site was determined to be 439 m^3/s, or 37 900 000 m^3/d. The one-in-10-year 7-day low-flow discharges (7Q10 low flow) were determined for both the open water condition and the annual case to be 188 m^3/s or 16 200 200 m^3/d and 169 m^3/s or 14 600 000 m^3/d, respectively.

By comparison, the design effluent discharge rate is 2.3 m^3/s or 199 000 m^3/day (assumed constant), which represents approximately 0.53% of mean annual flow, and 1.2% of the modelled open water 7Q10 flow. The modeling analysis was conducted for open water conditions as the model cannot account for ice cover effects. However, mixing under ice at the predicted lower flow value of 14 600 000 m^3/d is expected to be as good as, or better than, that achieved in the open water condition with a flow of 16 200 000 m^3/d (see Part 1, Section 5.4 – *Model Simulation Results* for comparison of depths and velocities under ice conditions).

Saskatchewan River design discharge values are summarized in **Table 2.2**.

Table 2.2
Saskatchewan River at Shore Gold Discharge Characteristics

Description	Daily Discharge		Notes
	(m^3/s)	Mm^3/day	
Mean Annual Discharge	439	37.9	
7Q10 for open water period	188	16.2	June 1 – October 31
7Q10 for annual period	169	14.6	Lowest flow of the year always occurs during winter with ice cover present

2.2 Channel Description

The river reach has two 90° bends within 4 km downstream of the proposed outfall. The radii of the first and second bends are approximately 650 and 500 m, respectively. The last 2 km of the river channel is straight. The width of the channel varies from a minimum 260 m at the proposed outfall location to a maximum of 380 m at a point 5 km downstream of the outfall. The typical side-slope of the channel is 7H:1V for the reach. The proposed outfall position is located opposite the north bank of the river channel where the side slope is approximately 8H:1V. The valley walls confine the channel. The river substrate material is predominantly comprised of sand and contains less than 10% gravel. There are occasional expressions of coarser material consisting mainly of cobbles to boulders. Relatively high roughness coefficients (n = 0.030 for the lower and 0.050 for the upper half of the reach) were selected as being representative for the river reach based on substrate and turbulent conditions at the site. The roughness coefficients were varied slightly for the low- and average-flow conditions during model calibrations.

3.0 FLOW AND TRANSPORT MODELING

Effluent plume dispersion and dilution calculations were performed with the help of a numerical flow and transport model. Estimates of chloride concentrations in the river associated with mine water discharge were determined using the model in the river reach downstream of the outfall location.

Two mixing scenarios were investigated corresponding to river flows for the 1:10-year 7-day open water low-flow (7Q10) condition, and for the annual average-flow condition. Positioning of the outfall pipe at the proposed discharge location was assumed to extend 70 m towards the middle of the river from the water edge corresponding to the average-flow condition. Model assumptions, construction, input data, calibration and results are summarized below.

3.1 Model Description

The model used in the analysis was AQUASEA (1992). AQUASEA is a two-dimensional, depth-averaged flow and transport model using a mixed (staggered) Galerkin finite element method with triangular elements. It is designed to simulate hydraulic flow in estuaries, rivers, lakes and coastal areas. The flow model is based on the solution of two-dimensional shallow water equations including bed resistance, wind stress and nonlinear convection terms. The transport model includes sources, decay, and convective and dispersive transport.

The simplifying assumptions used for the dispersion modeling of the mine water discharge at the Saskatchewan River outfall site were as follows:

- differences between the model averaged, and actual river depths, are small and will not change the flow regime and overall results obtained through the modeling;
- the bottom friction coefficient is estimated during the flow model calibration, and is based in part on field observations and other river reaches with similar morphology;
- other flow and transport parameters, such as transverse and longitudinal dispersion coefficients, were estimated from published data for similar river reaches;
- the effluent discharge is chemically conservative (chloride in solution);
- the flow in the river is uniform and steady; the effects of rapids and secondary currents on the flow regime and mixing are ignored. The reach exhibits generally consistent channel morphology within the modeled river reach;
- the river velocities govern mixing in the river; the initial mixing due to momentum of the mine water inflow is neglected. Mine water enters the river at a constant rate, discharged from the end of the outfall pipe at 70 m from the north riverbank at average flow, and is completely mixed with river water in the vertical water column;
- salinity concentrations in the plume are small enough so as not to create significant density gradients in any direction; and,
- no tributaries or streams enter or leave the study reach with the exception of the discharge from the mine water outfall.

In general, it should be noted that the assumptions used to construct the model provide an indication that the results of the dispersion modeling, and the associated chloride concentration distribution within the Saskatchewan River channel, are conservative.

Bathymetric information, based on previously collected data including field observations, was incorporated into the finite element grids constructed as shown in **Figure 3.1**. Initially all riverbed elevation data were contoured with a contouring software. The riverbed map was then completed by interpolation to integrate it with the topographic contours.

The river bottom elevations were digitized into the model elements as illustrated in **Figure 3.2**, and used for the respective flow calibration runs of high- (flow during the field program), average- and low-flow conditions.

The finite element model grid contains 12 775 triangular elements and 6 682 nodes. The element sizes vary from approximately 2 m near the outfall to 100 m further away from the source. Separate flow scenarios were analysed for the river, with and without the outfall operation, during model calibration and simulation stages.

Considering the assumption of uniform flow in the simulated river channel, and ignoring small head losses through the reach, hydraulic gradients in the model were estimated to be small (less than 0.0005). The hydraulic gradient for the upper half of the reach was estimated to be in the order of 0.0006 due to the shallower depths; and, the hydraulic gradient for the deeper lower half of the reach was estimated to be in the order of 0.00015. The hydraulic gradients were adjusted during the calibration of flow simulations and verified with the River2D model analysis results. The Manning roughness coefficient for the river reach was estimated from values listed in literature for similar channels. The river bottom friction constant (Chezy's coefficient - *C*) was calculated using Pavlovskii formula (Chow 1960):

$$C = 1.5 \frac{R^y}{n}$$

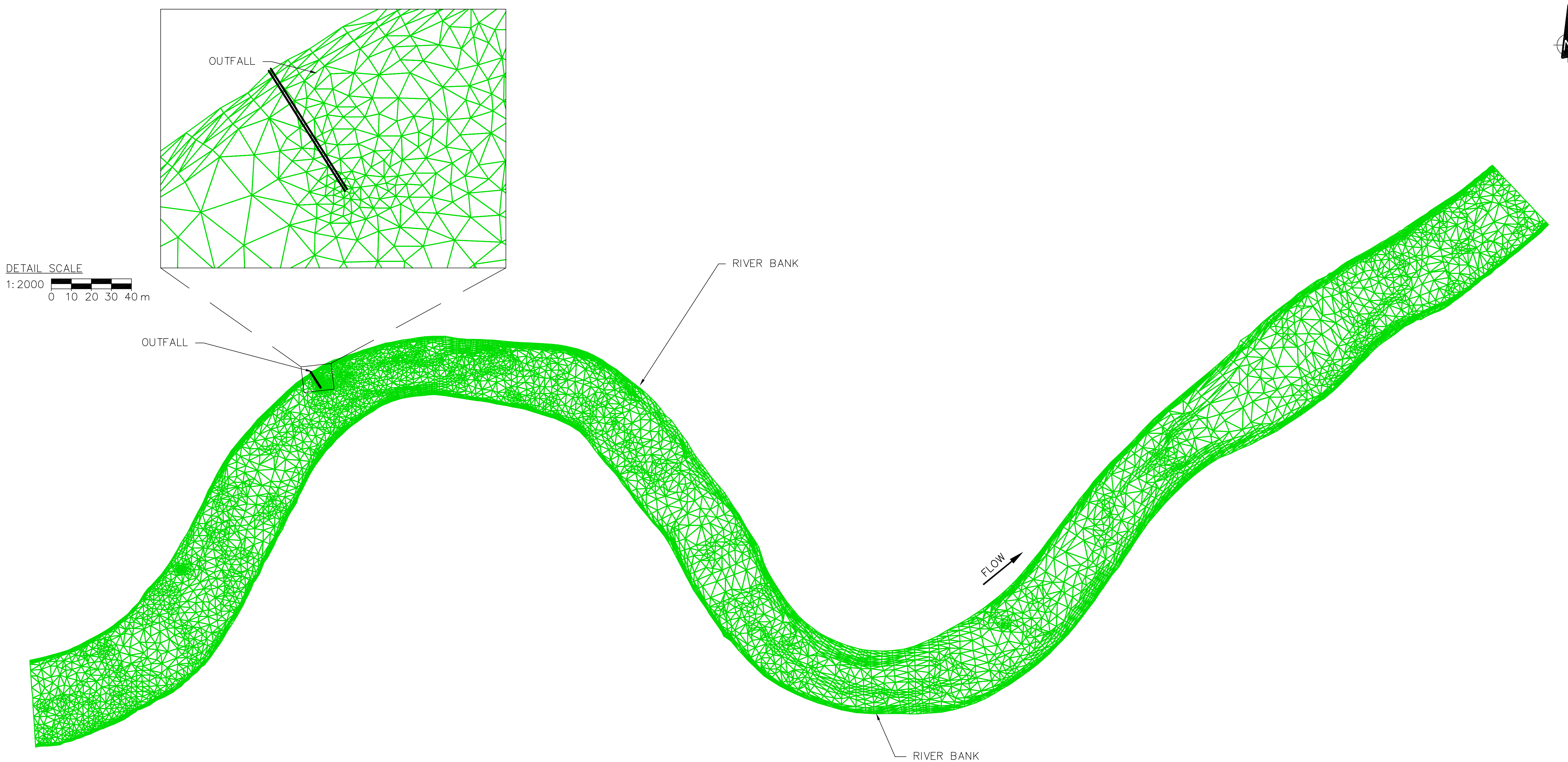
Where: n = the Manning roughness coefficient

R = the hydraulic radius calculated as a ratio of area of cross-section to the wetted perimeter

and

$$y = 1.5\sqrt{n} \qquad \text{for R<1.0 m}$$

In the model grid, flow and mass flux calculation boundaries were installed at points approximately 500 m, 4000 m and 6000 m downstream from the outfall location. Also, several time series nodes were selected at similar grid locations for flow and transport. Time series flow and concentration data accumulated at these calculation boundaries, and at the selected time series nodes, were used to check model variables at the end of each calibration and simulation run.



MAIN SCALE
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CLIENT	PROJECT: SHORE GOLD RIVER SURVEY	08–10
		FIGURE 3.1
AMEC PROJECT NO: SX03733	TITLE: FINE ELEMENT GRID AND MODEL BOUNDARIES	amec
DWN BY: CG / CHK BY: SS		

*PLOT 1:1=B (L)

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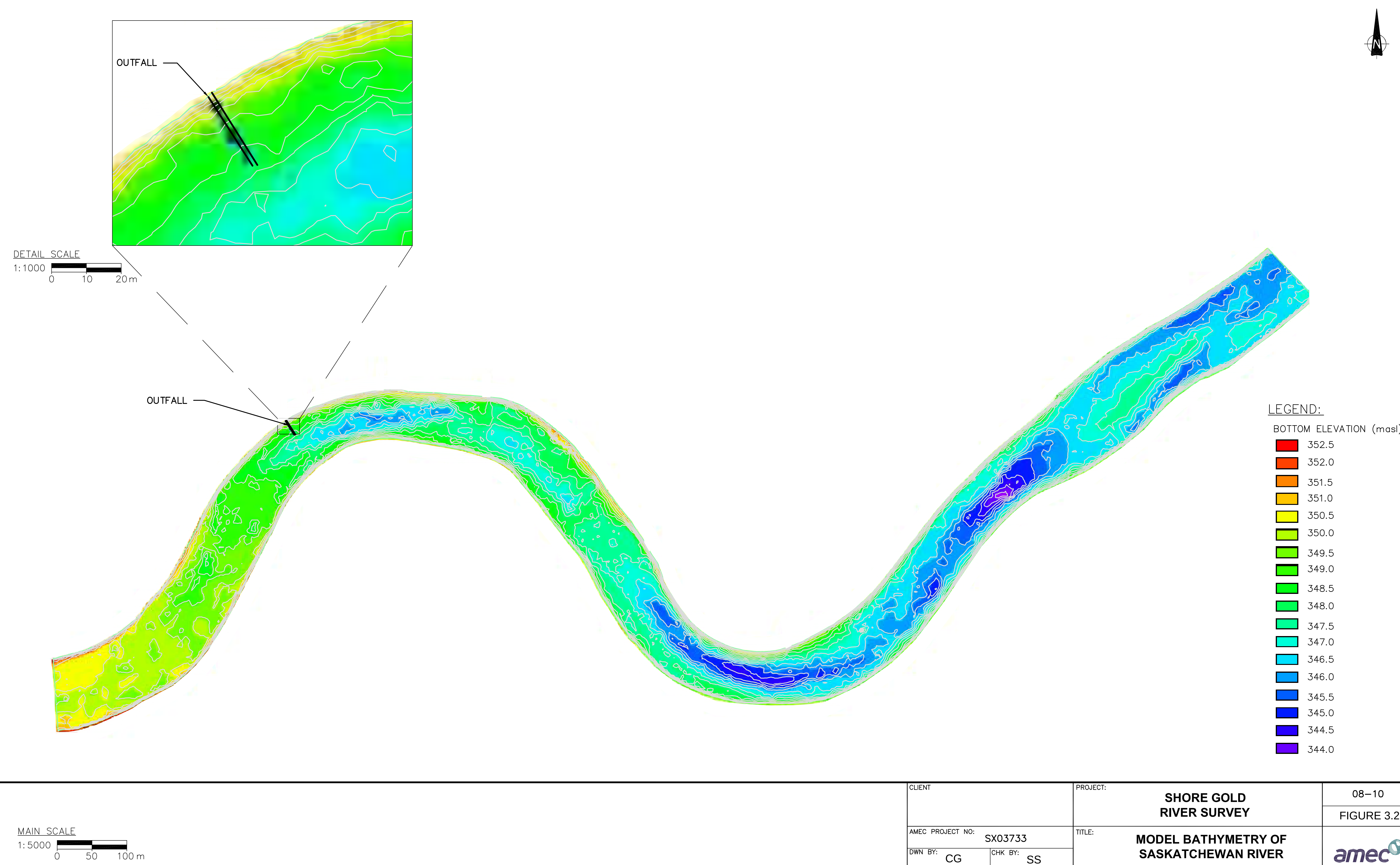

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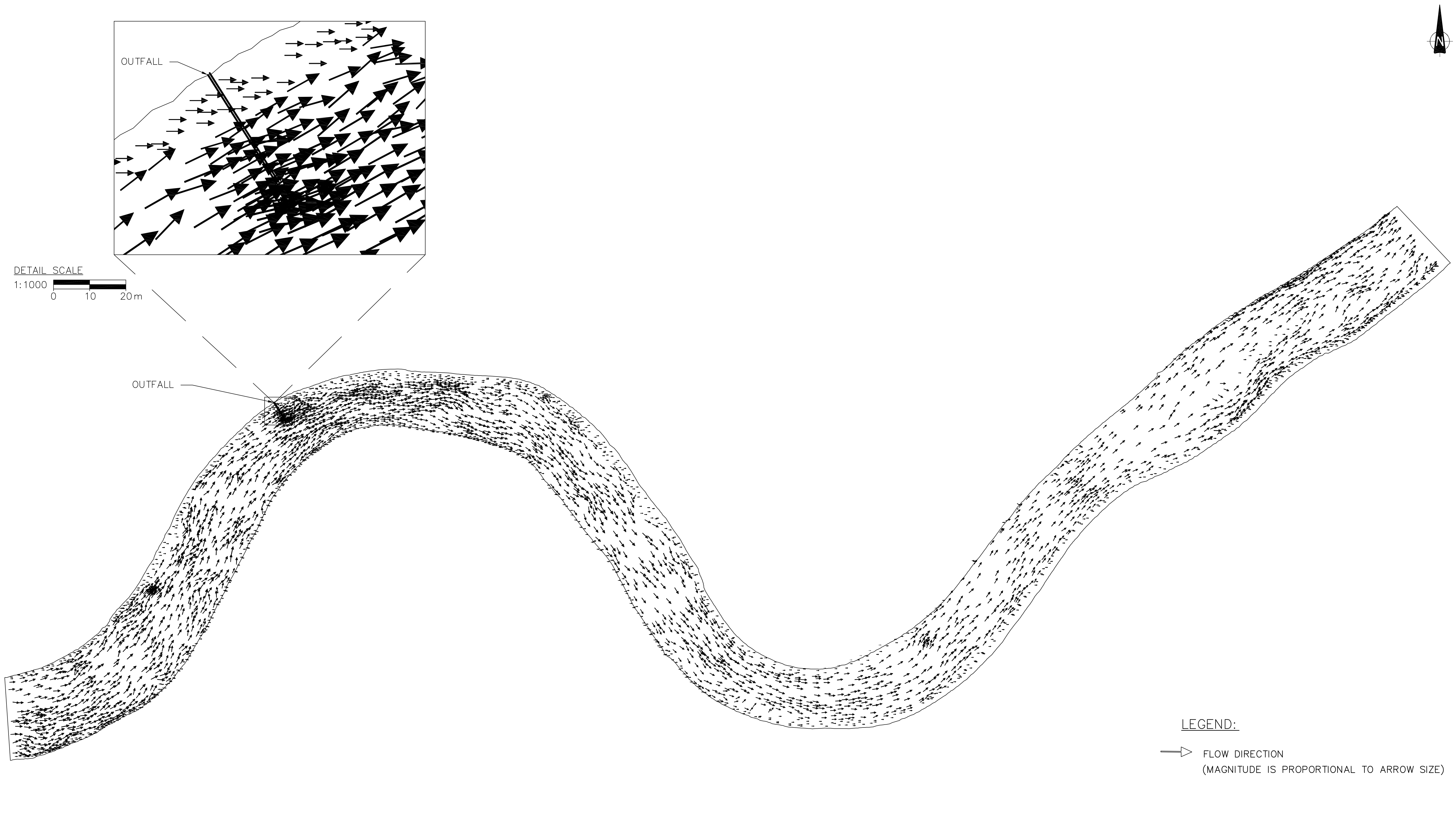
CLIENT	PROJECT: SHORE GOLD RIVER SURVEY	08–10
AMEC PROJECT NO: SX03733		FIGURE 3.2
DWN BY: CG / CHK BY: SS	TITLE: MODEL BATHYMETRY OF SASKATCHEWAN RIVER	amec

3.2 Model Calibration

The flow model was run to generate the observed high-flow rate (1168 m^3/s) by varying Chezy's friction coefficient and comparing the simulated velocities and flow rates throughout the river reach with the observed values. Modeled velocities were also compared with the estimated velocity distribution and flow rate calculated with another flow model (River2D). The estimated low-flow rate of 188 m^3/s, corresponding to the open water 7Q10 flow condition, was achieved in the model using Chezy's friction coefficients of 26 $m^{1/2}/s$ and 28 $m^{1/2}/s$ for the upstream halves and the downstream half of the river reach, respectively. The estimated flow rate of 439 m^3/s, corresponding to the average river flow condition, was achieved in the model simulations using Chezy's friction coefficients of 30 $m^{1/2}/s$ and 32 $m^{1/2}/s$ for the upstream half and the downstream halves of the river reach, respectively. The velocities modeled with AQUASEA were generally the same as those observed and/or found by River2D.

At the end of calibration, the simulated velocities for low-flow in the river varied from 0.08 m/s to 1.2 m/s. In the areas closer to the river banks the velocities would be much smaller, being as low as 0.08 m/s. The simulated velocity distribution in the river reach for the low-flow condition is shown in **Figure 3.3**. The arrow position, length and size reflect the direction and magnitude of flow velocity in the figure. In the average-flow simulations, flow velocities varied from 1.44 m/s near the outfall, to 0.88 m/s near the downstream end of the study reach. The flow parameters for each scenario after calibration are summarized in **Table 3.1**.



SCALE
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CLIENT		PROJECT: SHORE GOLD RIVER SURVEY	08–10
			FIGURE 3.3
AMEC PROJECT NO: SX03733		TITLE: SIMULATED VELOCITY DISTRIBUTION IN THE RIVER REACH (LOW FLOW CONDITION)	amec
DWN BY: CG	CHK BY: SS		

Table 3.1
Flow Parameters After Calibration

Scenario	Discharge m^3/s	Model Variable Depth (m)	Variable Channel Velocity (m/s)	Chezy's Coefficient C* $m^{1/2}/s$
Low Flow	188	0.0–4.3	0.0–1.2	26–28
Average Flow	439	0.0–5.0	0.0–1.4	30–32
High Flow	1168	0.0–7.1	0.0–1.6	36–38

* The numbers indicate average bottom friction factors assigned to the upstream and downstream half of the reach respectively.

3.3 Transport Model Simulations

The transport model was designed to use the same grid as well as the velocities generated in the flow model. Before transport runs were performed, the flow and velocities in the reach were generated through flow runs associated with each scenario. Mine water discharge was simulated in the transport model as a continuous source with a constant discharge rate, tracing chloride concentrations with time. The following assumptions were used in simulating the outfall:

- the outfall discharges mine water near the north bank of the river, where the water depth is greater than 2.5 m during the low flow. No diffuser is used at the end of the discharge pipe;
- the outfall pipe extends away from the bank, towards the middle of the river, making an approximate 90° angle with the river flow lines;
- the total mine water discharge rate is 199 000 m^3/day (2.3 m^3/s) and constant;
- the concentration of chloride in the mine water being discharged is 1725 mg/L;
- the low-flow rate in the river is 16 200 000 m^3/day (188 m^3/s), uniform and steady;
- the average flow rate in the river is 37 900 000 m^3/day (439 m^3/s), uniform and steady; and,
- flow conditions under ice were not investigated.

The modeling scenarios investigated plume migration in the river downstream of the outfall for low-flow and average-flow conditions for the 70 m outfall pipe. The upstream boundary condition was assigned a specified concentration of zero. The downstream boundary was assigned a zero concentration gradient indicating only convective transport of mass through the boundary because of the large distance away from the source. The source boundary was selected on the model grid to coincide with the outfall pipe as shown in **Figure 3.1**.

The main parameters required in the transport model – the longitudinal and transverse dispersion coefficients – were estimated from the literature for similar channels having uniform flows at 1.0 m^2/s and 0.3 m^2/s, respectively. The representative transverse dispersion coefficients, as determined by field tests and laboratory experiments, were summarized in Sumer (1976) and Beltaos (1978). The transverse dispersion coefficient is typically five to ten times smaller than the longitudinal dispersion coefficient for straight channels. Due to the meanders and river morphology a slightly larger transverse dispersion coefficient was used for the river reach.

Transport simulations were conducted for a chloride concentration of 1725 mg/L, estimated to exist in the mine water discharge at the end of the outfall pipe. For the purposes of modeling, chloride ions were again conservatively assumed to be resistant to decay, sorption or other processes that would remove them from solution in the river water. The background river chloride concentrations (see Section 1 for values) were assumed constant everywhere in the river reach modeled, and they were not included in the transport simulations.

Low Flow

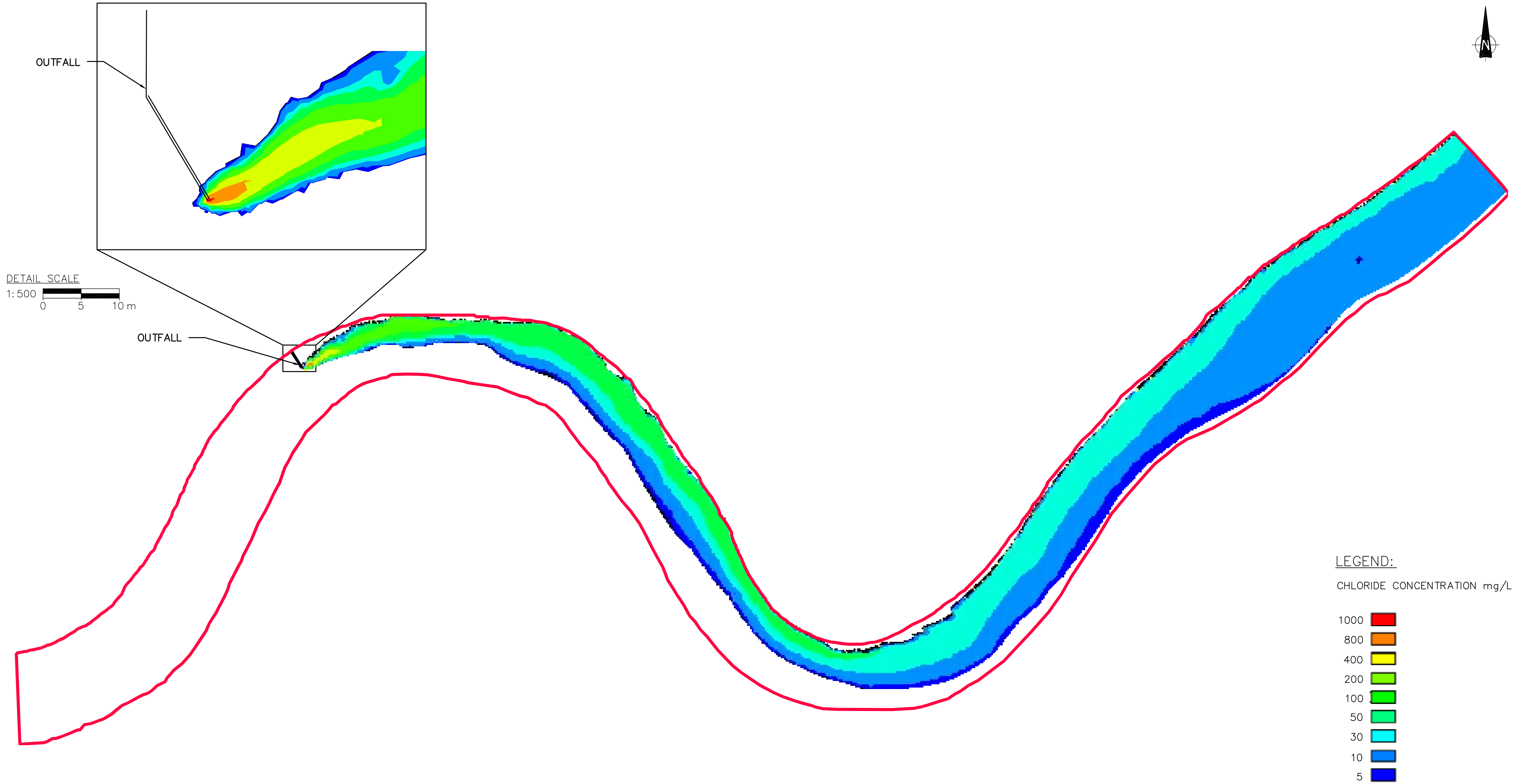
Table 3.2, pertaining to the 7Q10 low-flow condition for the 70 m outfall, summarizes chloride concentrations found after modeling along the riverbank at 100 m downstream of the outfall location, and at 500 to 1000 m intervals thereafter, progressing downstream from the outfall to the end of the modeled river reach (approximately 6 km downstream of the outfall). The lateral distances of 0 m, 10 m, 25 m, 50 m, 75 m, and 100 m, from the riverbank encompass the main portion of the dispersion plume to the end of the reach. The maximum concentrations for chloride are also shown for each downstream distance, recognizing that maximum concentrations do not necessarily coincide with any of the discrete profile distances of 0 m, 10 m, 25 m, 50 m, 75 m or 100 m from shore.

Figure 3.4, shows the modeled river concentrations of chloride in the river reach for low-flow conditions for the 70 m outfall. It should be noted that the tip of the outfall pipe is only 60 m away from the water edge corresponding to the low flow in the river.

Table 3.2
Simulated Concentrations of Chloride as a
Result of Mine Water Discharge into the River during Low Flow

Lateral Distance from the River Bank (m)*	Stations Along River (m)								
	100	500	1000	1500	2000	3000	4000	5000	6000
	Chloride Concentration (mg/l)								
0	0	6	7	8	10	24	43	39	33
10	2	90	76	36	63	39	43	39	33
25	26	94	74	69	62	46	42	38	32
50	252	93	67	64	53	45	41	36	30
75	7	59	54	58	43	41	37	32	27
100	0	11	39	45	29	33	32	28	23
125		0	28	27	17	24	25	23	18
150			13	14	9	14	19	21	13
175			0	5	5	6	14	19	11
200				0.5	0	0	9	17	9.5
250				0			4	14	9
300							0	13	9**
350								8**	
Max. at transect.	260	99	76	72	63	47	43	39	33

* Outfall at 60 m away from the river bank.
** Plume reaches the opposite river bank.



MAIN SCALE
1:5000
0 50 100 m

CLIENT	PROJECT: SHORE GOLD RIVER SURVEY	08–10
AMEC PROJECT NO: SX03733	TITLE: SIMULATED CHLORIDE CONCENTRATIONS (ABOVE GROUND) -LOW FLOW CONDITION -70M OUTFALL	FIGURE 3.4
DWN BY: CL / CHK BY: SS		amec

I:\SX\03733\03733-G01.dwg – FIG5 – Oct. 15, 2010 4:50pm – shelley.brown

*PLOT 1:1=B (L)

Based on the results of transport simulations for the low-flow condition, the plume generated for chloride follows the north shore of the river, expands transversally to approximately 120 m at 500 m and touches the north bank of the river. It contracts to 80 m at 750 m and expands again to 170 m at 1.1 km downstream of the outfall. The plume contracts to 110 m at 2.5 km, where the river enters the second bend and gets deeper. Later about 5 km downstream of the outfall, the plume expands laterally to approximately 350 m reaching to the other bank. The plume front travels at an average velocity of approximately 1.5 km/h, reaching the end of the reach (6.0 km) in about 4 hours. The plume front travels slower through the deeper sections of the river.

The mass flux entering and leaving the reach was computed in the model at flux boundaries installed into the model grids at various transects, to verify that no significant loss of chloride mass occurred in the reach during model simulations. Conservation of the total amount of chloride flowing in and out of the study reach was checked to verify model reliability against numerical dispersion and convergence in time. Review of the time series data accumulated in a file at the end of the low-flow transport simulations indicated that the total mass flux reached a steady state at 6.0 km downstream after approximately 4.5 hrs of operation of the outfall.

Modeling has demonstrated that the chloride concentration entering into the river would be immediately diluted by river water to approximately 62% at the outfall location during the open water low-flow conditions. It should be noted that the model does not solve for the near field (jet) dispersion, and that the concentrations in the river in close proximity to the outfall are approximate. The plume chloride concentration along the north bank at a distance of 500 m downstream of the outfall would be 90 mg/L. At a point 1000 m downstream, the chloride plume concentration near the bank would be diluted to approximately 76 mg/L for a 70 m outfall (**Table 3.2**). During low flow, at the end of the reach, the plumes' maximum chloride concentration was predicted at approximately 33 mg/L (less than 2% of the initial concentration). The lateral (across channel) maximum concentration gradient at the end of the reach was approximately 13 mg/L/100 m. A comparison of concentrations at the end of each river bend (approximately 2.0 km and 4.0 km downstream of the outfall) indicated that the plume expanded laterally at these two locations, likely due to higher velocities, shallower depths and greater transverse velocity gradients at these locations.

Another scenario with a shorter (40 m) outfall was also run to test the behaviour of the plume if the mine discharge is closer to the bank. Model results with the plume generated from the 40 m outfall indicated two to three times higher river concentrations within 500 m downstream of the outfall. At a point 1000 m downstream, the chloride plume concentration near the bank would be diluted to approximately 100 mg/L. Further downstream, the plume had similar characteristics to the one generated by the 70 m outfall scenario with chloride concentrations at the bank reducing to 35 mg/L before the plume left the study reach. The maximum difference between the 40 m and 70 m outfall plume concentrations at the end of the reach was 2 mg/L.

Mixing Under Ice Cover

An important feature of rivers and channels is that their morphology and flow-resistance behaviour vary interactively with flow and ice conditions. Depending on flow magnitude, ice covers modify the interaction, over a range of scales in space and time. The surface ice would create another friction boundary at the top of the water column, resulting in a greater variability in vertical velocity gradients compared to open water flow conditions. Ice-cover influences flow distribution, solute transport by ice, solute transport under ice, and channel morphology. The impacts can include raised water levels, laterally redistributed flow, secondary currents, and other effects. Hydraulic and physical properties of flow under ice and the influence of ice-cover on flow and bed load transport are discussed in Hains (2004), White (1999) and Smith (1995).

The 7Q10 low flow under ice cover was estimated as 169 m^3/s (14 600 000 m^3/d) for the study reach. The flow under ice would take place through a larger cross-sectional area with smaller average longitudinal velocities compared with the open water low-flow condition. However, under ice the flow would be diverted transversally in braided channels formed by ice jams and water frozen to the bottom at shallow areas. Mixing would be enhanced due to these braided channels and secondary currents. It was estimated that the mixing under ice cover would be greater than, or at least similar to, the open water low-flow scenario investigated with the numerical model. It should be noted that further downstream, where the plume is completely mixed with the low-flow rate given for the under-ice condition, the chloride concentrations would be in the order of 33 mg/L (23 mg/L above background).

River ice flow considerations are discussed in detail in the Preliminary Hydrotechnical Evaluation Report.

Average Flow

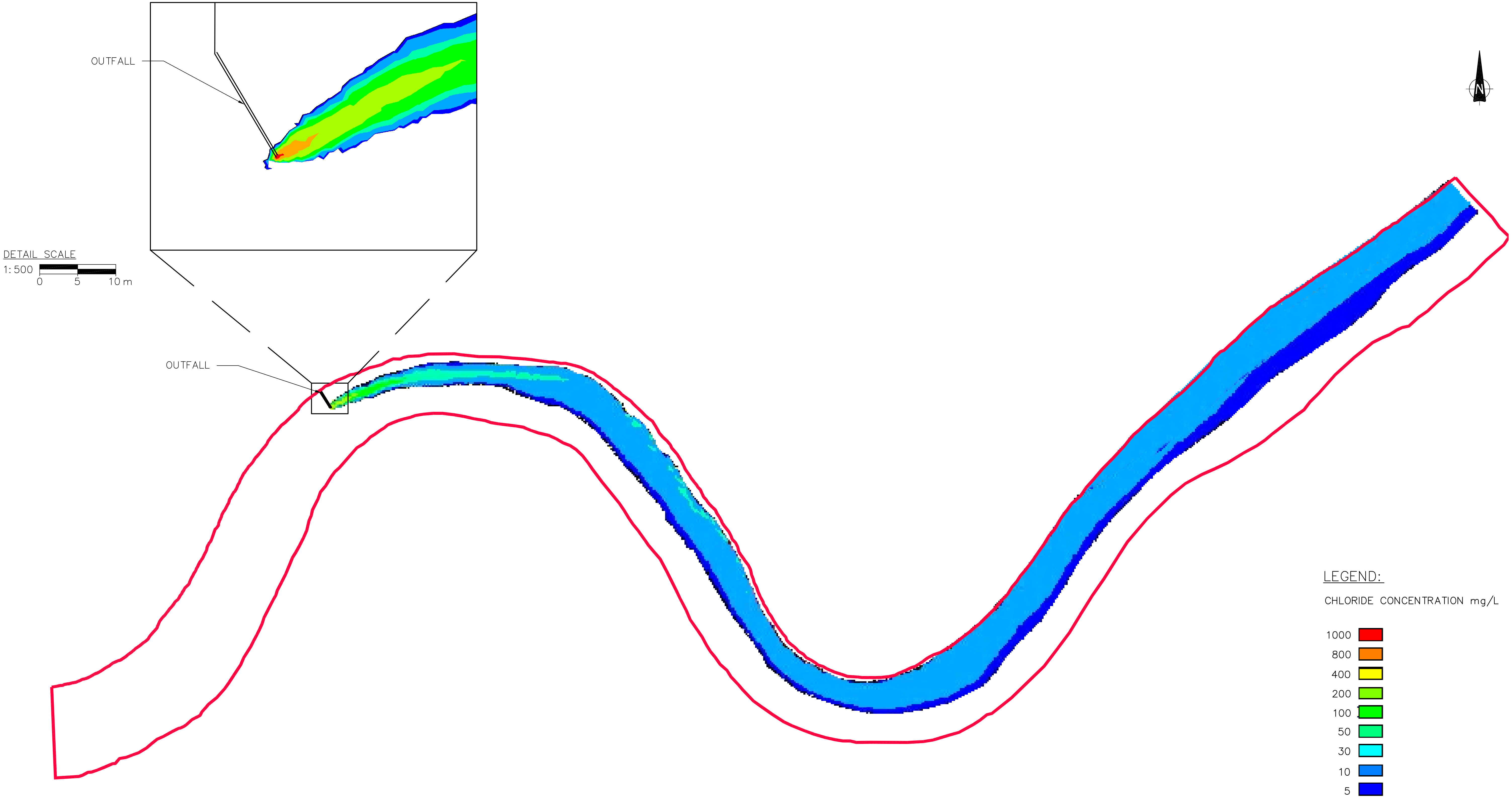
Table 3.3 summarizes the predicted concentrations chloride for average-flow conditions at the same locations mentioned for the low-flow condition. **Figure 3.5** shows modeled chloride concentration distribution in the river reach for average-flow conditions for the 70 m outfall. No model runs were done for 40 m outfall for average-flow conditions.

Model output concentrations are directly proportional to input loadings, such that if the chloride concentration of 1725 mg/L at the outfall were to be replaced with a chloride concentration of 862.5 mg/L, then river concentrations would be decreased by half, excluding background values.

Table 3.3
Simulated Concentrations of Chloride as a
Result of Mine Water Discharge into the River during Average Flow

Lateral Distance from the River Bank (m)*	Stations Along River (m)								
	100	500	1000	1500	2000	3000	4000	5000	6000
	Chloride Concentration (mg/l)								
0	0	2	20	26	26	25	22	23	20
10	0	5.5	28	29	30	26.5	24	23	20
25	0	23	29	30	29	26	23.8	22	19
50	67	42	31	28	27	24	22	20	17
75	55	15	27	26	21	21	19	17	14
100	0	0	19	20	15	15	15	13	11
125			11	12	9	9	10.5	10	7
150			5	5	3	4	6.5	8	4
175			0	0	0	0	4	7	3
200							1.5	5.5	2
250							0	3	1
300								1.5	0.5**
350								0	
Max. at transect.	116	42	31	30	30	26.5	24	23	20

* Outfall at 70 m away from the bank.
** Plume reaches the opposite river bank.



MAIN SCALE
1:5000
0 50 100 m

CLIENT	PROJECT: SHORE GOLD RIVER SURVEY	08–10
		FIGURE 3.5
AMEC PROJECT NO: SX03733	TITLE: SIMULATED CHLORIDE CONCENTRATIONS (ABOVE GROUND) AVERAGE FLOW CONDITION -70M OUTFALL	amec
DWN BY: CL / CHK BY: SS		

For the average-flow condition, transport simulations indicated that the chloride plume follows the north shore of the river, expands transversally to approximately 100 m at a point 500 m downstream of the outfall, and contracts to 75 m at 750 m downstream of the outfall. The plume then expands again to 160 m at 1.0 km downstream, and contracts down to 105 m at 2.5 km downstream before the second bend in the river reach. The modeled plume front travels at an average velocity of approximately 2.0 km/h, reaching the end of the study reach (6.0 km) in about 3 hours.

The mass flux entering and leaving the reach was computed in the model and it was verified that no significant loss of mass occurred in the reach during average-flow model simulations. In the average-flow condition, the chloride concentration entering into the river will stay separated from the bank and will not reach the north bank until a point approximately 700 m downstream of the outfall. At 1000 m downstream, the chloride concentration near the bank would be diluted to approximately 20 mg/L, (**Table 3.3**). In general, the chloride plume stayed near the outfall bank of the river for the entire length of the reach. During the average-flow condition, at the end of the reach, the plumes' maximum chloride concentration was approximately 20 mg/L (approximately 1% of the initial concentration). The lateral maximum concentration gradient was approximately 10 mg/L/100 m for chloride.

4.0 SUMMARY AND CONCLUSIONS

The loading of chloride ions to the Saskatchewan River, as a continuous mine water discharge, was estimated using a numerical flow and transport model. Hydraulic parameters (velocity, water depths, etc.) characteristic of the outfall reach were obtained from analysis of water level and discharge measurements for open water conditions in the reach, as well as detailed bathymetric data obtained through a comprehensive field program undertaken over the period of June 24th through June 28th, 2010. During this time, a detailed bathymetric survey, water level surveys and discharge measurements were completed. The channel bed survey was conducted from a boat with a depth sounder and attached GPS. Bathymetric contours were generated from the bed point data using a contouring software package and extrapolated to the river banks by interpolation with the topographic data.

Characteristic discharges at the site were obtained by statistical analysis of daily discharges computed from the WSC station data. The hydraulic parameters (velocity, water depths, etc.) characteristic of the outfall reach were obtained from analysis of water level and discharge measurements for the open water condition. These parameters were used as baseline data to calibrate the flow and transport model AQUASEA, which was used to estimate plume concentrations downstream of the proposed outfall location.

Based on a discharge of 199 000 m^3/day and mine water, chloride concentration of 1725 mg/L, the flow and transport modeling indicated that the outfall concentrations would be reduced by dilution and hydrodynamic mixing. During low flow, at a point 500 m downstream of the 70 m outfall, the predicted maximum chloride concentration was 90 mg/L near (10 m removed) the riverbank and the plume centreline was 25 m from the bank transversally, where a peak chloride concentration of 99 mg/L was predicted. The plume width was approximately 95 m for 70 m outfall at 500 m. For comparison, the federal government is currently in the process of developing a Canadian Water Quality Guideline for chloride for the protection of aquatic life. The suggested concentration value in the development document (which is preliminary) is 128 mg/L for the freshwater environment. British Columbia, the only province that to our knowledge has an established water quality guideline for the protection of aquatic life, has set a guideline value of 150 mg/L (Nagpal et al. 2003).

For the 40 m outfall offshore distance condition, , the plume centreline chloride concentration was predicted at 315 mg/L at a point 500 m downstream of the outfall. The plume width for this condition was approximately 85 m for 40 m outfall at 500 m. The plume generated by the 40 m outfall was closer to the river bank with higher peaks at the centreline. At 100 m downstream from the 40 m outfall the plume centreline chloride concentration was predicted at approximately 590 mg/L.

Modeling also predicted that the chloride concentrations would be reduced at faster rates in areas of the reach where there are relatively higher flow velocities, shallower depths and secondary currents due to meanders. Maximum chloride concentrations at the end of the reach, in the low-flow condition, were reduced to 33 mg/L (2% of initial concentration). The maximum transverse concentration gradient at the end of the reach for chloride was approximately 13 mg/L/100m.

Due to the conservative modeling assumptions as specified in Section 3.0, it should be noted that actual chloride concentrations in the Saskatchewan River during mine water discharge (excluding background additions to the computed values), are expected to be lower than those predicted by the modeling. Any features that influence the flow path (rapids, islands, meanders, etc.), the effects of which were averaged during modeling, would also increase turbulence, initiate secondary currents and promote mixing. These factors would further contribute to reducing overall predicted concentrations.

5.0 CLOSURE

The investigations and activities presented in this report were conducted in accordance with generally accepted hydrotechnical and environmental assessment principles and practice. The report has been compiled by AMEC based on information assembled by AMEC.

Should any questions arise concerning the preparation of this report or its conclusions, please contact the undersigned at your convenience.

Yours truly,

AMEC Earth & Environmental

03 December 2010

Sukru Sumer, Ph.D., P.Eng.
Senior Associate Environmental Engineer

Reviewed by:

David A. Simms, Ph.D.
Principal, Environmental Assessment and Resource Development

SS/DAS/hb/lb

Permit to Practice No. P-4546

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APPENDIX A

Velocity and Discharge Measurements

Save Close

UNIVERSITY OF ALBERTA

Discharge Measurement Summary

Date

Station Information

Station Number

Station Name

Location

Measurement Information

Measurement No.

Compiled By

Checked By

Personnel and Equipment

Party

Boat/Motor/Platform

Rating Information

Gage Height		Rating Discharge		Rating No.	
GH Change		Index Velocity		Meas. Rating	
% Diff.	0.0%	Rated Area		Control Code	

System Information

Serial #	M509
System Frequency	3000 kHz
Firmware Version	9.6
RiverSurveyor Ver	v4.50

System Setup

# of Cells	20	Averaging Interval	5.0
Cell Size	0.20	Magnetic Decl.	10.1
Blanking Distance	0.20	Salinity	0.00
Transducer Depth	0.06	Echo Sounder	Not Pres.

Discharge Calculation Settings

Velocity Ref.	GPS	Top Estimate	Power	Left Bank	Sloped
Track Ref.	GPS	Bottom Est.	Power	Right Bank	Sloped
Depth Reference	ADP	Area Method	none	Orient. Profiles	all

Computed Discharge Results

Width	329.2
Area	705.7
Mean Velocity	1.58
Discharge	1117.9
% Measured	67.0
Adj. Mean Velocity	0

Diagnostic Files

Moving Bed Test

Compass Cal

Pressure Cal

Depth Calibration

Measurement Results

		Discharge						Distance				Time		Mean Vel		#Profiles	
Tr#		Top	Middle	Bottom	Left	Right	Total	Left	Right	Total	Area	Start	End	Boat	Water	Total	Bad
16	R	207.3	779.02	191.33	0	0	1177.7	0.0	3.2	326.6	728.6	11:40	11:47	0.81	1.62	80	0
26	L	200.1	617.56	172.29	0	5.6515	995.6	3.0	6.0	326.9	631.2	11:47	11:56	0.62	1.58	103	0
27	R	204.27	797.59	179.52	0.97894	0	1182.4	8.0	6.0	328.5	747.2	11:56	12:02	0.91	1.58	69	0
35	L	198.47	732.36	128.43	0	-2.6705	1056.6	8.0	5.0	332.4	661.0	12:02	12:12	0.56	1.60	114	0
42	R	202.89	826.44	146.69	1.1379	0	1177.2	5.0	5.0	331.6	760.5	12:12	12:19	0.70	1.55	92	0
Mean		202.61	750.6	163.65	0.42337	0.59618	1117.9	4.8	5.0	329.2	705.7	Total	04:40	0.72	1.58	92	0
SDev		3.4764	81.852	25.594	0.58244	3.0534	86.538	3.4	1.1	2.7	56.5			0.14	0.03		
COV		0.017	0.109	0.156	1.376	5.122	0.077	0.713	0.227	0.008	0.080			0.198	0.016		

Tr16=AMEC1006241146.ADP; Tr26=AMEC1006241153.ADP; Tr27=AMEC1006241202.ADP; Tr35=AMEC1006241208.ADP; Tr42=AMEC1006241218.ADP;

Comments

Note: Units for the above parameters are: Distance (m), Velocity (m/s), Area (m2), Discharge (m3/s)

Save Close

UNIVERSITY OF ALBERTA

Discharge Measurement Summary

Date

Station Information
Station Number
Station Name
Location

Measurement Information
Measurement No.
Compiled By
Checked By

Personnel and Equipment
Party
Boat/Motor/Platform

Rating Information

Gage Height		Rating Discharge		Rating No.	
GH Change		Index Velocity		Meas. Rating	
% Diff.	0.0%	Rated Area		Control Code	

System Information

Serial #	M509
System Frequency	3000 kHz
Firmware Version	9.6
RiverSurveyor Ver	v4.50

System Setup

# of Cells	20	Averaging Interval	5.0
Cell Size	0.20	Magnetic Decl.	10.1
Blanking Distance	0.20	Salinity	0.00
Transducer Depth	0.06	Echo Sounder	Not Pres.

Discharge Calculation Settings

Velocity Ref.	GPS	Top Estimate	Power	Left Bank	Sloped
Track Ref.	GPS	Bottom Est.	Power	Right Bank	Sloped
Depth Reference	ADP	Area Method	none	Orient. Profiles	all

Computed Discharge Results

Width	302.2
Area	720.0
Mean Velocity	1.56
Discharge	1122.2
% Measured	68.5
Adj. Mean Velocity	0

Diagnostic Files
Moving Bed Test
Compass Cal
Pressure Cal
Depth Calibration

Measurement Results

		Discharge						Distance				Time		Mean Vel		#Profiles	
Tr#		Top	Middle	Bottom	Left	Right	Total	Left	Right	Total	Area	Start	End	Boat	Water	Total	Bad
47	R	172.27	797.12	125.78	7.7429	0	1102.9	10.0	6.0	299.3	703.1	13:03	13:10	0.64	1.57	88	0
51	L	183.94	687.92	252.56	0	-1.9118	1122.5	10.0	6.0	300.4	715.3	13:11	13:17	0.69	1.57	83	0
54	R	173.92	734.5	164.95	-2.1915	0	1071.2	10.0	7.0	304.4	678.3	13:20	13:28	0.56	1.58	103	0
60	*L*	*179.1*	*856.53*	*153.71*	*0*	*2.9938*	*1192.3*	*10.0*	*6.0*	*304.7*	*783.5*	*13:29*	*13:35*	*0.80*	*1.52*	*72*	*0*
Mean		177.31	769.02	174.25	1.3879	0.27049	1122.2	10.0	6.3	302.2	720.0	Total	04:40	0.67	1.56	87	0
SDev		5.2936	73.521	54.74	4.3609	2.0269	51.297	0.0	0.5	2.8	45.0			0.10	0.03		
COV		0.030	0.096	0.314	3.142	7.493	0.046	0.000	0.080	0.009	0.063			0.151	0.017		

Tr47=AMEC1006241309.ADP; Tr51=AMEC1006241317.ADP; Tr54=AMEC1006241326.ADP; Tr60=AMEC1006241335.ADP;

Comments

Note: Units for the above parameters are: Distance (m), Velocity (m/s), Area (m2), Discharge (m3/s)

Save Close

UNIVERSITY OF ALBERTA

Discharge Measurement Summary

Date

Station Information
Station Number
Station Name
Location

Measurement Information
Measurement No.
Compiled By
Checked By

Personnel and Equipment
Party
Boat/Motor/Platform

Rating Information

Gage Height		Rating Discharge		Rating No.
GH Change		Index Velocity		Meas. Rating
% Diff.	0.0%	Rated Area		Control Code

System Information

Serial #	M509
System Frequency	3000 kHz
Firmware Version	9.6
RiverSurveyor Ver	v4.50

System Setup

# of Cells	20	Averaging Interval	5.0
Cell Size	0.20	Magnetic Decl.	10.1
Blanking Distance	0.20	Salinity	0.00
Transducer Depth	0.07	Echo Sounder	Not Pres.

Discharge Calculation Settings

Velocity Ref.	GPS	Top Estimate	Power	Left Bank	Sloped
Track Ref.	GPS	Bottom Est.	Power	Right Bank	Sloped
Depth Reference	ADP	Area Method	none	Orient. Profiles	all

Computed Discharge Results

Width	324.8
Area	1157.0
Mean Velocity	1.01
Discharge	1168.0
% Measured	76.8
Adj. Mean Velocity	0

Diagnostic Files
Moving Bed Test
Compass Cal
Pressure Cal
Depth Calibration

Measurement Results

		Discharge						Distance				Time		Mean Vel		#Profiles	
Tr#		Top	Middle	Bottom	Left	Right	Total	Left	Right	Total	Area	Start	End	Boat	Water	Total	Bad
1	R	144.77	941.19	149.01	0.46286	0.13203	1235.6	2.8	1.8	324.4	1155.7	08:54	09:00	0.83	1.07	74	0
3	L	131.54	851.63	133.61	-0.26622	-0.21782	1116.3	2.8	1.8	325.3	1155.5	09:01	09:07	0.80	0.97	78	0
4	R	143.5	946.8	132.32	0.16495	0.29583	1223.1	1.7	2.1	324.0	1154.6	09:07	09:14	0.74	1.06	85	0
6	L	129.19	861.28	121.56	0.069379	0.32896	1112.4	3.0	1.8	325.3	1169.3	09:15	09:21	0.80	0.95	75	0
7	R	143.48	947.5	143.48	0.1314	0.40373	1235	1.7	2.7	325.1	1169.5	09:21	09:28	0.75	1.06	84	0
8	L	131.02	834.84	130.44	0.62211	0.23936	1097.2	3.4	1.8	325.1	1134.6	09:28	09:34	0.89	0.97	71	0
9	R	142.51	932.48	139.05	0.10777	0.26316	1214.4	1.7	1.9	324.3	1155.3	09:42	09:48	0.78	1.05	81	0
10	L	129.7	856.57	123.26	0.12552	0.25115	1109.9	2.7	1.8	325.1	1161.2	09:49	09:54	0.96	0.96	66	0
Mean		136.96	896.54	134.09	0.17722	0.21205	1168	2.5	2.0	324.8	1157.0	Total	01:00	0.82	1.01	77	0
SDev		7.1209	49.385	9.4595	0.2666	0.19031	63.689	0.7	0.3	0.5	11.0			0.07	0.05		
COV		0.052	0.055	0.071	1.504	0.897	0.055	0.273	0.161	0.002	0.009			0.087	0.053		

Tr1=AMEC1006280955.ADP; Tr3=AMEC1006281002.ADP; Tr4=AMEC1006281008.ADP; Tr6=AMEC1006281016.ADP; Tr7=AMEC1006281022.ADP; Tr8=AMEC1006281029.ADP; Tr9=AMEC1006281043.ADP; Tr10=AMEC1006281050.ADP;

Comments

Note: Units for the above parameters are: Distance (m), Velocity (m/s), Area (m2), Discharge (m3/s)

APPENDIX B

Monthly Flow Duration Curves

Appendix B
Monthly Flow Duration Curves

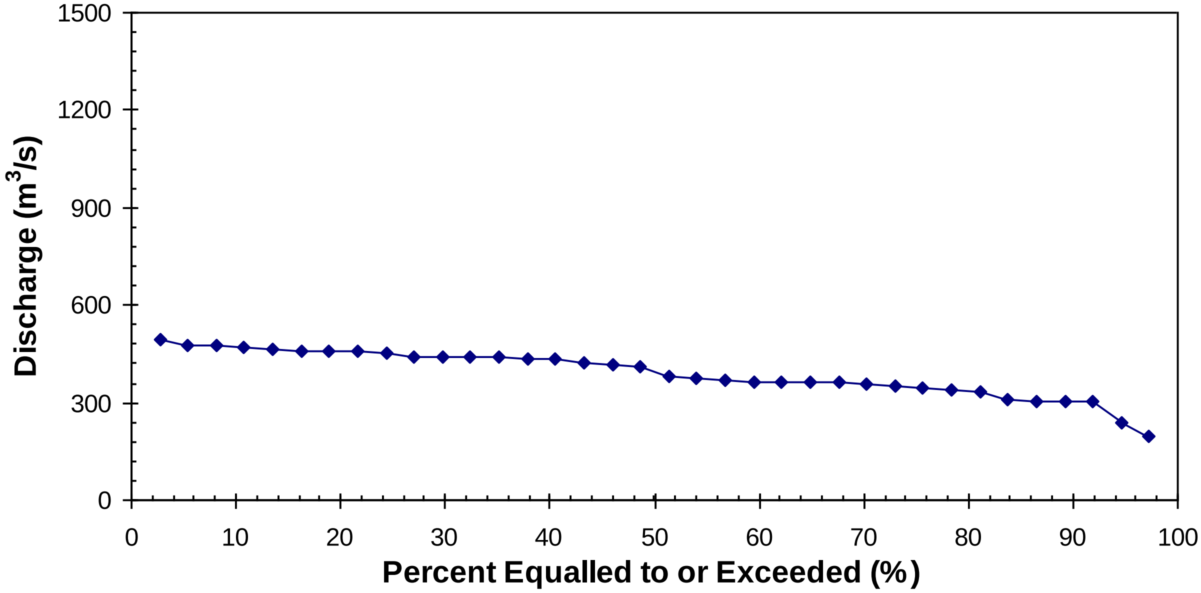

Figure B1 **Flow duration curve for January, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–1986; 1992–2009.**

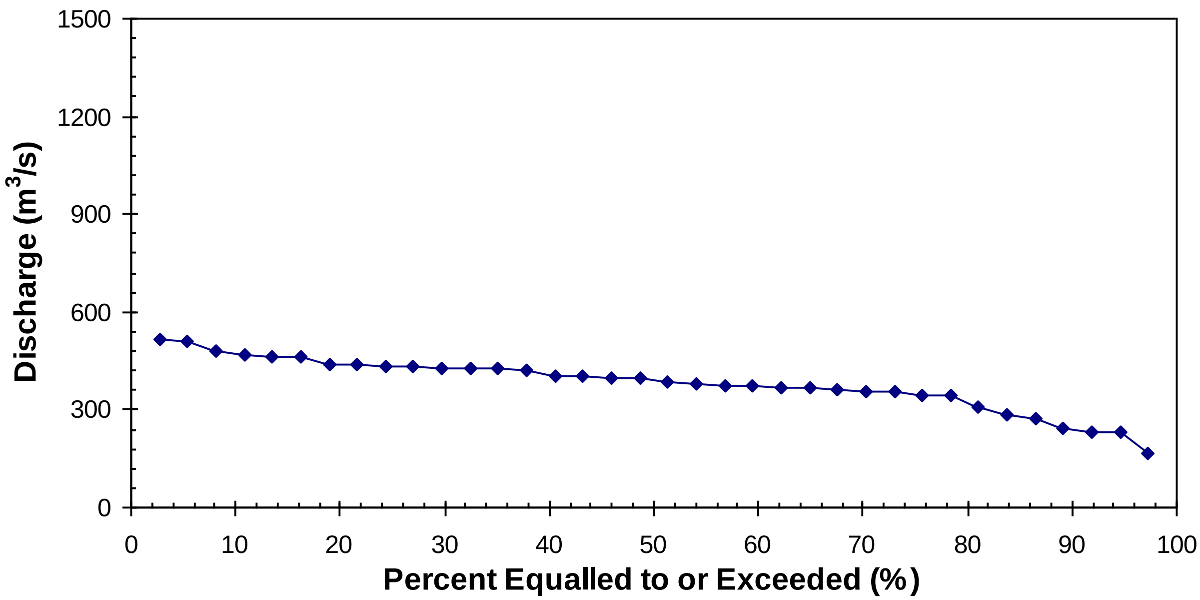

Figure B2 **Flow duration curve for February, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–1986; 1992–2009.**

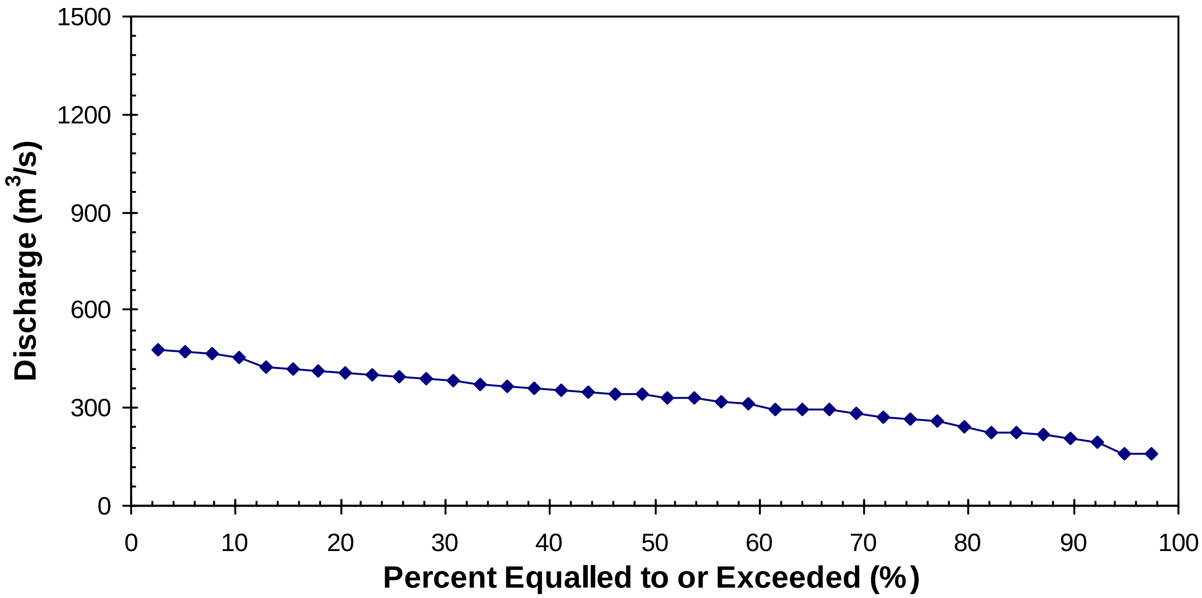

Figure B3 **Flow duration curve for March, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–1988; 1992–2009.**

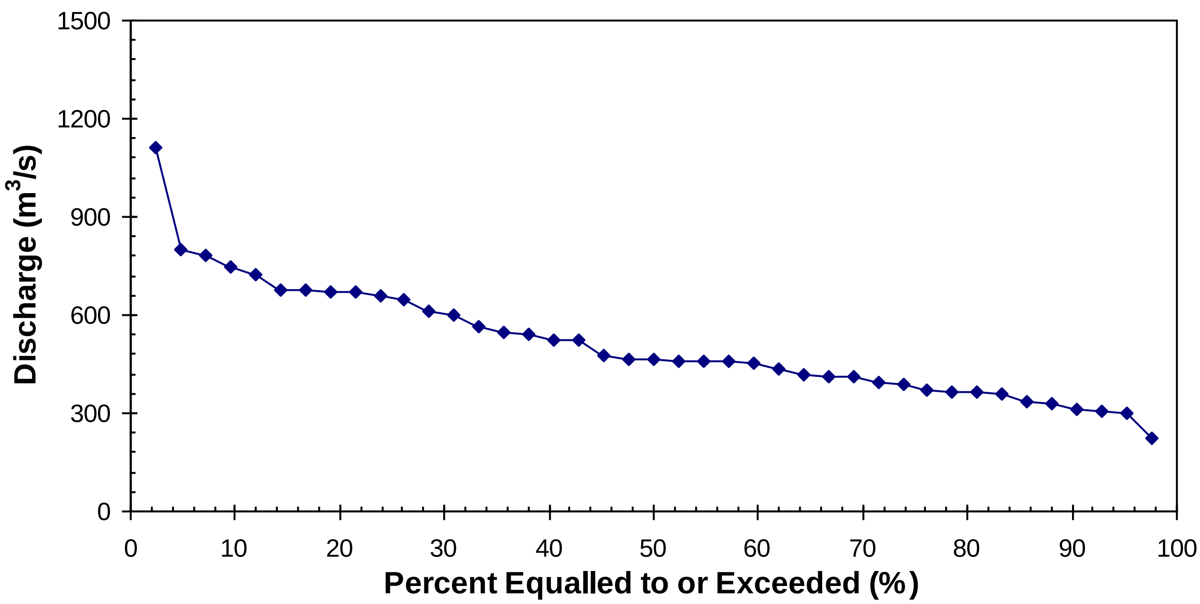

Figure B4 **Flow duration curve for April, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.**

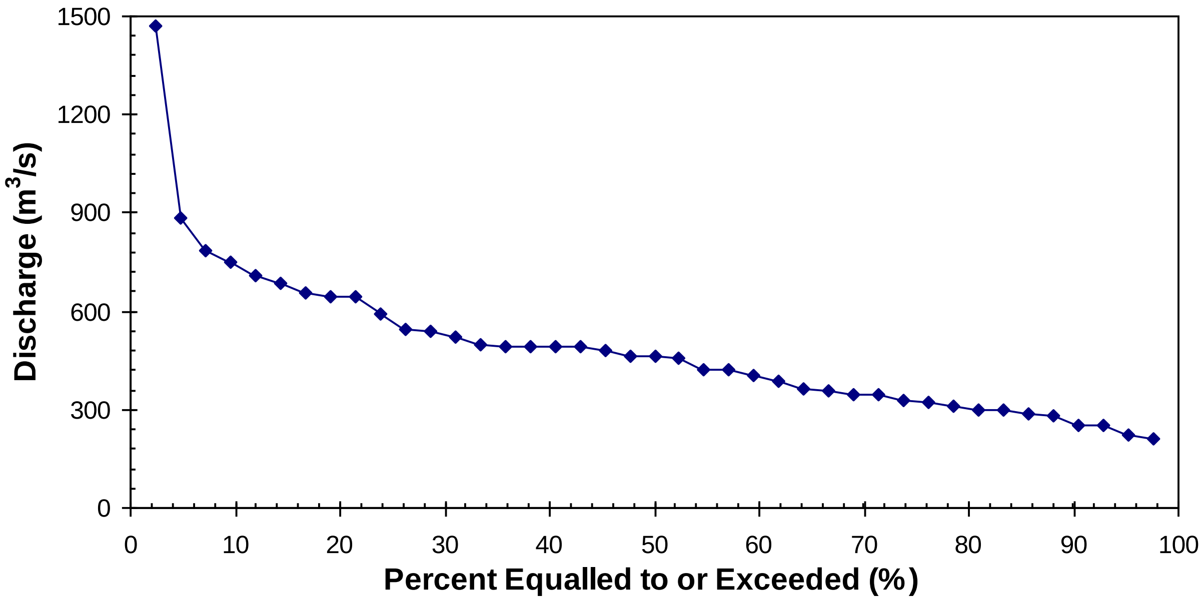

Figure B5 **Flow duration curve for May, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.**

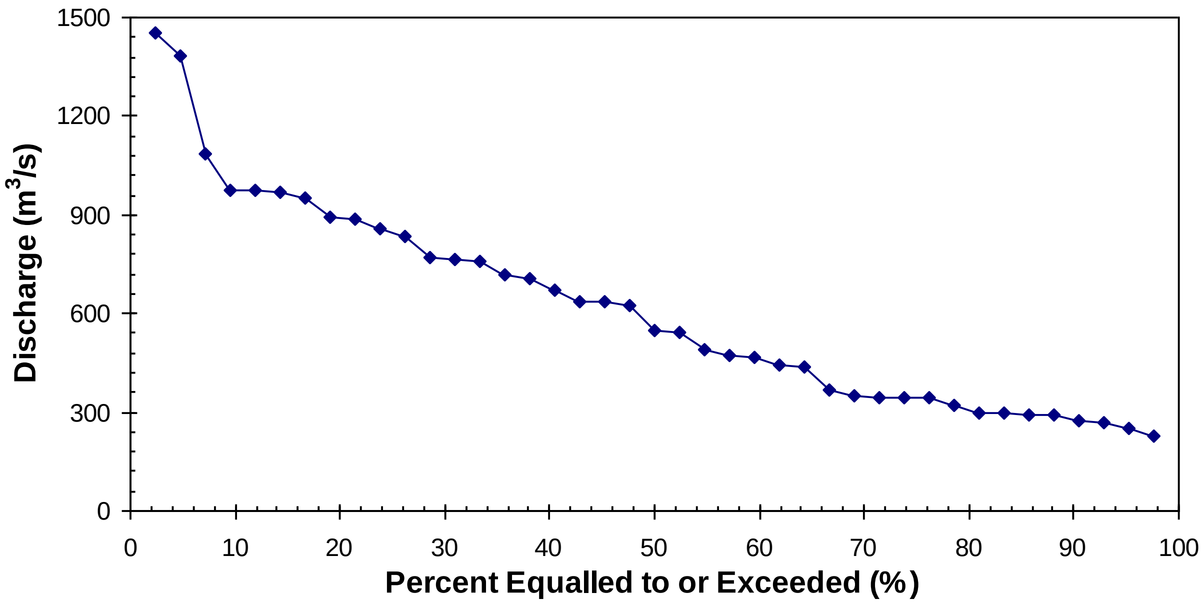

Figure B6 **Flow duration curve for June, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.**

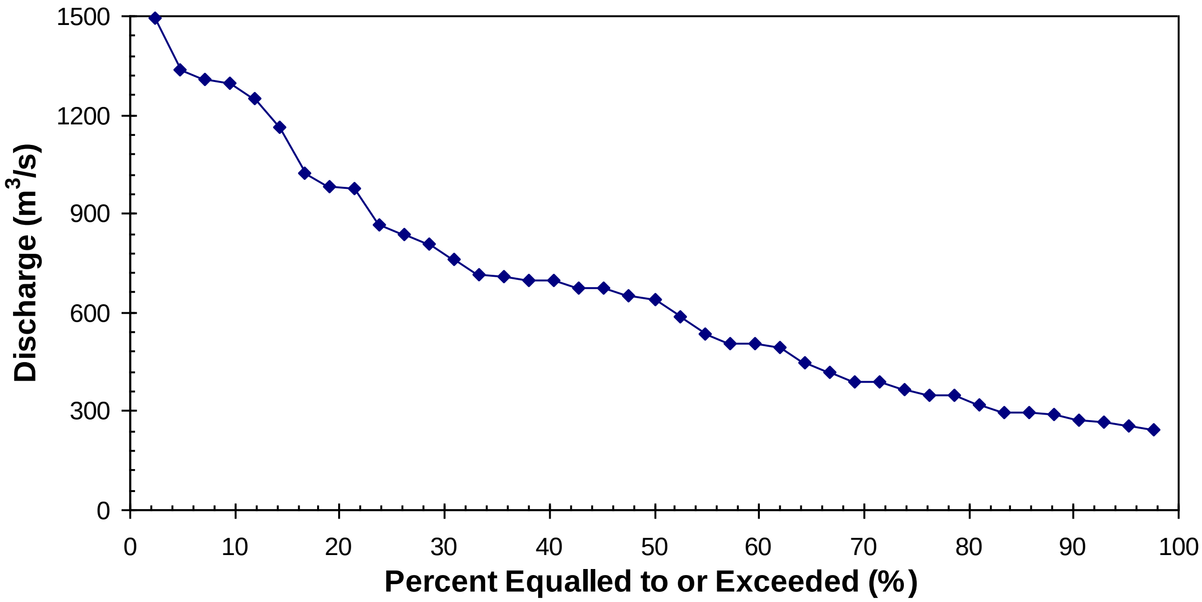

Figure B7 **Flow duration curve for July, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.**

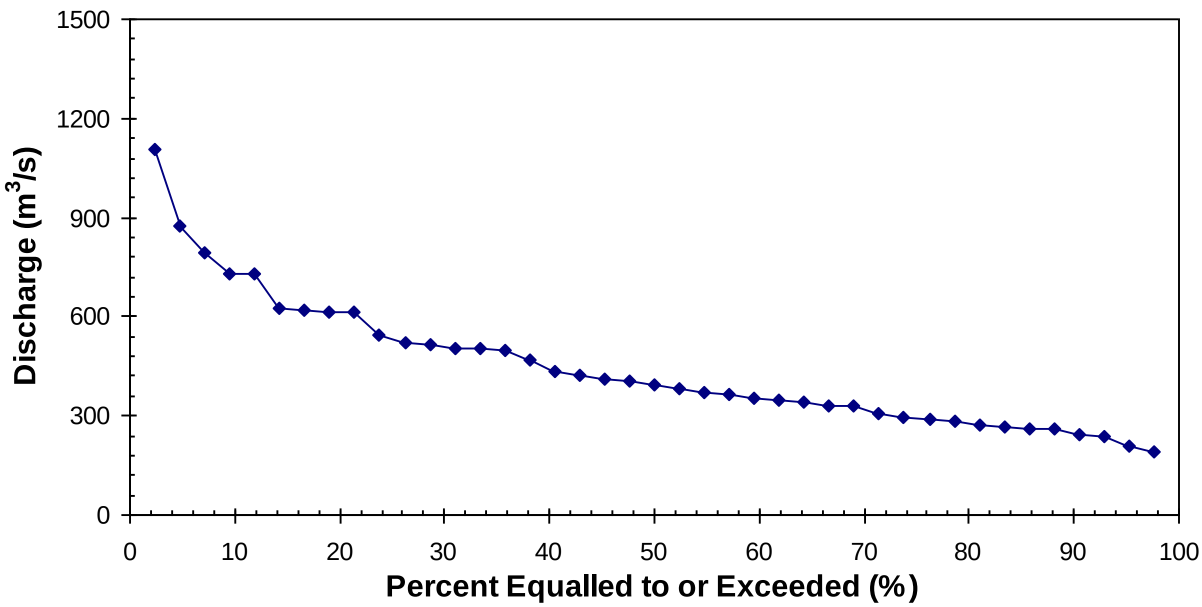

Figure B8 **Flow duration curve for August, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.**

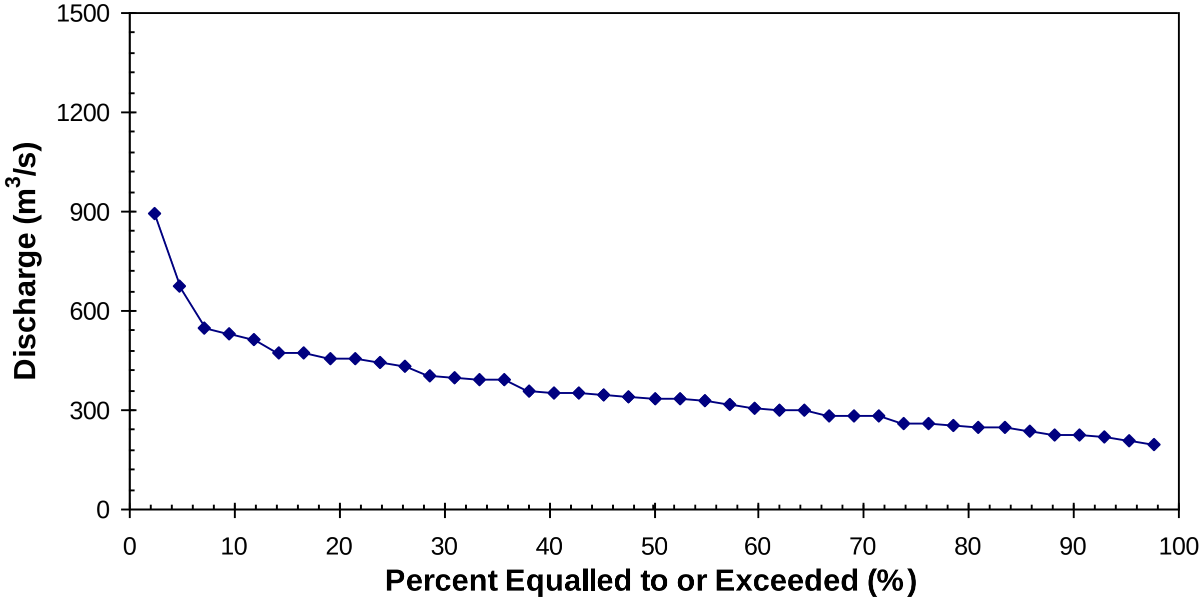

Figure B9 **Flow duration curve for September, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.**

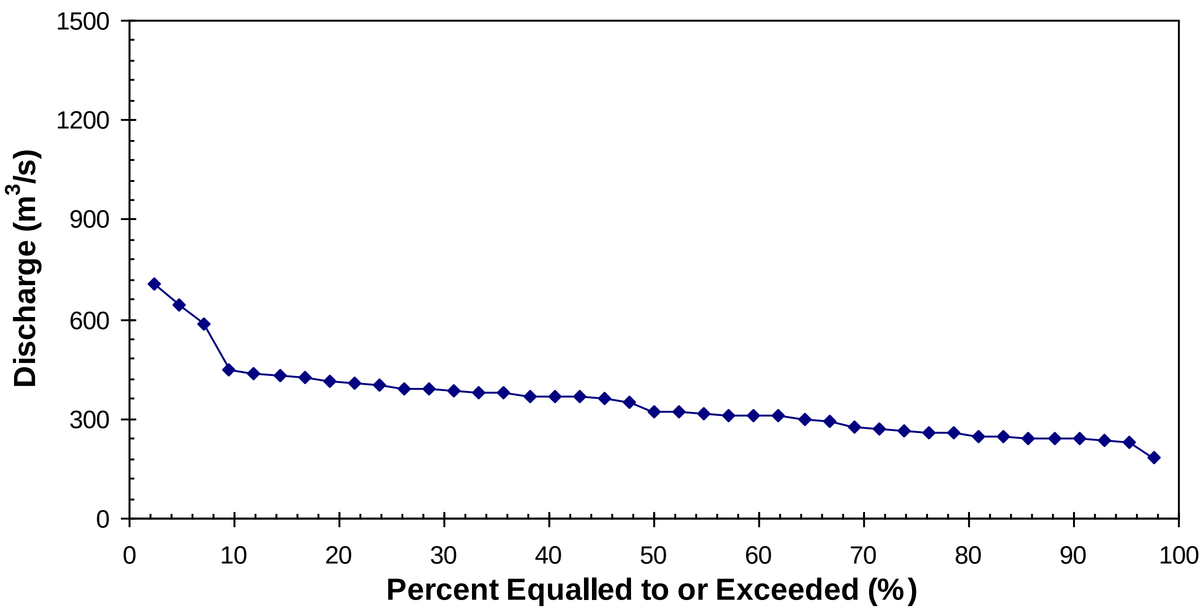

Figure B10 **Flow duration curve for October, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.**

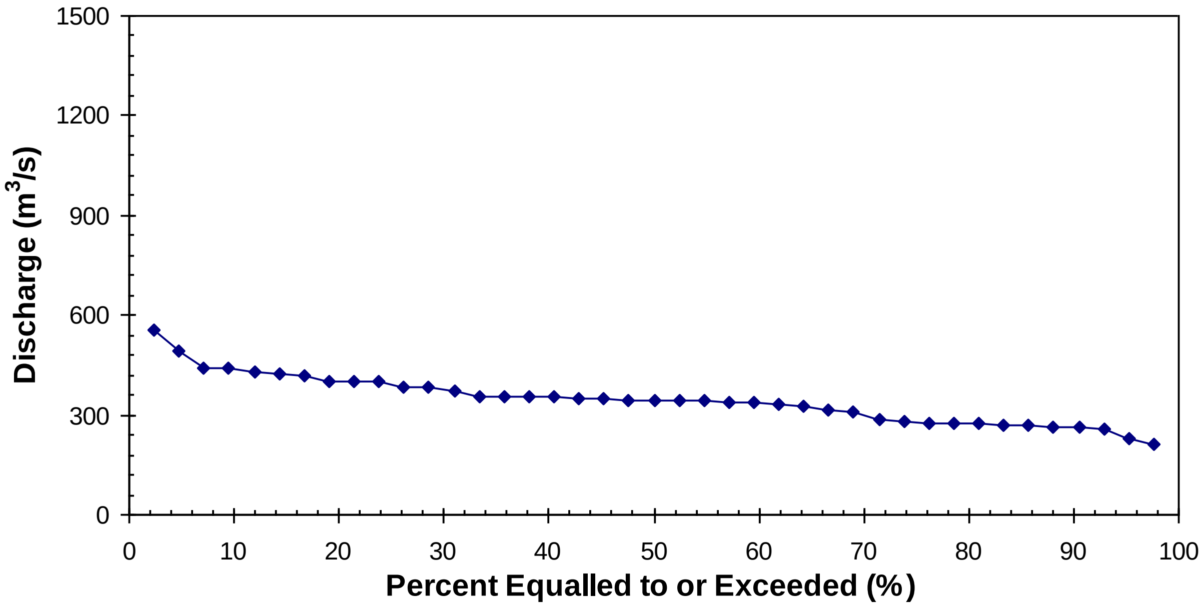

Figure B11 Flow duration curve for November, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.

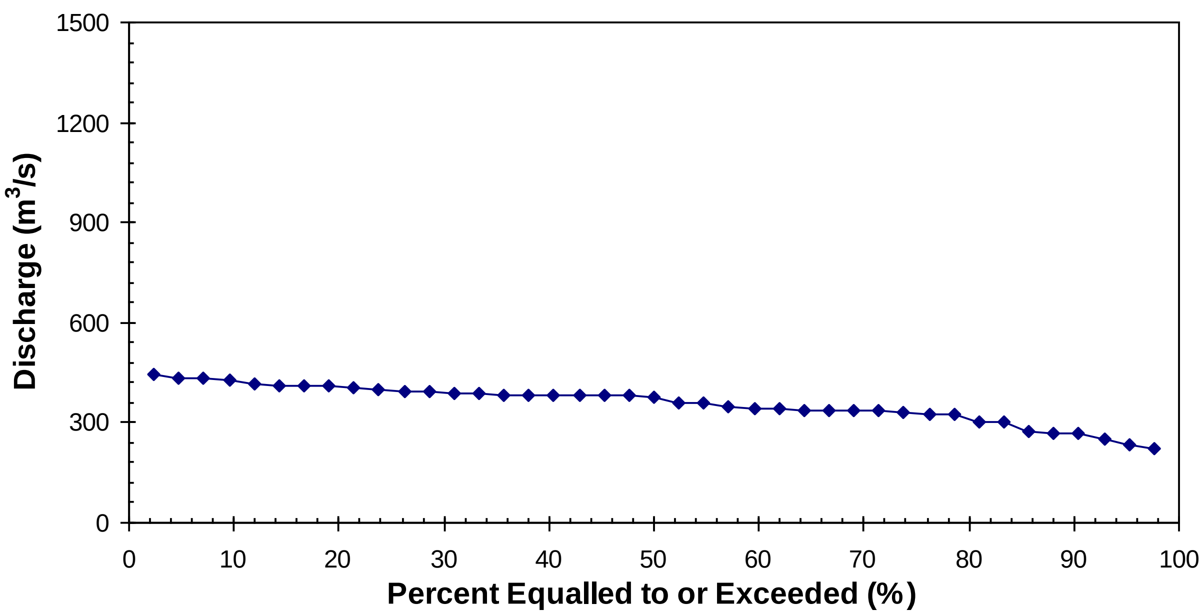

Figure B12 Flow duration curve for December, site based on data from the North Saskatchewan River at Prince Albert (05GG001) and the South Saskatchewan River at Saskatoon (05HG001), 1969–2009.

APPENDIX C

Detailed Bed Form Type Analysis

Appendix C
Detailed Bed Form Type Analysis

Stream Power and Mean Particle Diameter

The first relationship considered was of bed forms to stream power and median fall diameter of bed sediment (mean bed material size), D_{50}, as developed by Simons and Richardson (1966). The stream power can be defined as:

$$\text{Stream Power} = \tau_0 v \qquad [1]$$

where τ_0 = $v_*^2 \rho$ (local bed shear stress, N/m^2) [2]
v = local depth-averaged velocity (m/s)
v_* = local bed shear velocity (m/s)
ρ = density of water (assumed to be 1000 kg/m^3)

This relation has been shown to work well for natural streams; however, there has been noted difficulty in the case of large rivers (Simons and Şenturk, 1992). Under each of the three flow conditions (mean annual, calibration, and 1:100-year flow), the calculated stream power, given the mean bed material size of 0.28 mm, suggests that anti-dunes may form.

Froude Number and Hydraulic Depth

The next set of empirical relationships, developed by Athaullah (1968), are meant to be used together to best predict bed form type. The first relationship is a function of the Froude number, *Fr*, and the ratio of hydraulic mean depth (area of the cross-section divided by the top width of the channel) to the mean bed material size, R/D_{50}. The Froude number, Fr, is defined by the following:

$$Fr = \frac{v}{\sqrt{gD}} \qquad [3]$$

where g = acceleration due to gravity (assumed to be 9.81 m/s^2)
D = local water depth (m)

The Froude number represents a ratio of inertia and gravitational forces on the fluid, while R/D_{50} corresponds to the relative roughness (Simons and Şenturk, 1992). Under mean annual and calibration flow conditions, this relationship proposes that the flow is in the so-called lower regime. This relationship does not encompass the conditions present at the 1:100-year flow, so no conclusion about regime can be made for this flow scenario.

Slope and Shear Intensity Factor

The second relationship of the set is a function of the slope of the energy gradient and the shear intensity factor. The shear intensity factor, Ψ, is defined by the following equation:

$$\psi = \frac{v_*^2}{\left(\frac{\rho_s - \rho}{\rho}\right) g D_{50}} \qquad [4]$$

where ρ_s = density of sand (assumed to be 2650 kg/m^3)
D_{50} = mean bed material size (m)

The slope of the energy gradient represents the effect of form roughness on energy dissipation, while the shear intensity factor quantifies to the ability of the fluid to move sediment (Simons and Şenturk, 1992). Under each of the three flow conditions (mean annual, calibration and 1:100-year flow), this relationship suggests that the flow may be in a transitional state where dunes or anti-dunes may form.

Ratio of Bed Shear Velocity to Settling Velocity and Shear Velocity Reynolds Number

Several dimensionless bed form and flow regime discriminators were also examined. The first of these was a function of the ratio of bed shear velocity to settling velocity and the shear velocity Reynolds number, proposed by Liu (1957) and later extended by Simons and Richardson (1961). The particle settling velocity, v_s, as determined by Soulsby (1997) is as follows:

$$v_s = \frac{\nu}{D_{50}} \left[\left(0.36^2 + 1.049 D_*\right)^{1/2} - 10.36 \right] \qquad [5]$$

where ν = kinematic viscosity (assumed to be 1 x 10^{-6} m^2/s)

$$D_* = D_{50} \left[\frac{g\left(\frac{\rho_s - \rho}{\rho}\right)}{\nu^2} \right]^{1/3} \quad \text{(dimensionless particle parameter)} \qquad [6]$$

The shear velocity Reynolds number, Re*, is defined by the following equation:

$$R_e^* = \frac{v_* D_{50}}{\nu} \qquad [7]$$

Few field data were incorporated into Simons and Richardson's analysis; however, it may be used to make predictions of bed forms in flow depths of up to 3 m (Garcia, 2008). This is noted, as in the case of the 1:100-year flood, flow depths exceed 3 m, making the analysis invalid. Under mean annual and calibration flow conditions, this relationship suggests that anti-dunes may form.

Dimensionless Particle Diameter and Shear Velocity Reynolds Number

Another dimensionless relationship examined to predict bed forms is a function of the dimensionless particle diameter and the shear velocity Reynolds number, proposed by Bonnefille-Pernecker (after Bechteler *et al*, 1991). This relationship has been used to analyse sediment transport conditions in alpine rivers (Garcia, 2008). Under mean annual-flow conditions, this relationship suggests that a flat bed will form. Under the calibration and 1:100-year flow conditions, this relationship indicates that anti-dunes will form.

Froude Number and Ratio of Depth to Mean Particle Diameter

A bed form chart for medium sand developed by Vanoni (1974) was explored. This relationship of bed form to Froude number, *Fr*, and the ratio of water depth to sediment size, D/D_{50}, can provide reasonable estimates of bed forms for a flow depth up to approximately 3 m (Shen and Julien, 1993). This is noted, as in the case of the 1:100-year flood, flow depths exceed 3 m, making the analysis invalid. Under mean annual and calibration flow conditions, this relationship suggests that dunes may form.

Transport Stage Parameter and Mean Particle Diameter

Van Rijn (1984) proposed a relationship in which the transport stage parameter and dimensionless particle diameter can be used to predict bed form types. The transport stage parameter, T, is defined as:

$$T = \frac{\tau_s^* - \tau_c^*}{\tau_c^*} \qquad [8]$$

where $\tau_s^* = \dfrac{\tau_0}{\rho g \left(\dfrac{\rho_s - \rho}{\rho}\right) D_{50}}$ (bed shear stress due to skin/grain friction) [9]

$$\tau_c^* = \frac{1}{2}\left[0.22\left(D_{50}\frac{\sqrt{g\left(\frac{\rho_s - \rho}{\rho}\right)D_{50}}}{\nu}\right)^{-0.6} + 0.06\exp\left(-17.77\left(D_{50}\frac{\sqrt{g\left(\frac{\rho_s - \rho}{\rho}\right)D_{50}}}{\nu}\right)^{-0.6}\right)\right] \qquad [10]$$

(critical shear stress for motion based on the Sheild's diagram, as proposed by Brownlie, 1981)

Both laboratory experiments and field data were used to create this relationship. However, it must be noted that there have been reservations about the applicability of this relationship to large alluvial rivers where the Froude number is never larger than 0.2 to 0.3, as dunes have been found to exist where the relationship predicts that anti-dunes will form (Garcia, 2008). Under each of the three flow conditions (mean annual, calibration and 1:100-year flow), this relationship proposes that anti-dunes may form. Note here that the Froude number ranges from 0.21 to 0.23 for the various flow conditions, indicating that this relationship may not be adequate.

Grain Froude Number and Slope

Using the grain Froude number and slope, Brownlie (1983) created a relationship delineating the bed form transition zone from lower regime to upper regime. The grain Froude number, Fg, is defined as:

$$Fg = \frac{v}{\sqrt{g\left(\frac{\rho_s - \rho}{\rho}\right)D_{50}}} \qquad [11]$$

This analysis was based on both flume and river data (Garcia, 2008). Under mean annual and calibration flow conditions, this relationship suggests that flows proposed at the outfall/diffuser site will fall into the lower regime. However, for the 1:100-year flow, this relationship suggests that flows will be in the upper regime.

Mobility Parameter and Dimensionless Particle Diameter

The last relationship investigated was one proposed by van den Berg and van Gelder (1993). Bed forms are predicted as a function of the mobility parameter and the dimensionless particle diameter. The mobility parameter, θ', (van Rijn, 1984) is defined as follows:

$$\theta' = \frac{v^2}{g\left(\frac{\rho_s - \rho}{\rho}\right)D_{50}\left(C'\right)^2} \qquad [12]$$

where C' = $18\log\frac{4D}{D_{90}}$ (coefficent) [13]

D_{90} = sediment size for which 90% is finer (m)

This analysis was based on both flume and field observations (Garcia, 2008). Under each of the three flow conditions (mean annual, calibration and 1:100-year flow), this relationship suggests that ripples may form.

APPENDIX D

Detailed Bed Form Size Analysis

Appendix D
Detailed Bed Form Size Analysis

Allen (1963)

The first set of equations used was proposed by Allen (1963). Based on physical reasoning and analysis of observed data, the equations are as follows:

$$\log D = 0.8271 \log A + 0.8901 \qquad [14]$$

where D = local water depth (m)
A = dune height (m)

and,

$$\log A = 0.7384 \log L - 1.0746 \qquad [15]$$

where L = dune length (m)

Yalin (1964)

The second set of equations used was proposed by Yalin (1964). This set of equations is based on the study of geometric properties of ripples and dunes through the analysis of mechanical processes. The equations proposed are as follows:

$$\frac{A}{D} = \frac{1}{6} \qquad [16]$$

and, for large values of the Reynolds number, R_e^*:

$$L = 5D \qquad [17]$$

Goswami (1967)

Goswami (1967) presents an equation where dune height is a function of length. The ratio of mean flow depth to dune height is obtained from a graph that relates it with slope and mean particle diameter. Once this ratio is determined, the following relationship can be used:

$$A = 0.055 L^{0.87} \qquad [18]$$

Julien and Klaassen (1995)

Based on a number of laboratory experiments and field data, Julien and Klaassen (1995) propose a set of equations to estimate dune size. These are as follows:

$$\frac{L}{D} = 6.25 \qquad [19]$$

and

$$A = 2.5D\left(\frac{D_{50}}{D}\right)^{0.3} \qquad [20]$$

where D_{50} = mean bed material size (m)

Karim (1999)

Finally, the last set of equations examined was that of Karim (1999). This approach considers that energy loss due to form drag is related to the head loss across a sudden expansion in an open channel. Applicable to ripples, dunes, transition and anti-dunes, the equation has been proven to work well in laboratory settings; however, it does not fully capture the results of field data (Garcia, 2008). The equations are as follows:

$$\frac{A}{D} = \left[\frac{\left\{S_e - 0.0168\left(\frac{D_{50}}{D}\right)^{0.33} Fr^2\right\}\left(\frac{L}{D}\right)^{1.20}}{0.47 Fr^2}\right]^{0.73} \qquad [21]$$

where S_e = energy slope (m/m)
Fr = Froude number (dimensionless)

and, for dunes:

$$\frac{L}{D} = 6.25 \qquad [22]$$