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Updated Hydrotechnical Information for the Mackenzie River Bridge at Fort Providence

Prepared for:
JIVKO Engineering

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EXECUTIVE SUMMARY

The Fort Providence Combined Council Alliance, a consortium of local groups, is proposing the construction of a permanent bridge over the Mackenzie River at the present location of the Fort Providence ferry crossing. Trillium Engineering and Hydrographics Inc. was contracted to provide a hydrotechnical analysis of river flows, ice jam levels, ice forces, and pier scour to identify the main hydraulic issues and to support the initial preliminary design. This work was completed in November, 2002 and the results and supporting information are contained in the main body of the report.

High water levels at the crossing site are mostly a result of ice jams, and defensible ice jam design levels could be developed on the basis of recent field data collected at the site by various agencies. Ice forces on the bridge piers and scour around the piers depend on both the hydraulic conditions and the characteristics of the bridge structure. The investigation into the potential ice effects study found that ice forces on the bridge piers would be a significant concern both because of the extreme ice thicknesses that can occur upstream in Beaver Lake and the high water levels that can develop due to ice jams during breakup. A potential scour depth of 3 to 5 m was estimated, but the impacts of this depth were offset by the type of piers selected in the preliminary design. Furthermore, it was possible that relatively scour- resistant clay below the thin layer of alluvium in the river bed could reduce the amount of scour. Site specific bed material sampling was recommended as part of the foundation investigation to clarify the potential positive effects of the clays and to confirm the distribution and depth of the alluvium.

Following review of the preliminary crossing option and assessment of the results of the hydrotechnical evaluation, a number of alternative designs were proposed and the effects of various arrangements of the bridge substructure on ice loads and scour depths were evaluated. In January, 2004, Trillium completed an evaluation of ice loads and scour for the adopted pier-on -slab concept of the current crossing alternative. This evaluation is contained in Appendix A.

Changes to the pier layout to accommodate superstructure modifications created a situation where the angle at which ice floes approached the piers became an issue in the ice load design. Further investigation using a two-dimensional numerical flow model to estimate the ice floe approach angles indicated that the ice could strike the piers at angles of attack of up to 19° and that this angle varied across the channel. Since the ice crushing forces were potentially quite high, the expected ice loads were reduced by adopting a pier geometry which induced bending failures over a range of approach angles. This ultimately resulted in a wider pier.

The wider pier in turn increased the expected depth of scour around the base of the pier. Analysis indicated that, although the clay bed was probably resistant to scour at the shear stresses generated by the piers, the occurrence of fatally significant scour could not be ruled out completely. Therefore, to be prudent, it was recommended that the river bed around the piers at least be protected with a riprap armor layer, although it would be preferable that the piers be

founded on piles, to provide a measure of additional protection against scour. The use of riprap to protect the bed will require monitoring to ensure that the armor layer continues to provide the required scour protection and that it is repaired time a timely fashion if it is found to be damaged.

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November 25, 2002

JIVKO Engineering
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Attention: Mr. Jivko Jivkov, P. Eng.

RE: LETTER OF TRANSMITTAL

Dear Sir:

Following is our report containing the updated hydrotechnical information for the proposed Mackenzie River bridge at Fort Providence.

Ice is a significant concern for the design of the proposed bridge. The ice forces are quite high due to the extreme ice thicknesses that can occur upstream in Beaver Lake. The minimum height of the bridge is governed by the elevation to which ice can shove rather than the level of ice jams. Ice shoving onto the roadway is also a concern, so it is recommended that the low portions of the roadway on the approach fills be protected by a vertical barrier which will deflect the ice floes and stop them before they reach the roadway.

Scour is not expected to be a major problem at the proposed bridge site. Application of existing scour formulae suggest scour depths of 3 to 5 m due to the fine textured surface material at the bridge site. This is unlikely to occur due to the relatively scour resistant clay below the thin layer of alluvium in the river bed. Site specific bed material sampling is recommended as part of the foundation investigation in order to determine the grain size distribution and the depth of the alluvial material at the locations of the piers and abutments.

If you have any questions or comments about this report call me at 496-7671.

Sincerely,

Gary Van Der Vinne, M.Sc., P.Eng.
Trillium Engineering and Hydrographics Inc.

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1.0 INTRODUCTION

1.1 Background

Highway 3, which connects Yellowknife to the south, crosses the Mackenzie River at Fort Providence (Figure 1.1 and 1.2). At present, the river is crossed by either a ferry during open water or an ice bridge in winter. Both the ferry and the ice bridge incur significant annual operational costs. As well, this system has interruptions in the spring and in early winter when the ice bridge is not passable and the ferry cannot safely be used. Construction of an permanent bridge over the Mackenzie River would eliminate these issues.

In February, 2002, Andrew Gamble & Associates released a feasibility study for a bridge over the Mackenzie River. This study was commissioned by the Fort Providence Combined Council Alliance, a consortium of local groups advocating the construction of a permanent bridge. The study proposed a bridge over the Mackenzie River at the location of the ferry crossing (Ferry). The proposed design is a 1080 m long bridge supported by eight piers with twin, circular shafts that are 2.75 m in diameter and spaced 7.0 m apart (Figure 1.3). The piers are spaced 115 m apart, with a 200 m main span centered over the recommended navigation track in the river.

The technical components of the feasibility study were reviewed by at team led by BPTEC Engineering Group Ltd.. The review identified a need to re-examine and update the available hydrotechnical information at the site. In order to address the concerns of the review, Jivko Engineering engaged Trillium Engineering and Hydrographics Inc. to undertaken this work.

A number of previous studies have been carried out to assess the hydrotechnical characteristics at the site. Structural Engineering Services carried out a preliminary assessment of the crossing in 1958. In this study foundation, scour and ice characteristics were assessed. In 1975, Northwest Hydraulics Consultants Ltd. reassessed the scour and ice characteristics of the site and provided design recommendations. As part of a scope of work to optimize winter ferry operations, the University of Alberta carried out a four year study of breakup processes in the years from 1992 to 1995.

1.2 Objectives

The scope of this study is to update the existing hydrotechnical information at the Ferry to finalize the design of a bridge at the site. This hydrotechnical information consists of open water hydraulic characteristics, ice characteristics, and scour effects; and it will be examined to define the design high open water level, the design ice jam water level, ice forces on the bridge piers, and bed scour at the piers and within the bridge waterway.

The open water hydraulic component of the work will (1) examine the Great Slave Lake water levels and determine effects that wind conditions have on setup in the lake and the subsequent discharge, water levels, and current speeds at the bridge site; (2) develop a hydraulic framework for the reach between Great Slave Lake and Mills Lake to update the rating curve at the Water Survey of Canada (WSC) gauge; (3) update the WSC rating curve at the proposed bridge; (4) review the hydrologic records and update the discharge frequency curve and flow duration curves to reflect the post-1975 information; and (5) establish the hydraulic characteristics in the bridge waterway to assist in assessing navigation and scour issues.

The ice characteristics component of the work will (1) review historical regional ice thickness measurements and update the 1975 estimate of design thickness; (2) estimate ice strengths within the context of breakup observations carried out in the early 1990's; and (3) determine expected high ice levels within the context of the hydraulic framework on the basis of equilibrium ice jam theory and expected outflows from Great Slave Lake.

The scour component of the work will (1) estimate design scour depths on the basis of the hydraulic characteristics of the bridge waterway, the reported bed material characteristics, and the design high open water discharge; and (2) determine the need for scour protection at the abutments and provide recommendations for the appropriate rock sizes, apron widths and side slopes.

2.0 OPEN WATER HYDRAULIC CHARACTERISTICS

The proposed bridge crossing site on the Mackenzie River near Fort Providence lies approximately 65 kilometers downstream of the outlet of Great Slave Lake at the Ferry. The drainage area of the Mackenzie River at this location is approximately 980,000 km^2. Only one minor tributary enters the Mackenzie River between the bridge site and the lake so virtually all of the flow originates from Great Slave Lake. Drainage into Great Slave Lake originates from basins that drain most of northern British Columbia, Alberta and Saskatchewan, and also the southern and central portions of the Northwest Territories. The largest tributaries to the lake are the Peace and Athabasca Rivers via the Slave River, and the Talston and Hay Rivers. These rivers account for almost 60% of the drainage area of the Mackenzie River near Fort Providence, half of which comes from the Peace River basin alone. Flows in the Mackenzie River have been affected by regulation of the Peace River in northern British Columbia since 1968.

Due to the proximity of Great Slave Lake to the bridge site, flows at the bridge site are determined by the water levels in the lake. Storage in the lake causes peak outflows to be reduced and low outflows to be increased relative to inflows into the lake. Typically, maximum flows occur in July. The maximum recorded flow of 10,400 m^3/s is slightly less than twice the mean annual flow of 5,320 m^3/s. Minimum flows occur during the late winter in March or April. The minimum recorded flow of 1,040 m^3/s is only one-quarter of the mean annual flow. In general, minimum flows are produced in winter because the ice cover in the river below the outlet reduces the outflow from the lake. It should be noted that the minimum recorded historical flow occurred when the Peace River flows were reduced due to the filling of the Williston Reservoir after the completion of the Bennet Dam.

2.1 Great Slave Lake Water Levels

Great Slave Lake water levels are measured by WSC at five locations. Of the gauges at these locations, Great Slave Lake at Yellowknife Bay (07SB001) and Great Slave Lake at Hay River (07OB002) are most applicable to assessing the flows at the proposed bridge. The Hay River gauge is closest to the Mackenzie River but the Yellowknife Bay gauge has the longest and most consistent record. At Yellowknife Bay the period of record is from 1934 to present while at Hay River it is from 1959-1970 and 1983 to present.

Mean monthly water levels for these stations published by WSC are summarized in Table 2.1. Mean monthly water levels are highest in the month of July with levels of about 156.8 m. The lowest lake levels are observed in November and December with levels of about 156.5 m. It is evident that, on the average, there is very little difference in the monthly water levels.

Daily water levels at Yellowknife Bay have ranged from 156.09 m to 157.26 m. The highest maximum instantaneous water level was 157.34 m recorded on September 26, 1996. Daily water levels at Hay River have varied from a minimum of 156.04 m to a maximum of 157.38 m. The highest maximum instantaneous water level was 158.10 m on May 6, 1989; however, this water level was probably affected by breakup in the Hay River which occurred during the same period. Figure 2.1 which shows water levels at Hay River and Yellowknife Bay for 1989, illustrates this point. The daily water level at Hay River rose and fell by about 0.35 m over a five day period while the Yellowknife Bay water level remained more or less constant. Also, the maximum instantaneous water level of 158.10 m at Hay River was 1.24 m higher than the water level at Yellowknife Bay. This peak water level coincided with breakup on the Hay River, thus it is likely that the breakup in the river had affected the gauge level. The gauge record also shows rapid increases of up to 0.50 m in the mean daily water levels in the other years during the breakup period.

Table 2.1 Great Slave Lake mean water levels

Period	Yellowknife Bay	Hay River[1]
January	156.531	156.514
February	156.555	156.548
March	156.570	156.574
April	156.570	156.587
May	156.630	156.628
June	156.754	156.729
July	156.814	156.776
August	156.808	156.765
September	156.720	156.667
October	156.612	156.563
November	156.523	156.472
December	156.499	156.480
Annual	156.640	156.599

[1] includes effects of datum shift

A comparison of Yellowknife Bay and Hay River daily water levels shows that in the period of 1959-1970, when discharge measurements were being carried out at Fort Providence, water levels at Hay River were generally 0.28 m lower than those at Yellowknife Bay (Figure 2.2). During the period of 1995-1997 when discharges were again being reported at Fort Providence, water levels at Hay River tended to be about 0.10 m higher than those at Yellowknife Bay. This discontinuity in datum, combined with the fluctuations during breakup at Hay River, makes the Yellowknife Bay gauge a more reliable measure of the water level in Great Slave Lake for the purposes of predicting discharges at Fort Providence.

2.2 Channel Characteristics

The site of the proposed bridge is located at the present ferry crossing in a reach of the Mackenzie River between Beaver Lake and Meridian Island (Figure 1.2). This reach is bounded on the upstream side by a wide section of the river known as Beaver Lake and on the downstream side by a split in the river as it flows around Meridian Island. The main channel of the Mackenzie River flows north of Meridian Island through the Providence Rapids. The Big Snye channel on the south side of the island serves as a by-pass channel during high stage events.

River geometry for the Mackenzie River between Great Slave Lake and Mills Lake was obtained by using cross-sections surveyed by the University of Alberta in 1992. These sections were supplemented by additional cross-sections surveyed by Trillium Engineering in 2002 (Figure 2.3). The supplemental sections provided additional geometry information for the ferry crossing and for the by-pass channels at Meridian Island. The locations of these sections can be seen in Figure 1.2

A one-dimensional hydraulic model of the channel network was developed using the HEC-RAS computer program. The channel network included the Big Snye and Little Snye channels as well as the main channel. The Big Snye diverts flow from the main channel upstream of Providence Rapids at high flows. Some of this diverted flow returns to the main channel downstream of the Providence Rapids through the Little Snye with the remainder flowing into Mills Lake via the Big Snye. The locations of these channels are shown in Figure 1.2.

The flow split at the Big Snye affects water levels at the bridge by reducing the discharge and water level in the main channel downstream of the split. This effect is especially important when water levels are high during ice jam events. During open water, the Big Snye carries about 5% of the total flow at low flow and up to 12% of the total flow at high flows. During the winter, very little flow occurs in the Big Snye but when ice jams form in the Providence Rapids, the Big Snye can carry as much as half of the total flow. The Little Snye returns about 10% of the diverted flow back to the main channel below the rapids.

Water surface profiles surveyed on August 29, 1991 and July 11, 1992 (Hicks et al., 1992) were used to calibrate a one-dimensional hydraulic model of the reach using the HEC-RAS computer program. The hydraulic model simulated the main channel as well as the Big Snye and Little Snye. The open water Manning roughness ranges between 0.020 and 0.022 except in the rapids section in the main channel at Meridian Island where it is 0.030. The calculated water surface profiles are compared to the measured water levels in Figure 2.4.

The reach containing the proposed bridge crossing is about 7 km in length and has a width of 1100 to 1600 m. Typical cross-sections in this reach are shown in Figure 2.3, including the constricted section at the Ferry. These cross sections show that there is a deep channel 100 to 400 m wide which meanders within a generally wide and shallow river section. The deep channel tends to be wider at the Ferry. At the open water median flow (6,600 m^3/s), the mean depth is about 3.0 m while maximum depths are about 7.2 m. The average channel velocity is 1.7 m/s. The mean bed slope through the reach is approximately 0.33 m/km.

The river bed is composed of clay till overlain by a thin layer of alluvial material. The alluvial material is composed of sand, gravel, cobble, and even small boulders. The alluvial layer is up to 1.8 m thick in some places but in other places the clay is exposed.

2.3 Rating Curves

Rating curves for open water conditions were constructed using the summaries of discharge measurements made by WSC. These summaries include discharge measurements taken at both Dory Point and the Ferry. This data is shown in Figure 2.5 along with the present WSC rating curve at the Ferry (rating table number 3). Rating curves were also developed for these two locations using HEC-RAS, as calibrated from the open water profiles of August 29, 1991 and July 11, 1992. The HEC-RAS rating curve at the Ferry is within 0.05 m of WSC rating. This difference is within the accuracy of HEC-RAS and confirms the applicability of the hydraulic geometry model used in the simulation. The HEC-RAS analysis shows a shift of about -0.69 m in the rating curve from Dory Point to the Ferry.

The relationship between discharges recorded on the Mackenzie River near Fort Providence and Great Slave Lake water levels at Yellowknife Bay is shown in Figure 2.6. Discharge varies by about 1,000 m^3/s for a given water level; likely due in part to wind effects causing higher or lower water levels at the outlet. The simulated lake outflow rating curve is also shown in Figure 2.6. An approximate lower bound of the data can be defined by shifting the lake outflow rating curve by 0.06 m. This curve representing the lower bound of the data is appropriate for calculating extreme values of discharge from the Yellowknife Bay water levels because it provides an upper limit of discharge, rather than a mean discharge, for a given water level. The lower rating curve was used to estimate discharges on the basis of Great Slave lake water levels to extend the period of record of annual maximum discharges (see discharge frequency analysis presented in Section 2.5.2).

2.4 Wind Effects

Great Slave Lake water levels are affected by wind conditions over the lake. According to the 1961 to 1990 climate normals for Hay River, the prevailing wind direction varies from the northeast to south during the summer months and changes to the west during the early winter. The mean wind speeds are between 11 and 14 km/hr with no discernable seasonal variation. The most frequent wind direction during the year is from the east with a mean speed of 13 km/hr. Extreme hourly wind speeds range from 50 to 90 km/hr with the highest speeds from the north and northwest. The mean extreme hourly wind is about 60 km/hr.

For the outlet location in the southwest corner of the lake, the fetch varies from 25 to 200 km depending on the prevailing wind direction. Using a fetch of 150 km and a mean lake depth of 60 m, a prevailing east wind of 13 km/hr would produce a wind setup of less than 0.01 m at the mouth of the Mackenzie River. By comparison, a extreme hourly east wind of 60 km/hr would produce a wind setup of 0.15 m. This setup could increase the flow in the river by about 500 m^3/s.

The effect of wind on the river water levels at the Ferry was analyzed by adding the shear stress due to wind blowing over the water surface to shear stress due to the water flowing over the bed. The wind shear stress is a function of the square of velocity as well as the drag coefficient. Wind setup measurements on lakes suggest that the drag coefficient of air over a water surface is about 1.4×10^{-3}, thus a 60 km/hr wind will produce a shear stress of about 0.5 N/m^2 on the water surface. This shear stress is about 5 to 15% of the typical bed shear stress in the natural channel. Thus, if a 60 km/hr wind blows down the channel, water levels may drop as much as 0.40 m. Conversely, if a wind of this magnitude blows against the current, the water level may rise by as much as 0.4 m.

The long fetch lengths of the river at Beaver Lake makes it possible for significant wind induced waves to develop on the water surface. The fetch length upstream of the Ferry is as great as 45 km in the east-south-east direction. A wind of 60 km/hr in this direction may produce waves of about 0.9 m in height.

2.5 Discharge

Hydrometric records for the Mackenzie River near Fort Providence (10FB001) are available for the years 1958-1998 from WSC. Between 1958 and 1992 the gauge was located approximately 1.5 km upstream of the Ferry, near Dory Point. On September 30, 1992 the gauge was relocated downstream to the Ferry. Discharge records are incomplete over the period of record with records available for the years 1961-1978 and 1995-1997. Water level records are more complete and are available for the years 1962-1996, and 1998. Miscellaneous discharge and water level records are also available for the years 1958-1961.

The reported mean daily discharges for the years 1962-1978 and 1995-1997 are plotted in Figure 2.7. The figure shows that flows range between 1,000 and 3,000 m^3/s from January to mid-April. Flows begin to increase in mid-April or early May as the ice melts in the river below the outlet of Great Slave Lake. Flows in the open water period, from June to the beginning of November, range between 4,000 and 9,000 m^3/s. Peak flows generally occur in June or July when lake levels are highest. Flows decrease sharply in November as the river below the lake outlet freezes over, falling to between 2,000 and 3,500 m^3/s by the beginning of December. From December to April there is a continuing gradual decrease in flows.

2.5.1 Flow Duration

Discharge and water level records from Dory Point and the Ferry were combined to produce flow duration curves. Annual, winter, and open water flow duration curves are shown in Figure 2.8 and the data is summarized in Table 2.2. The annual and winter flow durations were calculated from reported discharges alone, because there was no reliable method of determining flows from ice affected water levels. The reported open water flows were augmented by converting the reported water levels to flows using the rating curves shown in Figure 2.5. The open water season was defined as June 15 to October 15 where records did not indicate the actual ice free period.

During the open water period the discharges at Fort Providence range from 3,170 to 10,400 m^3/s with a median flow of 6,600 m^3/s. During the winter period the discharges are much lower - ranging from 1,160 to 4,100 m^3/s with a median flow of 2,140 m^3/s. Annual daily mean discharges range from a maximum of 10,400 m^3/s (based on the open water record) to a minimum of 1,040 m^3/s. The median annual flow is 5,320 m^3/s.

The flow durations have been affected by hydro-power development on the Peace River in northern British Columbia. Discharge in the period of 1968 – 1972, during which time Williston Lake reservoir was being filled, represent the minimum flows on record. The flow duration curves would be 100 to 300 m^3/s higher over their entire range if this period was not included in the historical data set.

Table 2.2 Flow duration for the Mackenzie River near Fort Providence

Percentage of Time Exceeded	Daily Mean Discharges (m^3/s)		
	Annual	Winter (Dec. to Mar.)	Open water[1] (Jun. 15 to Oct. 15)
Maximum	8,950	4,100	10,400
5%	8,310	2,970	8,610
10%	7,905	2,800	8,300
20%	7,250	2,640	7,730
50%	5,320	2,140	6,600
80%	2,310	1,510	5,410
90%	1,650	1,360	4,870
95%	1,420	1,300	4,520
Minimum	1,040	1,160	3,170

[1] Maximum open water flows are higher than annual flows because the open water flow record has been extended to include water level records. This could not be done for ice affected flows so the winter and annual flows have a shorter record length.

2.5.2 Flow Frequencies

The record of annual extreme discharges reported by WSC at Fort Providence contains only seven annual maximum instantaneous and 13 annual maximum daily discharge events for the period 1964 to 1997. The maximum instantaneous discharges are only 1% higher than the maximum daily discharge – that is they are virtually identical. When the daily and instantaneous data were combined, the period of record is extended to 15 events. However, this length of record is still insufficient to reliably predict a 100 year discharge.

The 15 year record of peak discharges reported by WSC was extended to 37 years by incorporating the peak water level record. Water levels recorded at the Fort Providence gauge were converted to mean daily discharges using rating curves shown in Figure 2.5. The discharges were then converted to maximum instantaneous discharges using the ratio of 1.01. It should be noted that the water levels used in this analysis were limited to the open water season because the rating curves are not valid during the ice affected period. The maximum discharges obtained from this analysis will not include annual maximums which may have occurred before June 15.

The 37 year record of peak discharges and water levels reported by WSC was extended to 63 years by incorporating the Great Slave Lake water level record. Lake levels recorded at the Yellowknife Bay gauge were converted to discharges at the Ferry using the lower bound lake outlet rating curve shown in Figure 2.6. The discharges obtained from this curve match the known peak discharges whereas the simulated outlet rating curve under-predicted the extreme values. The simulated rating curve was fit to the average water level for a given discharge, however extreme discharges would tend to occur when wind setup in the lake caused water levels to be higher at the outlet than they were at the Yellowknife Bay gauge. These lake level derived peak flows provided 26 years of additional annual peak flows.

The frequency distribution of the maximum instantaneous discharges are shown in Figure 2.9. Normal distributions are presented for the reported discharges, for the extended record that includes discharges derived from the mean daily water levels measured at Fort Providence, and for the complete extended record that includes the additional discharges derived from the Great Slave Lake water levels at Yellowknife Bay. The three distributions are quite similar although the record that includes the lake levels reduces the importance of the two highest flows of 10,400 m^3/s that are derived from the Fort Providence water level record.

The results of the flow frequency analysis for the extended record are summarized in Table 2.3. The estimated flow frequencies range from a 100-year discharge of 10,500 m^3/s to a 2 year discharge of 7,840 m^3/s. This 100-year discharge value is only 34% greater than the 2-year discharge. This small spread in discharge reflects the regulatory effect that Great Slave Lake has on the flows in the Mackenzie River near Fort Providence. The 100-year discharge of 10,500 m^3/s is adopted as the design open water flow.

Table 2.3 Flow frequencies for the Mackenzie River near Fort Providence

Return Period (years)	Maximum Instantaneous Discharge (m^3/s)	Maximum Open water Elevation (m)
100	10,500	152.26
50	10,200	152.18
20	9,730	152.07
10	9,310	151.97
5	8,810	151.84
2	7,840	151.61

2.6 Bridge Hydraulics

The proposed bridge crossing is situated at the existing ferry crossing which is located approximately at the mid-point of the Beaver Lake-Meridian Island reach. The layout and plan of the proposed bridge are shown in Figure 1.3. The proposed bridge is 1080 m long and is supported by eight piers with twin, circular shafts that are 2.75 m in diameter and spaced apart by 7.0 m on centre. The piers are spaced 115 m apart, with a 200 m main span centered over the recommended navigation track in the river. The proposed bridge is perpendicular to the deep water channel but is skewed about 10° LHF to the banks.

The bridge approaches and abutments utilize the existing ferry landings, however the bridge waterway is offset from the 970 m wide ferry waterway in order to position the center of the bridge over the navigation track. The proposed bridge alignment results in the south abutment encroaching approximately 50 m further into the river than the present ferry landing while the north abutment is set back about 130 m from the end of the existing ferry landing.

The top width of the bridge waterway will be about 1050 m at the design discharge of 10,500 m^3/s and water level of 152.44 m. This width is greater than the constricted top width of 970 m produced by the existing ferry landings thus backwater effects and velocities will be slightly less than the present conditions at the existing ferry landings. Outflows from Great Slave lake will not be reduced because of the bridge. The mean and maximum flow depths (neglecting any scour) are 5.6 and 7.9 m respectively at the design discharge. The design mean velocity through the bridge will be 1.8 m/s.

2.7 Navigation

Navigation is affected by water levels and velocities. High water levels reduce the navigation clearance below the bridge superstructure while low water levels reduce the clearance above the bed. High velocities through the bridge waterway may also impede navigation.

Water level durations in the bridge waterway are summarized in Table 2.4. These water levels were derived from the open water discharge durations given in Table 2.2. The water level in the bridge waterway corresponding to the maximum recorded discharge during the open water period is 152.34 m. This is slightly lower than the 100-year open water elevation of 152.44 m given in Table 2.3. The minimum open water level is 149.85 m. The water level is between 150.46 m and 151.87 m 90% of the time.

Average velocities through the bridge waterway were also calculated for the open water discharge durations. These values are summarized in Table 2.4. Average channel velocity varies from 0.98 to 1.78 m/s. Maximum velocities associated with the maximum depth in the cross section are also summarized in Table 2.4. Maximum channel velocity varies from 1.39 to 2.25 m/s.

The water levels summarized in Table 2.4 are based on calm conditions. Wind effects such as those discussed in Section 2.4 will cause water levels to rise or fall by as much as 0.4 m. This process occurs in the existing channel and is not affected significantly by the proposed bridge geometry.

The proposed bridge provides for a high clearance navigation track below the main span. The bottom chord elevation of the main span is a maximum of 178.76 m in the centre of the span but is only 174.55 m at a locations 40 m from each of the main span piers. These locations define the limits of the navigation track. With these bottom chord elevations, the proposed bridge would provide a minimum clearance of at least 22.68 m for 95% of the open water season. At the 100-year water level, the clearance would be 22.21 m.

Table 2.4 Water level and velocities for various flow durations in the proposed bridge waterway

Percentage of Time Exceeded	Open Water Discharge (m^3/s)	Open Water Elevation (m)	Minimum Clearance (m)	Mean Velocity (m/s)	Maximum Velocity (m/s)
Maximum	10,400	152.34	22.21	1.78	2.25
5%	8,610	151.87	22.68	1.61	2.06
10%	8,300	151.76	22.79	1.59	2.04
20%	7,730	151.46	23.09	1.54	2.00
50%	6,600	151.20	23.35	1.42	1.87
80%	5,410	150.81	23.74	1.28	1.72
90%	4,870	150.59	23.96	1.22	1.65
95%	4,520	150.46	24.09	1.17	1.60
Minimum	3,170	149.85	24.70	0.98	1.39

3.0 ICE CHARACTERISTICS

Local ice characteristics have significant impacts on the design of bridges in northern climates. Where ice jams occur, peak breakup water levels due to ice jams are typically higher than peak open water levels. The ice-related water levels often are used to determine the minimum bridge elevation. As well, ice thickness and its attendant strength play a significant part in generating ice loads on piers. Breakup processes also effect the elevation at which the ice load is applied. Secondary breakup processes such as ice shoves can lead to even higher levels than those produced by ice jams. Should ice shoving be a significant process at a given location there is a need to consider the implications of this process.

3.1 Breakup

Breakup in the Mackenzie River in the vicinity of Fort Providence tends to follow a regular pattern. As the solar radiation and air temperature increase during the spring period, a region of open water forms at the outlet of Great Slave Lake due to thermal processes and as the backwater effects of the ice in the river lessen, the discharge in the river begins to increase. This increase in discharge, combined with the deterioration of the ice cover due to solar radiation, promotes a dynamic breakup in the vicinity of Fort Providence. The first movement of the ice sheet typically occurs in the upper portion of Providence Rapids where the hydraulic forces are large and the ice is thin. This ice shoves downstream, dislodges the thicker ice in the lower portion of Providence Rapids, and forms an ice jam in the lower portion of the rapids.

The ice jam in the lower portion of Providence Rapids produces a backwater effect at the Ferry which lifts and cracks the ice. The ice in this reach then breaks into large floes which drift through the open water reach upstream of the ice jam and accumulate against the head of the ice jam. This accumulation typically extends through the rapids but not into the Ferry reach. Short jams of this type produce many of the typical peak breakup water levels at the Ferry.

As thermal melting continues, and the open water at the outlet to Great Slave Lake extends downstream into of Beaver Lake, the ice cover in Beaver Lake tends to break into large floes. These large floes accumulate at the downstream end of Beaver Lake until the ice floes are flushed into the narrower Ferry reach as the flow increases and the ice deteriorates. Typically, the channel downstream is sufficiently free of ice to pass this ice run and no significant ice jamming develops. These ice runs produce peak water levels at the ferry which are similar to those produced by the short jams in Providence Rapids.

Occasionally, a jam is still in place in Providence Rapids when the Beaver Lake ice begins to run. In this case, the Beaver Lake ice accumulates at the head of the jam and increases the length of the jam so that it extends into the Ferry reach. The hydraulic forces in the reach do not appear to be sufficient to cause consolidation of the large floes and the floes tend to juxtapose against each other, accumulating upstream in a single layer. The increased water levels also cause more water to flow down the Big Snye which can reduce water levels in the Providence Rapids, thus limiting the height of the jam at the Ferry. Long jams of this type produce the highest peak breakup water levels.

3.2 Ice Jam Elevation

Peak water levels at the proposed bridge site are a function of the breakup pattern and the prevailing flows in the river at the time of breakup. Given the historical data at the site, it is appropriate to treat the peak ice jam levels in a statistical manner – with due consideration of the mechanisms which may limit the maximum ice-related water level.

The WSC gauge data provide only a fair record of the peak ice-related water levels that have been experienced at the site. Water levels at or near the Ferry have been monitored since 1964, however frequent gaps in the data occur during the breakup period. Due to these gaps in the record, the peak breakup water level could be reliably determined in only 20 of the 34 years in which data was collected. Even so, some of these records are only daily maximums rather than instantaneous maximums. As well, the breakup records before 1993 were measured upstream at Dory Point so the water levels from this site had to be adjusted downward by 0.69 m to account for the difference in rating curves between the sites, as discussed in Section 2.3.

A frequency analysis of the 20 years of maximum breakup water levels is shown in Figure 3.1. A log-normal frequency distribution fits the data well, however extrapolation of this distribution results in a unreasonable value of 158.45 m for the 100-year breakup level. The elevation predicted by this extrapolation is more than 1.0 m higher than the highest recorded level in Great Slave Lake: a physical impossibility. Clearly a more process based method of estimating the 100-year peak breakup level is required.

The maximum historical peak breakup water level of 155.60 m was observed in 1992 after the formation of a long jam. These long jams can be simulated using the HEC-RAS ice jam model to determine the peak breakup elevations under extreme breakup flow conditions. The calibration of the ice jam model was carried out in two phases. First, the calibration of under-ice roughness in the steep Providence Rapids reach was done using water level data measured on May 3, 1992 after a short jam formed in the reach. Water levels calculated with an under-ice roughness of 0.058 reproduced the measurements reasonably well (Figure 3.2). The applicability of these parameters was confirmed by simulating the ice-related water surface profile measured on May 7, 1992 at a higher discharge after the short jam shoved farther downstream. Second, calibration of under-ice roughness in the Ferry reach was carried out using water level data measured on May 18, 1992 (and again with a higher discharge) after a long jam formed in this reach. During the formation of long jams, the floes tend to juxtapose against each other accumulating upstream as a single layer of ice floes, therefore the ice in the ferry reach is thinner and smoother than the ice in the jam in Providence Rapids. The water levels generated by adopting an under-ice roughness of 0.028 in the Ferry reach were found to simulate the measurements reasonably well (Figure 3.2).

The results of the ice jam model indicated that the maximum water levels at the Ferry occur when the ice jam extends just upstream of the Ferry. However, if the length of the jam should extend further upstream, discharge from the lake would reduce due to backwater effects at the outlet. This reduced discharge, in turn, would reduce the water level at the Ferry. Therefore, as the head of the ice jam progresses upstream through the Ferry reach, water levels will rise to a maximum before falling due the discharge reduction as the head advances upstream. This maximum water level scenario - with the head of the jam just upstream of the Ferry - was adopted as the long jam geometry for design purposes.

Rating curves at the outlet to Great Slave Lake and at the Ferry were calculated for the adopted long jam geometry (Figure 3.3) assuming an ice thickness of 1.54 m. Note that for an ice thickness of 1.54 m, the lake outlet rating curve is not effected at discharges less than 8000 m^3/s. Year-to-year ice thickness variation may cause the long jam water levels at the Ferry to shift ±0.25 m from the values shown in Figure 3.3.

Since water levels at the Ferry are primarily a function of river discharge, and the discharge is determined by the water level at the lake outlet, estimates of discharge can be determined from Great Slave Lake water levels using the long jam rating curve at the lake outlet shown in Figure 3.3. Once discharges are obtained from the lake outlet rating curve, the corresponding water level at the Ferry can be obtained from the Ferry rating curve. The frequency distribution of monthly Great Slave Lake water levels for May (the month that most ice jams occur) are shown in Figure 3.1.

It should be noted that the long jam analysis uses steady lake levels and does not reflect short term water level fluctuations. These fluctuations can produce peak water levels that are up to 0.5 m higher than the steady state ice jam water levels. Thus, 0.5 m should be added to the water levels obtained from the long jam Ferry rating curve to capture the peak ice jam water levels that may be experienced.

The resulting long jam water level frequency distribution at the Ferry is shown in Figure 3.1. These long jam water levels are 1 to 2 m lower than the mean May water levels in Great Slave Lake. This long jam distribution defines the upper limit of ice jam elevations and should be used to determine ice jam levels in cases where the water levels estimated from the previous obtained log-normal distribution of the measured maximum water levels exceed the long jam water levels. Where long jam levels exceed the values of the log-normal distribution of the measured maximum water levels, the log-normal distribution should be used. This transition is indicated in Figure 3.1 by solid and dashed lines – the regions of the curves with solid lines are applicable while the dashed lines are shown only to indicate trends. The composite water level frequency distribution is summarized in Table 3.1. According to the composite long jam frequency distribution, the 100-year ice jam water level is 156.0 m. This water level is only 0.5 m below the elevation of the road on the south bank of the river.

The above ice jam analysis was carried out assuming flow could be diverted into the Big Snye channel. It is estimated that 50% of the flow is diverted down the Big Snye channel when the 100-year ice jam water level occurs. If the Big Snye channel was blocked, all of the flow would go through the Providence Rapids channel, possibly raising water level at the proposed bridge sufficiently to overtop the road. The increased flow in the Providence Rapids reach would either raise upstream water levels significantly or the water would find an alternate pathway downstream to Mills Lake, possibly through the town site. Thus, it is recommended that the Big Snye be maintained free of obstructions.

Table 3.1 Breakup water levels and ice load elevations for various return periods

Return Period (years)	Breakup Water Level at Ferry (m)	Ice Load Elevation (m)
100	155.98	155.30
50	155.86	155.22
20	155.69	155.19
10	154.70	154.18
5	153.75	153.19
2	152.54	151.90

At least 1.5 m of freeboard should be added to the ice jam water level to pass ice floes which are shoved up on one another and projecting above the water level. Therefore, the minimum bridge elevation should be at least 157.5 m. This minimum bridge elevation of 157.5 m is lower than the 158.8 m recommended by Northwest Hydraulics Consultants (1975). The higher value in the previous study is due to the 3.0 m of freeboard recommended. The high ice elevation of 155.8 m prescribed by Northwest Hydraulics Consultants is slightly lower than the elevation of 156.0 m determined from the present analysis.

3.3 Ice Thickness

Ice thickness measurements are available from WSC records and from observations reported by Hicks, Cui and Andres (1995). The WSC gauge near Fort Providence has 17 years of ice thickness measurements obtained during winter discharge measurements while the WSC gauge downstream on the Mackenzie River at Fort Simpson has a much longer 40 year record length. Although the observations by Hicks et al. were made over a short period of four years, they provide a spatial distribution of ice thickness from downstream of Fort Providence to upstream of Beaver Lake.

The measurements from the WSC gauge near Fort Providence were obtained during the periods of 1958-1972 and 1992-1995. The frequency distribution of the annual maximum width-averaged ice thickness from these measurements is shown in Figure 3.4. This data set can be described by a normal distribution with a mean value of 0.99 m and a standard deviation of 0.12 m.

Ice thickness measurements at the WSC gauge at Fort Simpson were obtained for the period from 1958 to 2000. The frequency distribution of the annual maximum width-averaged ice thickness from these measurements is shown in Figure 3.4 for comparison with the data from the Fort Providence gauge. The Fort Simpson data set can be described using a normal distribution with a mean value of 1.28 m and a standard deviation of 0.24 m. The data has one outlier - a thickness of 1.98 m that deviates substantially from the other data, possibly due to the inclusion of frazil in the measurement.

The ice thicknesses at Fort Providence and Fort Simpson for various return periods are summarized in Table 3.2. The 50-year ice thickness at Fort Providence is 1.24 m compared to a 50-year ice thickness of 1.76 m at Fort Simpson. The relatively low values of ice thickness in the Fort Providence data set relative to the Fort Simpson data set may be partly due to the practice of measuring discharge where the cross section intersects the abandoned ferry channel where freeze-up was delayed due to the ferry operation.

Table 3.2 Ice thicknesses for various return periods

Return Period (years)	Ft. Providence WSC Gauge Ice Thickness[1] (m)	Ft. Simpson WSC Gauge Ice Thickness[1] (m)	Ft. Providence Upstream Maximum Ice Thickness[2] (m)
100	1.27	1.83	1.87
50	1.24	1.76	1.83
20	1.19	1.67	1.77
10	1.14	1.58	1.72
5	1.09	1.48	1.66
2	0.99	1.28	1.54

[1] Based on measured data
[2] Calculated on the basis of climate characteristics

The four years of ice thickness measurements carried out by Hicks et al. (1995) provide an indication of the spatial variability of ice thickness near Fort Providence. Late winter ice thicknesses along the river for each of the four years are shown in Figure 3.5. The data indicate that the ice cover at the ferry is not representative of the thickest ice that might occur in the immediate vicinity of the proposed bridge.

There are two reaches where thicker ice forms. One reach is in the rapids between Big River and the Fort Providence Dock. Freeze-up jams in this steep reach likely produce thick accumulations of frazil slush. Ice growth through this frazil slush is more rapid because some ice is already present. Ice thicknesses in this reach do not impact the bridge design because the reach is downstream of the proposed bridge site.

The other reach where thicker ice forms is that between Beaver Lake and Dory Point. Ice thicknesses in this reach are relevant because floes from this reach pass by the proposed bridge during breakup. In this reach, the frazil pans that form during freeze-up in the wide channel upstream, shove over each other as they are transported downstream into the narrower channel.

This stacking of frazil pan crusts produces a thick initial ice thickness with frazil ice underneath. The maximum thickness in the upstream reach for each of the four years is given in Figure 3.4 for comparison with the WSC data. The maximum thicknesses are greater even than the WSC thicknesses for the same return periods at Fort Simpson. However the four years of record are insufficient to determine a reliable frequency distribution. It is evident that the record length needs to be increased. Given the lack of field data, the only recourse is to simulate the expected annual maximum ice thickness in the reach upstream of the proposed bridge for the period of available climate records.

An model that calculates ice growth, based on measured air temperature and snowfall records at Hay River, was calibrated to the existing ice thickness data. Comparisons of the measured and simulated maximum ice thicknesses for the four years of data are shown in Figure 3.6. The simulations in Figure 3.6 were carried out using a heat transfer coefficient of 15 $W/m^2/°C$, an average snow cover density of 175 kg/m^3, and a porosity of 0.6 in the frazil slush accumulation below the solid ice. Assuming only that the channel was filled with frazil at the initiation of freeze-up (without a significant thickness of solid ice due to the stacking of ice pans) could not account for the ice thicknesses observed. The model could only be calibrated to the data by assuming that a solid cover 0.6 to 1.1 m thick formed at the surface of the frazil slush at the initiation of freeze-up. This could occur by the shoving and stacking of several layers of frazil pan crusts.

The calibrated model was used to simulate ice thicknesses using 34 years of climate data from Hay River between 1964 and 1999. An average initial thickness of 0.9 m was assumed in each of the years and the growth of thermal ice within the frazil accumulation was calculated on a daily basis with due consideration to the snowfall and air temperature. Depending on the year, the late winter ice thickness ranged from a minimum of 1.35 m to a maximum of 1.85 m. The frequency distribution of the maximum ice thicknesses from these simulations is shown in Figure 3.3 for comparison with the measured data. The simulated ice thicknesses can be described using a normal distribution with a mean ice thickness of 1.54 m and a standard deviation of 0.14 m. The simulation results are consistent with the measured maximum ice thicknesses but have a lower standard deviation than was indicated by the four years of measured data. These ice thicknesses are also summarized in Table 3.2. The 50-year maximum ice thickness upstream of the proposed bridge site is estimated to be 1.83 m.

It is recommended that the 50-year simulated maximum upstream ice thickness of 1.83 m be used for evaluating ice forces on the bridge piers. This is somewhat lower than the 1.98 m estimated by Northwest Hydraulic Consultants (1975) for this site, however their original estimate was extrapolated from only one year of observations. The use of a 50-year thickness is appropriate when the thickness is combined with extreme values of ice strength and elevation to provide the design moment for the bridge piers.

3.4 Ice Strength

Ice strength is more difficult to define than ice thickness due to the lack of measurements and the large variability in those measurements. NWT Transportation (1992) measured compressive strengths in natural ice near the ice bridge in 1989 and 1991. The measurements ranged from

720 kPa to 8650 kPa. The mean strength in 1989 was 2920 kPa while in 1991 it was 6180 kPa. No temperature data were reported but the measurements were taken during ice bridge construction so presumably the temperatures were significantly less than 0°C.

Ice strength is a function of ice grain size, porosity and temperature. Undeteriorated frazil ice with crystals ranging in size from 1 to 3 mm has a typical strength about 2500 kPa at 0°C with no internal melting. Columnar ice with crystals ranging in size from 5 to 25 mm has a strength of about 900 kPa at 0°C with no internal melting. A strength of 6180 kPa measured in 1991 could only have developed in cold, competent, small-grained ice.

The ice cover upstream of Dory Point in Beaver Lake breaks up due to decreasing ice strength from solar radiation as well as from increasing discharge as the open water area at the outlet to Great Slave Lake increases in size. There is also some uplift effects from ice jams occurring downstream. There is no chance of a mid-winter breakup due to the flow regulation provided by Great Slave Lake. Typically the ice moves out about 12 days after the snow cover disappears but it has been observed to occur in as little as 8 days (Hicks et al., 1995). During this time period, the depth-averaged ice strength is expected to deteriorate to values of about 1100 kPa due to the absorption of solar radiation.

It is recommended that 1100 kPa be adopted as the design ice strength. This is also the strength recommended in the CAN/CSA-S6-88 Design of Highway Bridges "*where breakup or ice movement occurs at melting temperatures, but the ice moves in large pieces and is internally sound*". This strength corresponds to the 2100 kPa ice pressure specified by Northwest Hydraulic Consultants (1975) using older design standards.

3.5 Ice Load Elevation

The mechanism of long jam formation at the proposed bridge site makes it possible that ice loads can occur when the ice jam is at its peak elevation. The ice jam forms by ice floes accumulating at the head of the ice jam so floe velocities can be significant even when the jam is forming.

Ice loads are applied at the center of the ice thickness so ice load elevations will be lower than the ice jam water levels discussed in the previous section. The ice load elevation is 42% of the ice thickness below the water surface because the ice is floating. For a design ice jam water level of 156.0 m and ice thickness of 1.83 m, the ice load elevation is 155.2 m. This ice load elevation is higher than the 154.9 m recommended by Northwest Hydraulic Consultants (1975). The higher value in the present study is likely due to the application herein of a more sophisticated method to assess the ice jam levels.

The proposed piers consist of twin circular shafts with a connecting web extending vertically over the range of expected ice load elevations. This web is required to increase the resistance of the piers to ice loads by transferring load from the front shaft to the rear shaft. The minimum, as well as the maximum, ice load elevation must be known to define the position of the web. The minimum water level at the time of first ice movement at the Ferry over a four year period was observed to be 151.4 m (Hicks et al., 1995); however, the minimum pre-breakup water level recorded at the WSC gauge adjusted to the Ferry is 150.8 m. The recommended ice load elevation is 42% of the design ice thickness below this minimum water level at elevation 150.0 m.

3.6 Ice Loads

Ice loads on bridge piers are primarily caused by dynamic ice movement but thermal and uplift load can also be a factor in pier design. Dynamic ice loads on the proposed piers will be produced by crushing failure of ice floes against the vertical face of the pier. The ice will primarily drift with the current, however wind effects can change the direction of the floes significantly. The ice floes can be quite large due to the size of the channel. Floes as large as 800 m by 800 m were observed by Hicks et al. (1995). Thermal forces are not expected to be of concern at the proposed bridge location because the ice sheet is expected to surround the piers and balance the thermal forces on each side of the piers. Uplift loads can occur when rapid water level increases occur when the ice sheet is frozen to the piers. This may occur at the proposed bridge site when an ice jam forms downstream in the Providence Rapids reach.

The potential ice loads on the bridge piers are quite large. The proposed piers are constructed of two round shafts 2.75 m in diameter and 7.0 m apart on centre, connected with a thin web from elevation 149.5 m to the top of the pier. The recommended ice thickness of 1.83 m and ice strength of 1100 kPa would produce a crushing load of 11.5 MN on the lead shaft in the direction of the long axis of the pier. There is no reduction for kinetic energy limitations because the ice floes can be quite large. The proposed bottom elevation of the web of 149.5 m is 0.5 m lower than the recommended minimum load elevation. The maximum ice load elevation is 155.2 m so extending the web to the tops of the piers is more than sufficient for transferring ice loads between the shafts.

The proposed bridge is perpendicular to the deep water channel but is skewed about 10° LHF to the banks. Thus, if large floes are drifting parallel to the banks, they will contact the piers at a 10° angle from the long axis of the pier.

Wind may also cause ice loads to be skewed relative to the long axis of the piers. An maximum hourly wind speed of 60 km/hr blowing transverse to the flow direction may occur, however, the risk of this extreme wind occurring during the design ice load event is extremely small. A more common mean hourly wind of 12 km/hr will produce a velocity component of about 0.1 m/s transverse to the ice jam affected water velocity of 0.9 m/s. This will produce a skew of about 6°, which when added to the channel skew, results in a potential total skew of 16°.

The potential ice load skew of 16° relative to the long axis of the piers may produce loads transverse to the long axis of the pier on either the front shaft and rear shaft individually or simultaneously on both shafts. On the front shaft, a transverse component of the ice load of 3.2 MN can occur when the total load of 11.5 MN is applied at angle of 16°., The total load on the rear shaft is limited to 9.8 MN because about 75% of the shaft diameter is exposed to the ice can occur at a 16° skew. The component of this load transverse to the long axis of the pier is 2.7 MN. The connecting web is not exposed directly to ice loads at skew angle of 16° but may incur smaller loads of up to 1.5 MN due to contact with ice rubble.

Even at a 16° skew, the ice floes can be large enough to have sufficient kinetic energy to crush through the entire length of the pier. An ice floe up to 230 m wide may strike a single pier and this floe can be up to three times longer than it is wide. Ice floes striking the piers supporting the central navigational span may be even larger, up to 315 m wide and 900 m long. The CAN/CSA-S6-88 Design of Highway Bridges states that the projected width of the pier should be used to determine the ice load when the load is skewed relative to the long axis of the pier. The greater pier width for the skewed load results in a lower effective ice pressure than for a single shaft. This reduced effective ice pressure is consistent with the unlikelihood that the peak loads on the two shafts would occur simultaneously. Thus, the maximum total load on a pier is less that the sum of the maximum loads on the individual shafts. The maximum total load on the pier is 15.0 MN with a transverse component of 4.1 MN. These dynamic ice loads are summarized in Table 3.3.

Uplift loads can occur when rapid water level increases occur when the ice sheet is frozen to the piers. This may occur at the proposed bridge site when an ice jam forms downstream in the Providence Rapids reach. The maximum total load acting upward is expected to be 1.6 MN and will be equally distributed to the two shafts. These uplift loads are also summarized in Table 3.3.

Table 3.3 Ice loads on bridge piers

Load Type	Front Shaft Load (MN)	Rear Shaft Load (MN)	Total Pier Load (MN)
Dynamic Ice Load[1]	11.5	9.8	15.0
Transverse[2] Component of Dynamic Load	3.2	2.7	4.1
Uplift Load	0.8	0.8	1.6

[1] For proposed pier geometry
[2] Relative to the long axis of the pier.

3.7 Ice Shoves

The approach embankments project significantly into the channel and therefore will be exposed to dynamic ice action. Ice floes drifting down the channel during breakup will strike the slopes of the upstream sides of the approach embankments. The kinetic energy of these floes can be quite large so the ice will ride up the slope some distance before coming to a halt. These ice shoves are significant for two reasons; first, if the height of the abutments are too low, the shoved ice will contact the bottom of the girders and second, if the approach embankments are too low, ice will shove on to the roadway on the approach embankments.

Observations in 1992 (Hicks, 1992) indicate that shoves on the shoreline can be as high as 4.0 m above the water level. Ice accumulations as high as 8.0 m above the water level were observed on the north ferry landing but this was due to the ice rubble generated from crushing against the vertical sheet pile on the upstream side of the ferry landing.

The height of an ice shove depends on the size of the floe, the approach velocity of the floe, and the slope of the embankment upon which the floe is impinging. The floe velocity is a function of the discharge and the water level at the bridge. For a given discharge during the breakup period, high velocities tend to occur at low water levels and low velocities at high levels, thus there is a limit to how high a given ice floe can shove up a given slope. For example, simulations of energy conservation during ice shoves indicate that the height of a shove would be 3.5 m above the water level for a single large floe three river widths in length with a velocity of 1.6 m/s against the 1.5:1 slope of the embankment. This velocity can only occur at low open water elevations when there is no ice jam backwater effect and with an ambient water level of 152.0 m, so the elevation of the top of the ice in the shove would be 155.5 m. During a severe long jam, the water velocity is only about 0.9 m/s at an ambient water level of 156.0 m. The same large ice floe at this slower velocity would only shove 2.0 m above the water level to reach an elevation of 158.0 m.

The above shove elevations indicate that ice shoves on the embankments will reach higher elevations than would result only from jamming of the ice. The analysis suggests that either the minimum bridge elevation be raised to 159.0 m to provide 1.0 m freeboard against ice shoves or the abutment headslopes should be protected from these ice shoves. However, protecting the headslopes from a potential 16° angle of attack will require the channel to be constricted further.

Ice will shove onto the roadway only when the bottom of the leading edge of an ice floe reaches the roadway elevation of 156.5 m. This will only occur during severe long jams when the water level is as high or higher than 155.3 m, a elevation which has a 10-year return period during breakup. The highest shoves would occur when the ice is relatively thin since the bottom of the floating ice is at a higher elevation. For example, the roadway surface would need to be raised to 157.2 m to stop a 0.8 m thick ice floe from shoving onto the roadway during an extreme event. Alternatively, the roadway could be protected against ice shoves by some mechanism which stops the ice before it reaches the roadway surface.

One method of protecting the roadway and headslopes is to construct a spur parallel to each of the embankments on the upstream side of the bridge. These spurs would cause the ice to shove up the side of the spur and allow the rubble to accumulate in the space between the spur and embankment. To be effective, the spur must be high enough to trigger bending failure in the ice floe so the top of the spur should be 0.3 m higher than the bottom of the ice cover. The elevation of the top of the spur required to protect against a 100-year ice shove is 155.6 m. Individual pieces of ice rubble are estimated to be between 5 and 10 m in length so the top of the spurs should be at least 15 m from the edge of the roadway to accommodate the rubble. The spurs may have side-slopes of up to 1.5:1.

The protection may be composed of isolated mounds rather than a continuous structure to reduce the amount of material required. The spacing between the peaks of the mounds should not exceed 30 m to ensure that the floes are broken or deflected by the mounds.

Another method of protecting the roadway from ice shoves is to construct a vertical barrier along the upstream embankment slope to stop the shove before it reaches the roadway surface. The bottom of the barrier should be at an elevation of 155.6 m so that bending failure is initiated on the embankment slope before the floe makes contact with the barrier. The barrier should extend up to an elevation of 157.2 m to stop the floe without causing ice rubble to overtop the structure. This structure must be designed to withstand an unit ice load of 20 kN/m over the width of the structure.

The choice between protection alternatives will depend on the relative construction and maintenance costs of the structures as well as environmental and aesthetic considerations. This cost analysis is beyond the scope of this study, however, it would appear at first glace that the vertical barrier is the best alternative due to the smaller amount material required, the lack of unsightly mounds in the upstream channel, and that further in-channel construction is not required.

4.0 SCOUR

In general, scour in bridge waterways is caused by the increased local flow velocities which occur due to the presence of the bridge. Scour holes at the base of bridge piers and abutments can reduce the effectiveness of the foundations and, in the extreme, cause the foundations to fail. As well, constriction scour may reduce the bed elevation across an entire bridge waterway due to increased mean velocity caused by a reduced width in the bridge waterway. Scour depths are a function of bridge geometry, flow velocity and bed material size.

4.1 Bed Material

A foundation investigation carried out by Structural Engineering Services Ltd. (1958) indicated that the river bed is composed of clay till overlain by a thin layer of alluvial material. The alluvial material is composed of sand, gravel, cobble and even small boulders. The alluvial layer is up to 1.8 m thick in some places but in other places the clay is exposed. This clay is generally hard and dry and may contain stones 50 mm to 150 mm in diameter.

No grain size analysis of the alluvial material is available so only estimates of material sizes can be made from the descriptions. Sand-gravel-cobble mixtures tend to have median grain sizes of about 20 mm and 90 percent of the material would be less than about 100 mm in diameter. Additional bed material sampling, as part of the subsurface investigation, should be done at the locations of the piers and abutments to determine the actual grain size distribution before the bridge design is finalized.

4.2 Cross-Section Stability

Clays have a wide range of resistance to erosion and testing equipment is not generally available to assess the erodibility. Thus alternative methods of evaluating resistance to erosion must be used. For example, the shape and stability of the cross-section shape can give an indication of ability of the clay to resist erosion.

An evaluation of bed stability by Northwest Hydraulics Consultants (1975) indicates that the location of the navigation track is stable – suggesting that the clay bed of the channel is relatively non-erodable. However, a comparison of soundings between 1947 and 1975 by Northwest Hydraulics Consultants indicated that local bed levels may have changed as much as 3 m. It was suggested that these apparent changes were mainly due to lack of precision in locations between the surveys.

A comparison of WSC cross-sections surveyed at the Ferry with a cross-section from the 2002 survey carried out by Trillium Engineering and Hydrographics Inc. is shown in Figure 4.1. This comparison indicates that the general shape of the cross-section has remained the same over a 37 year period, however, there are local differences in bed elevation of up to 2 m. These differences may be due to temporal changes in bed elevation, but are most likely due to differences in location of the measurements longitudinally in the river. The persistence of the general cross-section shape indicates that the much of the bed material is relatively non-erodable, even with historical flows approaching the design discharge.

The minimum bed elevation at the ferry site is about 2.3 m lower than the mean bed level, or 1.43 times the mean depth at the design flow. This ratio is somewhat greater than that expected for a straight channel. However, the thalweg meanders within the channel so the channel may behave more like a moderately curved channel where the maximum depth may be as high as 1.5 times the mean depth. The minimum bed level is currently near the center of the channel and is unlikely to shift from this location. However, portions of the bed composed of transportable material may become mobile or shift to a different location during flood flows. This process may cause local and temporary lowering of bed levels of about 1 to 2 m.

4.3 Constriction Scour

The reduced width in a bridge waterway will cause the velocity to increase through the waterway relative to the natural channel. This increased velocity may transport bed material from the bridge waterway, causing the bridge waterway to lower locally. The increase in depth in the bridge waterway is a function of the ratio of natural channel width to the constricted channel width.

The proposed bridge waterway width of 1050 m is 70% of the upstream channel width. The constriction scour expected for this width constriction is about 1.4 m at a design flow of 10,500 m^3/s, however, the width constriction caused by the proposed bridge is slightly less than the present constriction due to the ferry landings so no additional scour is expected due to the presence of the bridge. At present, the mean depth in the ferry waterway is about 1.8 m greater than in the natural section upstream at Dory Point, which is slightly greater than predicted from the constriction scour analysis. The ferry landings were already in place in 1958, so if the bed material is susceptible to scour, the bed level has likely adjusted to the constricted width already and has already attained its maximum constriction scour.

The scour depth in the constriction can also be estimated by determining the flow depth necessary to reduce the mean velocity to the point at which it cannot transport the bed material. This competent mean velocity is a function of bed material size and flow depth. The competent velocity of the alluvial sand-gravel-cobble mixture will be about 2.0 m/s which is greater than the mean velocity through the bridge waterway at the design flow, thus this method also indicates that no further constriction scour will take place. Still, it would be prudent to assess the actual bed material before proceeding with final bridge design.

4.4 Pier Scour

Scour also occurs around the bridge piers due to the flow pattern produced by the piers. The magnitude of pier scour that would develop is a function of the pier width, flow intensity and bed material. Depth of scour will depend on whether the bed material at the pier is alluvial or clay and, if it is alluvial, the depth of the alluvial material. Pier scour estimates should be added to any constriction scour or scour due to thalweg shifting.

Scour depths in the alluvial material can be estimated using the adopted bed material size distribution described in Section 4.1. For a given bed material, pier scour is primarily a function of pier width, the water depth, and flow velocity (Richardson, 1990 and Melville, 2000). At the design discharge of 10,500 m^3/s, the maximum flow depth is expected to be 7.9 m with a velocity of 2.3 m/s. Under these flow conditions, the scour depth in the alluvium is estimated to be about 3.3 m. This scour should develop relatively quickly in alluvial material; however, if the depth of the alluvium is less than this scour depth, the scour will develop at a much slower rate once it reaches the clay level. Again, it would be prudent to assess the actual bed material before proceeding with final bridge design.

Ultimate scour depths in clay are expected to be as great as those which occur in sand (Briaud et al., 1999 and Ting et al., 2001), however the shear stress on the bed must exceed the critical shear stress of the clay for scour to occur and the time required to reach the ultimate scour depth also depends on the properties of the clay. Scour depths in sand are estimated to be about 4.9 m. The proposed bridge site is located downstream of the outlet to Great Slave Lake so flood flows have longer durations than typical river floods. Unfortunately, it is difficult to quantify the erosional properties of clay without specialized laboratory equipment. However, the insitu behaviour of the clay at the ferry landings suggest that the critical shear stress is quite high since little scour is evident.

Given the uncertainty of scour in clay materials, the piers should be designed for an ultimate scour depth of 4.9 m. However, since the ultimate scour will not occur after a single flood event, it may be more cost effective to design for a smaller amount of scour of about 2 m, but monitor the scour development. If significant pier scour occurs, the scour holes can be filled with rock to stop additional scour. Class II riprap with a median diameter of 0.5 m would be sufficiently large to be stable under design flow conditions. The choice of design scour depth will depend on the relative cost of providing the deeper scour protection versus to the cost of monitoring and subsequent riprap placement.

4.5 Abutment Scour

The bridge approaches and abutments utilize the existing ferry landings, however the bridge waterway is offset from the 970 m wide ferry waterway in order to position the center of the bridge over the navigation track. The proposed bridge alignment results in the south abutment encroaching approximately 50 m further into the river than the present ferry landing while the north abutment is set back about 130 m from the end of the existing ferry landing.

Scour occurs at the abutments due to the flow disturbance produced in their vicinity. This type of local scour is a function of the abutment geometry as well as flow intensity and bed material. Abutment scour estimates should be added to any constriction scour or thalweg shifting which may occur.

Methods of estimating abutment scour at long abutments have been developed by based on both extensive laboratory measurements (Melville, 2000) and based on field data on scour at the noses of spurs on the Mississippi River (Richardson, 1990). Both these methods produce seemingly excessive scour depths, especially in light of the bed materials at the proposed bridge site. The expected scour would be about 3.0 m even if the abutments are treated as large spurs projecting into the flow rather than as abutments.

The existing ferry landings project significantly into the flow at present but do not exhibit abutment scour of the magnitude suggested by the above scour calculations. The 2002 survey carried out by Trillium Engineering did not detect any scour hole at the shorter south landing and found only about 1.0 m of possible scour next to the sheet pile at the longer north landing. These measurements suggest that the bed at the ferry landings is quite stable and that only 1 to 3 m of additional scour should expected depending on the thickness of the alluvial layer at this location. Again, it would be prudent to assess the actual bed material before proceeding with final bridge design.

4.6 Abutment Protection

The abutment headslopes should be protected to prevent erosion of the fill material due to the action of scour, waves, and ice. Protection against scour is required on both the slope and the bed at the base of the slope. Protection against waves and ice is required only in the elevation range at which these processes occur. Slope protection options include rock riprap, concrete, gabions, and sheet piling. Concrete on the side slopes with rock on the apron could be a reasonable alternative but given the ice affects at the site gabions are not recommended. Rock riprap is usually the most cost-effective material and, furthermore, the flexibility of riprap allows for minor settling of fill material on the slope without compromising the protection.

The USACE (1991) recommends sizing riprap according to velocity and headslope angle. The mean velocity through the bridge opening is 1.9 m/s. According to this method, Class I riprap with a median diameter of 0.3 m is adequate to protect the slope even at 1.5:1. This appears too small given that the local velocity due to flow acceleration around the headslope may increase the local velocity by 50% relative to the mean flow (Croad, 1989). In this case, Class II riprap with a median diameter of 0.5 m is required to protect the slope for the proposed 1.5:1 slope angle. Class I rock with a median diameter of 0.3 m may be used if the headslope angle is reduced to 2:1.

Wind induced waves are also a significant issue. The fetch length is as long as 45 km in the south-east and east-south-east direction and an extreme hourly wind of 60 km/hr in this direction may produce waves of about 0.9 m in height. Class II riprap with a median diameter of 0.5 m is required to protect the slope against wave of this magnitude. The riprap should extend as high as 155.0 m to protect against wave run-up which may be as high as 2.7 m.

Ice may damage riprap either by plucking it off the slope during a rapid rise in water level or sliding over the slope during an ice run. No rapid rises in water level are expected to occur when a significant amount of ice is attached to the riprap; because the ice in the ferry reach needs to breakup and move downstream into the rapids in order for an ice jam to form and, in turn, raise the water at the bridge. This can only occur after significant melting at the shore around the abutment. Still, ice floes are likely to slide up the riprap slope and dislodge rocks unless they are large enough to resist the force.

Riprap will be damaged unless it is large enough to resist ice action. Analysis of riprap stability suggests that the stable riprap size varies with the square-root of ice thickness (Carter, 2001). A median riprap diameter of 2.2 m on a 1.5:1 slope is required to resist ice floes with the ice characteristics used in the assessment of ice forces. Even for deteriorated ice with a thickness of 1.0 m, a median diameter of 0.95 m is required for riprap stability. Thus, it appears that the riprap size required for hydraulic design is inadequate to resist the ice conditions at the site. For protection against ice, a modified Class III riprap with a median diameter of 1.0 m can be used in areas exposed to direct ice action, however, this riprap may sustain significant damage if ice floes with the ice characteristics used in the pier design impact with the riprap.

Rock riprap should be well graded according to the specifications as summarized in Table 4.1. The rocks should be angular and should tend to cubical rather than thin slabs to provide maximum stability. The riprap should be placed on the slope in a layer 1.5 times the median diameter in thickness overtop of a geotextile filter fabric. The riprap should be placed up to an elevation of at least 156.0 m to prevent erosion damage during ice jam events and from wave action. As well, a launching apron with a thickness of two times the median diameter should be provided at the base of the slope. Class II riprap is sufficient for the aprons since they are below the level of significant ice action. This launching apron need only extend out 6.0 m to protect against 3.0 m of scour. This length should be optimized once the results of the bed material sampling are made available.

Table 4.1 Rock riprap gradations

Size	**Class II**		**Modified Class III**	
	Diameter (m)	**Weight (kg)**	**Diameter (m)**	**Weight (kg)**
100% less than	0.8	700	1.5	5000
20% greater than	0.6	300	1.2	2400
50% greater than	0.5	200	1.0	1500
80% greater than	0.3	30	0.6	300

The location of the slope protection on the approach fills will depend somewhat on the final approach fill geometry adopted. If protecting spurs are not used, the headslope and adjacent 50 m on the upstream side of the embankment should be protected against ice action with modified Class III riprap. On the downstream side of the bridge, this modified Class III headslope protection need only extend to an angle of 45° from the bridge. The remainder of the embankments should be protected against wave action with Class II riprap. The riprap on the approach embankments will not need to resist ice impacts if spurs are constructed upstream of the approach fills to protect against ice shoves. In this case, the headslope, the spur nose and adjacent 50 m on the upstream side of the spur would require the modified Class III riprap with a median of 1.0 m for protection from ice. The remainder of the fill exposed to wave action should be protected with Class II riprap. Damage to the riprap would not be catastrophic because it would still function as intended in the short term, however the riprap should be maintained on an annual basis.

Other forms of slope protection should be considered in areas where ice action is a concern if annual riprap maintenance costs are to be avoided. Vertical sheet piles may be used but they would need to extend up to an elevation of 155.6 m to be effective for protection against ice shoves. These vertical structures would also be required to withstand ice crushing along the entire upstream length of the embankment. Concrete slope protection is probably a better choice because ice will readily slide up the slope and fail in bending. Concrete slope protection tends to be more expensive than riprap but will not require annual maintenance to remain effective. The concrete would still require a riprap launching apron at the base of the slope to protection against scour in the thin alluvial layer.

5.0 SUMMARY

This study updated the existing hydrotechnical information at the Ferry to provide design criteria for the final design of a bridge across the Mackenzie River at Fort Providence. This hydrotechnical information is summarized below in three sections: open water hydraulic characteristics, ice characteristics and scour. The hydrotechnical design criteria are summarized in Table 5.1.

5.1 Open Water Hydraulic Characteristics

Open water hydraulics were simulated using HEC-RAS to determine rating curves at the Ferry and at the outlet to Great Slave Lake. Water levels in Great Slave Lake control flows in the Mackenzie River. Great Slave Lake water levels were defined by data from Yellowknife Bay. Wind can effect water levels at the lake outlet by 0.15 m causing discharge changes of up to 500 m^3/s.

The 100-year annual maximum discharge of 10,500 m^3/s is recommended as the design open water discharge. At this discharge, the top width in the proposed bridge waterway is about 1050 m at a water level of 152.44 m. The mean and maximum depths are 5.6 and 7.9 m respectively at the design discharge. The design mean velocity through the bridge will be 1.8 m/s with a maximum value of 2.3 m/s. At the 100-year water level, the navigational clearance would be 22.21 m.

During the open water period the discharges at Fort Providence have ranged from 3,170 to 10,400 m^3/s with a median flow of 6,600 m^3/s. The water level in the bridge waterway for these discharges ranged from a low of 149.85 m to a high of 152.34 m. The proposed bridge would provide a minimum clearance of at least 22.68 m for 95% of the open water season. Average velocity in the bridge waterway varied from 0.98 to 1.78 m/s while maximum velocity varied from 1.39 to 2.25 m/s.

5.2 Ice Characteristics

Breakup at the Ferry is initiated by backwater from ice jams forming in Providence Rapids which lift and crack the ice in the ferry reach. Maximum breakup water levels at the Ferry are produced when Beaver Lake ice runs downstream to the head of the jam and accumulates upstream to form a long jam which extends upstream to the Ferry. The maximum historical peak breakup water level of 155.60 m was observed in 1992 after the formation of a long jam**.**

Simulations indicated that the discharge and maximum ice-related water levels at the Ferry are a function of water level in Great Slave Lake and the backwater effect from the ice jams. The 100-year ice jam water level obtained from the ice jam simulations is 156.0 m. At least 1.5 m of freeboard should be added to the ice jam water level to pass ice floes which are shoved up on one another and projecting above the water level, therefore, the minimum bridge elevation should be at least 157.5 m. Breakup water levels at the Ferry could be even higher if the Big Snye channel is blocked so this channel should be maintained free of obstructions.

Ice thickness in Beaver Lake is greater than the ice thickness in the ferry reach due to shoving and stacking of pans at freeze-up. Four years of measurements in Beaver Lake were used to calibrate an ice growth model which was used to extend the period of record to the 34 years of available climate data. The 50-year simulated ice thickness in Beaver Lake of 1.83 m is adopted as the design ice thickness. This thickness is similar to the extreme thicknesses at Fort Simpson.

Limited strength measurements in winter indicated that the Beaver Lake ice is relatively strong. Ice movement occurs at melting temperatures, but the ice moves in large pieces and may be internally sound so it is recommended that 1100 kPa be adopted as the design ice strength.

The recommended ice thickness of 1.83 m and ice strength of 1100 kPa would produce a longitudinal crushing load of 11.5 MN on the lead shaft. A skew of up to 16° may cause a transverse component of 3.2 MN on the front shaft. On the rear shaft, the total load is limited to 9.8 MN at an angle of 16°. The component of this load transverse to the long axis of the pier is 2.7 MN. The maximum total load on the pier is limited to 15.0 MN with a transverse component of 4.1 MN. For a design ice jam water level of 156.0 m and ice thickness of 1.83 m, the ice load elevation is 155.2 m. The minimum ice load elevation is 150.0 m.

Ice floes drifting down the channel during breakup may strike the upstream sides of the approach embankments and shove up the slopes. It is estimated that these shoves can reach an elevation of 158.0 m. The minimum bridge elevation should be raised to 159.0 m to provide 1.0 m freeboard against ice shoves.

The roadway and abutments should be protected from ice shoves. A vertical barrier constructed along the upstream embankment slope to stop the shove before it reaches the roadway surface appears to be the best alternative for protection. The bottom of the barrier should be at an elevation of 155.6 m so that bending failure is initiated on the embankment slope before the floe makes contact with the barrier. The barrier should extend up to an elevation of 157.2 m to stop the floe without causing ice rubble to overtop the structure. This structure must be designed to withstand an unit ice load of 20 kN/m over the width of the structure.

5.3 Scour

The river bed is composed of clay till overlain by a thin layer of alluvial material composed of sand, gravel, cobble and even small boulders. The alluvial layer is up to 1.8 m thick in some places but in other places the clay is exposed. The cross-section at the ferry is stable which indicates that the clay is relatively non-erodable. Bed material at the site should be assessed as part of the subsurface investigation before proceeding with final bridge design.

The maximum constriction scour is expected to be about 1.4 m at a design flow of 10,500 m^3/s The bridge will not constrict the channel further than the existing constriction at the ferry landings, so if the bed material is susceptible to scour, the bed level has likely adjusted to the constricted width already and has already attained its maximum construction scour.

Pier scour depth in the alluvium is estimated to be about 3.3 m but may be limited by the depth of the alluvium. Ultimate pier scour depths in clay are estimated to be about 4.9 m, however, this scour occurs at a slow rate and only when the critical shear stress is exceeded. It may be more cost effective to design for a smaller scour depth of about 2.0 m, but monitor the scour development. If significant pier scour occurs, the scour holes can be filled with Class II riprap with a median diameter of 0.5 m.

Maximum abutment scour is estimated to be 3.0 m if the abutments are treated as large spurs projecting into the flow. Measurements suggest that up to 1.0 m of abutment scour has occurred at the existing ferry landings. Class II riprap (median diameter of 0.5 m) aprons 6.0 m long and 1.0 m thick are sufficient to protect against abutment scour at the bridge site.

Class II riprap with a median diameter of 0.5 m is required to protect the slopes from erosion and wave action but a modified Class III riprap with a median diameter of 1.0 m is required to provide some stability in the areas exposed directly to ice action. These areas include the headslopes and the adjacent 50 m on the upstream sides. Even this larger riprap may require annual maintenance to repair minor damage. Concrete slope protection should be considered if annual riprap maintenance costs are to be avoided. The riprap should extend as high as 156.0 m to protect against wave run-up and ice action.

Table 5.1 Summary of design criteria

Parameter	Value
Open Water Hydraulic Characteristics	
100-year open water discharge	10,500 m^3/s
100-year open water elevation	152.44 m
Minimum clearance at 100-year elevation	22.21 m
100-year mean velocity in bridge waterway	1.8 m/s
Ice Characteristics	
100-year ice jam elevation	156.0 m
100-year ice shove elevation	158.0 m
Minimum bridge elevation	159.0 m
Top of vertical ice shove barrier elevation	157.2 m
Bottom of vertical ice shove barrier elevation	155.6 m
Design ice thickness	1.83 m
Design ice strength	1100 kPa
Maximum ice load elevation	155.2 m
Minimum ice load elevation	150.0 m
Design ice loads	See Table 3.3
Scour	
Additional constriction scour depth	0.0 m
Ultimate pier scour depth	4.9 m
Abutment scour depth	3.0 m
Median diameter of riprap exposed to ice action	1.0 m
Median diameter of riprap not exposed to ice action	0.5 m
Top of riprap elevation	156.0 m
Riprap apron length	6.0 m

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ADDENDUM

Ice Load and Scour Analysis for New Pier Geometry

Jan. 8, 2004

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January 8, 2004

JIVKO Engineering
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Attention: Mr. Jivko Jivkov, P. Eng.

Dear Sir:

RE: Ice Load and Scour Analysis for New Pier Geometry.

The following letter report describes the results of the re-evaluation of ice loads and scour for the pier geometry proposed by J.R. Spronken and Associates Ltd on Dec. 17, 2003. This pier geometry was proposed to address concerns regarding ice loads resulting from previous analysis. The re-evaluation of scour also incorporates the recent geotechnical data collected by EBA Engineering Consultants Ltd. (2003). Please refer to our original report (Trillium, 2002) for background information and an evaluation of other hydrotechnical issues at the bridge site.

The proposed pier configuration consists of large-diameter oval-shaped pier bases which taper in the ice action zone to smaller platforms which support the steel bents that carry the bridge superstructure. The two large central piers (Pier 4) are about 13.4 m long and 7.3 m wide at the bottom and support the cable stays for the 190 m long main span. The other six piers (Piers 1-3) are about 12.3 m long and 6.9 m wide and support the shorter 112.5 m spans. The noses of the pier bases are sloped at 63.4° (2:1) from horizontal while the sides slope in at an angle of 71.6° (3:1) from the horizontal. The sloped surfaces of the piers begin at elevation 149.0 m and extend up to an elevation of 156.0 m.

All the piers have 2.5 m thick footings with the tops of the footings located 1.0 m below the local bed level. The footings are about 17.5 m long and 12.5 m wide.

The bridge is aligned with the existing ferry landings so the piers are skewed to the general direction of the approach flow. This skew can be a significant issue for both the ice loads and scour patterns.

1.0 Ice Loads

Bending loads and ice load elevations were assessed for the new pier geometry. The angle of attack, or skew, of the ice floes relative to the long axis of the piers was assessed for the new geometry and the effect of this skew on the ice loads was evaluated. As well, kinetic energy limitations of the ice floes were assessed to determine if the skewed ice loads can be reduced relative to the loads resulting from contact over the entire projected width of the piers.

1.1 Bending Loads

The proposed pier geometry incorporates a vertical pier nose angle of 63.4° in order to induce bending failures in the ice floes. The ice loads generated by these bending failures are much less than the loads generated by crushing failure on a pier with a similar width and a vertical nose because bending loads are not a function of pier width. The bending failure load generated by a floe with the design ice strength of 1100 kPa and the design ice thickness of 1.83 m is 9.0 MN.

Analysis of low flows indicates that the water level at the bridge location can be as low as 149.56 m during the winter period; however, the water level tends to rise before ice movement occurs. Thus the lower limit of the sloped surfaces of the piers at elevation 149.0 m appears to be sufficient to ensure bending occurs during breakup.

According to the Canadian Highway Bridge Design Code CAN/CSA-S6-00, where the longitudinal axis of the pier is reasonably parallel to the direction of the movement of ice, two design cases should be investigated. In one case, a transverse load of 15% of the longitudinal load should be applied to the pier nose at the same time as the longitudinal bending load. In the second case, a transverse load of 34% of the longitudinal load should be applied to the round pier nose at the same time as 50% of the longitudinal bending load. These load cases are summarized in Table 1.

1.3 Effects of Flow Alignment

The original hydrotechnical analysis indicated that the bridge was aligned with the deep water channel but was skewed 10° relative to the banks. In order to determine which direction the ice floes would approach the piers, flow patterns through the bridge opening were investigated with a 2-D numerical model incorporating simplified river bed geometry obtained from the navigation chart for the reach. The two-dimensional flow analysis indicated that the flow in the northern portion of the channel would approach the bridge at an angle of about 3° RHF while the flow in the southern portion of the channel would approach the bridge at an angle of about 16° LHF (Figure 1). In addition, a wind blowing across the channel may induce a lateral velocity component to the ice floes. A mean wind speed of 12 km/hr could induce a transverse velocity component of 0.05 m/s in the 3 km long approach reach which, when combined with the ice jam affected floe velocity of 0.9 m/s, would increasing the approach angle by 3°. This analysis indicates that the approach angle may vary from 6° RHF to 19° LHF depending on the pier location and the wind direction.

The bridge design code indicates that if the piers are skewed relative to the direction of the ice floes the projected width of the piers should be used to calculate ice loads. However, the side slopes on the piers will induce bending failures for ice floes striking even at large angles of attack. At an angle of attack of 19°, the effective nose angle in the direction of travel of the floe is only 46°, thus the maximum transverse load due to skew is 3.3 MN. Longitudinal loads of 10% of the transverse loads occur at the same time, due to the effects of friction as the floe strikes the pier flank. These ice loads are summarized in Table 1.

Table 1 Summary of ice loads and elevations

Load Type	Load Elevation (m)	Standard Piers		Central Piers	
		Longitudinal Load (MN)	Transverse Load (MN)	Longitudinal Load (MN)	Transverse Load (MN)
Nose load at 63.4° slope: Case 1	155.2	9.0	1.4	9.0	1.4
Nose load at 63.4° slope: Case 2	155.2	4.5	3.1	4.5	3.1
Flank load at 16° skew to flow	155.2	0.3	3.3	0.3	3.3

2.0 Scour

Two scour related issues were investigated for the proposed bridge geometry: (1) long term general scour due to channel degradation and (2) pier scour. Pier scour was re-analyzed because the pier shape was changed considerably from that considered in the earlier assessments. Long term degradation may be a concern for the new pier design because the integrity of the shallow spread footing foundation design is particularly susceptible to lowering of the bed. The benefit of additional scour protection such as leaving the cofferdam sheetpiles in place after construction was assessed as well.

2.1 Bed Material Characteristics

The geotechnical data collected by EBA Engineering Consultants Ltd in April, 2003, indicated that the surface of the bed consists of a thin layer of non-cohesive material that is underlain by a cohesive material. This cohesive material contains thick sand lenses near the south bank at piers 1 and 2.

The material on the bed surface consisted of a mixture of gravel, cobbles and boulders. The thickness of this non-cohesive material ranges from 0.1 m at the location of the first pier from the south ferry landing to 0.9 m at the location of the second pier from the north ferry landing but the thickness typically ranges from 0.3 to 0.6 m at the other locations. This suggests that the non-cohesive material is merely a thin veneer that may not actually control the bed levels during floods. It is possible that this material is mobilized during a flood and in many areas the cohesive material is exposed.

No grain size distribution of the non-cohesive material are available at present so only estimates of material sizes can be made from the descriptions. Sand-gravel-cobble mixtures tend to have median grain sizes of about 20 mm and 90 percent of the material would be less than about

100 mm. The critical hydraulic shear stress (the hydraulic shear stress above which the material begins to move) for 20 mm material is about 18 Pa while for 100 mm material it is about 90 Pa. As recommended previously, grain size distributions of this non-cohesive material need to be collected in order to determine whether the non-cohesive material mobilizes during high flows.

The cohesive material is described as a stiff to very stiff clay till which is composed of about 30% clay, 45% silt, 20% sand and 5% gravel. The cohesive strength of the material is about 20 kPa. The material has low to medium plasticity with a liquid limit and plastic limit of about 30 and 15 respectively. The moisture content of the cohesive material is about 10%. It is likely that the water content of the material varies with distance below the surface of the till. The till near the surface likely has a greater water content than the till lower down. For example, a sample of the cohesive material provided by EBA appeared to be softer at one end, possibly due to higher moisture content in the softer material. This higher moisture content would be more representative of the cohesive material when it is wetted due to contact with the water at the bed surface. Thus, the properties of the cohesive material at the surface of the bed may be quite different from the properties in the drier material below and may be more susceptible to erosion.

It is not possible to accurately estimate the critical hydraulic shear stress (the hydraulic shear stress above which significant erosion rates occur) from the above parameters. According to Andres (1983), the critical hydraulic shear stress of cohesive material with 30% clay content is typically less than 8 Pa, but on the other hand it could be as high as 12 Pa to 25 Pa if its cohesive strength of 20 kPa is considered. Recent work by Mazurek (2001) indicates that a “prepared clay” with more or less the same soil properties as that at the bridge site had a critical shear stress of about 50 Pa. It is evident that it is difficult to be precise about the erodibility potential of cohesive materials because it is difficult to directly quantify the in-situ critical hydraulic shear stress and erosion rate of a cohesive material even with specialized laboratory equipment. The tests that have been developed are not standardized and produce highly variable results that may not be directly useful for scour design at prototype scale.

Some indication of the stability of the bed can be determined from an evaluation of the in-situ behaviour of the bed material. A comparison of bed levels in the ferry waterway obtained from navigation charts (survey date unknown) with those surveyed in 2002 indicate that the bed elevation near the south landing may have decreased by as much as 3.5 m in the time between the surveys. Comparisons of the bed stability at specific cross section locations reported previously (Trillium, 2002) also found differences in bed elevations of up to 3.0 m. It is unclear whether these apparent changes were due to (1) a lack of precision in horizontal control, (2) movement of non-cohesive bed material, or (3) erosion of cohesive material. Thus, an evaluation of bed stability on the basis historical cross-section changes is inconclusive.

The in-situ behaviour of the river bed near the ferry landings suggest that the resistance of the bed material to scour is quite high since no distinct scour holes are evident at the noses of the landings where the shear stress is amplified. The analysis of shear stress amplification by Andres (1983) and Molinas et al. (1998), and the evaluation of the flow geometry near the sheet piles protecting the ferry landings, indicate that the maximum bed shear stress at the landing is likely between 4 and 5 times the bed shear stress of the approach flow. The maximum bed shear stress of the approach flow at the landings with a local energy slope of about 0.0001 and flow depth of

4 m is about 4 Pa, therefore the maximum bed shear stress developed at the landing is estimated to be 16 – 20 Pa. This suggests that the critical shear stress of the bed material is likely greater than 16 Pa. However this estimate is somewhat equivocal, because the resistance to scour may be due to the layer of non-cohesive material above the cohesive material. In this light it would be useful to: (1) measure the grain size distribution of the non-cohesive bed material so that its stability can be assessed and (2) determine if the clay is exposed to the flow near the sheet piles and if it is directly resisting scour.

2.3 General Scour

General scour occurs when the general bed level is lowered due to net transport of bed material out of the reach. This generally occurs if the bed is erodable and in reaches where the sediment transport capacity of the river is greater than the amount of material transported into the reach. Two main general scour processes have been identified, (1) local constriction scour and (2) general degradation.

Local channel scour may occur at constrictions such as the ferry crossing when local flow velocities increase the bed material transport rate and the bed responds by lowering itself locally. Some of this type of scour is evident at the site but the historical channel surveys are not precise enough to determine the amount. In any case, the bridge does not constrict the channel any further than the existing ferry landings so the bridge is not expected to increase constriction scour. Even so, present bed levels in the constriction may be reduced to the level of the cohesive material during high flows if the non-cohesive surface material is mobilized. This does not appear to be the case, however, because the maximum shear stress of the flow in the constriction is estimated to be about 8 Pa which is less than the estimated 18 Pa needed to mobilize the surface material. Never-the-less, measurement of the grain size distribution of the non-cohesive bed material is needed to determine the stability of this material under design flow conditions.

The other general scour process is general degradation of the channel. This can occur (1) due to the upstream progression of a steep reach of the river where head cutting of the channel leads to massive lowering of the bed or (2) due to the interruption of sediment supply from upstream which produces a net transport of bed material out of the reach. The channel profile between Fort Providence Rapids and the proposed bridge site suggests that the rapids formed by a head cutting type of process. However, the location of these rapids has been stable for many years and the lag deposits (boulders) lining the bed in the rapids suggest that the bed in the rapids is stable and not likely to progress upstream.

The proposed bridge site is downstream of Great Slave Lake which acts as a sediment trap. The river, therefore, transports very little sediment into the reach from upstream and, given the local hydraulics at the bridge site, the transport capacity of the river is greater than the sediment inflow. Thus, if the bed material in the vicinity of the bridge would be mobile, there would be a net transport out of the reach, and general degradation would occur. If the bed material (either cohesive or non-cohesive) would be resistant to movement at flood flows, the bed elevation would be stable.

The water levels in Great Slave Lake would be expected to decrease over time if general degradation were occurring in the reach. Mean annual water levels at Yellowknife Bay from 1941 to 1999 indicate that the water level has actually increased slightly over this time (Figure 2). The lake level records do not suggest that general degradation has occurred in this time period. However, the period of record is short and other effects such as inflows to the lake may mask any long term trends.

The local water level and discharge measurements at the Water Survey of Canada gauge at Dory Point just upstream of the proposed bridge site can also provide an indication of general stability of the local reach. Water levels for similar discharges have remained steady over the 18 years that measurements were available at this site, suggesting that general degradation has not occurred. Again, however, the period of record is short and thus it may not be indicative of long term trends.

2.3 Pier Scour

Scour occurs at bridge piers because of the amplification of the bed shear stress due to the acceleration of flow around the piers. The magnitude of pier scour that would develop is primarily a function of the pier width, the flow intensity, and the angle of attack of the flow (pier skew angle). Scour occurs when the shear stress at the base of the pier exceeds the critical hydraulic shear stress of the material around the pier. A scour hole typically develops at the nose of a pier when the pier is aligned with the flow direction. The presence of a spread footing may limit the scour depth to the top of the footing if it is wide enough and deep enough; but, if the footing is too shallow, the wider footing will produce greater scour depths than the pier itself would produce. As the angle of attack of the flow increases the scour hole widens as well as deepens and the location of the maximum depth shifts toward the rear of the pier on the side facing the flow.

The literature suggests that for specific situations the shear stress at piers is amplified by a factor of between 1.1 (Andres, 1983) and 11 (Chiew, 1995) times the shear stress of the approach flow. However, a summary of laboratory scour data in non-cohesive materials by Melville and Coleman (2000) indicates that pier scour is generally initiated when the approach velocity is about one-half of the critical velocity for the material around the pier. This velocity ratio is equivalent to a shear stress amplification of 4.0, since shear stress varies with the square of velocity and scour is initiated when the critical shear stress is exceeded at the pier. On this basis, a shear stress amplification factor of 4.0 has been adopted herein for the proposed pier geometry.

The bed shear stress of the approach flow at the piers is as high as 8 Pa at the design flow of 10,500 m^3/s with a local energy slope of about 0.0001 and a depth of about 8 m. Therefore the maximum bed shear stress at the piers is likely about 32 Pa. Thus, on the basis of the estimated critical shear stress of 16 Pa or greater at the ferry landing, it is possible that scour could occur around the base of the pier. On the other hand, the critical shear stress of the cohesive material that was estimated at the ferry landing is a minimum estimate and it is quite possible that the critical shear stress could be much higher. Given that the critical shear stress could be as high as 50 Pa, it is possible that little scour will occur for this pier geometry. All in all, the assessment of the erodibility of the cohesive material is still quite speculative, but this evaluation does not completely rule out scour at the piers. Therefore, to be conservative, the effects of scour should be assessed and, if the effects are significant, protection against scour should be incorporated into the design.

Should scour occur at the base of the pier, the depth of the scour and the extent of the scour hole depends on the shear stress amplification and the mobility of the bed material. As the scour hole develops, the shear stress in the hole is reduced and when this shear stress approaches the critical shear stress of the bed material, the scour stops. For the case of cohesive materials, armouring of the scour hole (accumulation of coarse material that is greater than the median size of the ambient material) does not occur and the ultimate scour depths are expected to be as great as those which occur in sand (Briaud et al., 1999 and Ting et al., 2001). According to the CSU method (Richardson et al., 1990), the ultimate scour depth at the nose of the 6.9 m wide standard piers could be as high as 6.0 m assuming that: (1) the bed material is erodable; (2) the footing width does not affect the scour; (3) the flow is aligned with the pier, and (4) the sloped pier nose reduces scour by a factor of 0.75. With these same assumptions, the scour depth at the nose of the 7.3 m wide central piers may be as high as 6.3 m.

As discussed in Section 2.2, general degradation of the cohesive material is not apparent, indicating that the critical shear stress is not exceeded and only clear water scour will occur at the piers. The CSU method does not require input of bed material size or critical shear stress so it does not explicitly account for situations where the approach shear stress is less than the critical shear stress. With Melville and Coleman's (2000) approach, however, the live-bed scour depth at a pier is calculated as 2.4 times the pier width, and several factors are used to adjust this value for various conditions that include pier shape and pier alignment. For clear water scour, the approach velocity, V_a is less than the critical velocity, V_c of the bed material. In this situation the live-bed scour depth is reduced by the value of the ratio, V_a/V_c. Since velocity varies with the square-root of shear stress, V_a/V_c can be determined from the ratio of approach flow shear stress to the critical shear stress, $\hat{o}_a / \hat{o}_c$.

At the proposed bridge site, the maximum approach flow shear stress is 8 Pa. This is only one-half of the minimum estimated critical shear stress of 16 Pa and so V_a/V_c. 0.7. Thus, the maximum scour depth is expected to be only 70% of the live bed scour depth or 1.7 times the pier width. Based on this method, the ultimate scour depth at the piers, not including the effects of footing width and pier alignment, is expected to be 8.8 m and 9.4 m respectively for the standard and central piers. These scour depths are somewhat higher than the ultimate scour depths obtained from the CSU approach.

As discussed previously in Section 1.3, the angle of attack of the flow relative to some of the piers is as high as 16°. This angle of attack can cause the scour depth to increase by a factor of 1.2 and will tend to shift the position of the scour hole to the side of the pier exposed to the flow. At an angle of attack of 16°, the ultimate scour depth for the standard piers may be as high as 10.6 m and the ultimate scour depth for the wider central piers may be as high as 11.3 m. These scour depths would extend below the level of the bottom of the footings. The presence of the proposed footing will limit the scour adjacent to the pier but the scour will extend past the edge of the footing.

Shallow spread footings will become exposed to the flow if scour progresses around the edges of the footing. Should this occur, the scour depth will increase due to the increased effective width of the structure. For example, with flow aligned along the pier, it is estimated that the proposed 12.5 m wide footing, which is buried so that the top of the footing is 1.0 m below the bed, would increase the potential scour by about 10% (Melville and Coleman, 2000). If no other scour protection is adopted, the top of the footing would need to be at least 75% of the pier width below the bed to stop the footing from increasing the scour at the noses of the piers. However, at this burial depth, scour may still develop down the sides of the footing if the footing is too narrow. Spread footings should be buried at least one pier width deep and be at least four times wider than the pier to ensure the scour hole is contained on the top of the footing. Due to the width of the proposed piers, the footing burial depths derived from the above analysis are quite large. However, shallower footing depths are allowable as long as some form of scour protection is adopted. The tops of the footings should be at least 1.0 m below the bed surface so that the footings are not exposed to the flow if the non-cohesive material is mobilized. The natural bed elevations above the footings should be restored using Class1 riprap with a median diameter of 0.3 m.

The proposed footing widths are 12.5 m wide, thus the footings project about 0.35 to 0.40 times the pier widths from the sides of the piers. Melville and Coleman (2000) indicate the footings should extend at least 1.5 pier diameters (about 11.0 m) out from the pier in all directions to limit scour to the top of the footing. Thus the proposed footings are not sufficient to limit scour and thus additional scour protection is required.

The rate at which scour occurs depends on the type of bed material at the base of the pier. If the bed material at the pier is non-cohesive, the rate of scour will be high and a deep scour hole will form relatively quickly. If the bed material is cohesive, scour will develop at a much slower rate. Measured scour rates in cohesive materials range from 1 to 1000 mm/hr (Briaud at al. 1999). The scour rate tends to increase with the excess shear stress above the critical shear stress of the material; thus, the highest rates reported likely occurred in weak clays subjected to high shear stresses. The cohesive material at the bridge site is more likely to be at the low end of the range because it has a relatively high critical shear stress and the shear stress is not likely to be exceeded by a large factor. Thus a scour rate in the order of 0.02 m/day is expected to occur if the critical shear stress of the cohesive material is exceeded.

As a scour hole develops, the scour depth approaches the ultimate scour depth asymptotically. Therefore, if the critical shear stress were exceeded continuously for an entire open water season, the scour depth would be about 3.1 m for both the standard and central piers if the initial scour rate was 0.02 m/day. However, if the initial scour rate was 0.05 m/day, as much as 5.3 m of scour could occur at a standard pier and 5.6 m at a central pier. Given that the storage effects of Great Slave Lake causes high flows to maintained for long periods, it is possible that significant scour could occur during a single open water season if the critical shear stress is exceeded.

2.4 Scour Protection

Although significant scour is unlikely, should it occur, the scour holes could develop quite rapidly. Annual monitoring may not be of sufficient frequency to detect serious scour in time to mitigate the effects of the scour. Therefore, scour protection should be incorporated in the pier design.

In the event that scour should extend below the footing, scour protection at the piers would be improved by leaving in place the cofferdam sheet pile used in construction of the footing (cut off at the top of the footing). This would provide additional security by containing the material under the footing. It is beyond the scope of this report to assess the stability of the pier foundations in this situation but sheet piles are not likely to provide sufficient protection against the ultimate scour depth should it occur.

Standard practice is that spread footings alone are typically only used on solid rock where no scour can occur. The Guide to Bridge Hydraulics (TAC, 2001) recommends the use of piles under a spread footing in erodable material as a secondary line of defence against scour. In fact, the best method of scour protection at the proposed bridge site may be to use a pile or caisson type foundation rather than the spread footing approach. These types of foundations typically extend farther into the bed and can be designed to accommodate greater scour depths without special protective measures.

One method which could be used to protect the proposed piers against pier scour is to place riprap aprons around the piers about 1.5 pier diameters out from the edges of the piers. Class 1 riprap with a median diameter of 0.3 m should be placed 11.0 m out from the edges of the piers. The riprap aprons should be 0.6 m thick and may be placed on the bed surface. The riprap should be non-uniform in size to provide the required stability. The Guide to Bridge Hydraulics (TAC, 2001) recommends the following gradation for Class 1 riprap:

100% smaller than 500 mm (200 kg)
at least 20% larger than 350 mm (80 kg)
at least 50% larger than 300 mm (50 kg)
at least 80% larger than 200 mm (15 kg)

The riprap should be angular and as near to equi-dimensional as practicable and should be composed of sound, solid rock. No filter layer is required because the existing non-cohesive material on the bed surface should provide appropriate protection to the clay layer. This layer of non-cohesive material will also provide a rough surface on which to place the riprap which will improve the stability of the riprap.

The potential for failure due to scour is higher at the second pier from the south bank because a 2 m thick sand lens underlies a 3 m thick layer of cohesive material. Thus, if scour were to progress through the cohesive material it may proceed more rapidly through the sand rather than proceeding more slowing through additional cohesive material as at the other pier locations. The effect of the presence of this scour hole through the sand layer near the edge of the riprap apron and the associated wetting of the sand lens on the foundation stability should be investigated thoroughly; and, if necessary, piles which extend down below the sand lens should be included to support this pier.

3.0 Summary of Recommendations for New Pier Geometry

The recommendations resulting from the evaluation of ice loads are as follows:

1) The piers should be designed to withstand a longitudinal bending load on the 63.4° nose slope of 9.0 MN with a transverse load of 1.46 MN and a longitudinal load of 4.5 MN with a transverse load of 3.1 MN.
2) Two-dimensional flow analysis indicates piers may be skewed 16° relative to the flow direction and that ice may strike the flank of a pier at angle of as much as 19° due to wind effects. This will produce loads normal to the sloped pier flank of 3.3 MN. The attendant longitudinal loads along the long axis of the pier would be 10% of the normal loads.

The recommendations resulting from the evaluation of pier scour and general scour are as follows:

1) General scour of the cohesive material is not expected to occur at the bridge site. Analysis of historical water level data provided no indication of channel degradation and the proposed bridge will not constrict the channel further than the existing ferry landings.
2) The non-cohesive surface material is not expected to mobilize in the bridge constriction but the grain size distribution of the surface material should be determined to confirm this resistance to motion and to assess the protective effects of the surface material at the piers.
3) Ultimate scour depths for the proposed pier geometry are expected to be greater than 9.0 m (including the effects of the 16° angle of attack) if the cohesive material is erodable and no scour protection measures are adopted.
4) Analyses of hydraulic shear stresses at the ferry landings and piers indicate that pier scour cannot be ruled out so scour protection measures should be adopted. Thus, the spread footing design should be modified to protect against the ultimate scour levels.
5) The footings should be positioned so that the tops of the footings are at least 1.0 m below the bed so that the footings are not exposed to the flow if the non-cohesive surface material is mobilized. The excavations should be backfilled to the original bed level with Class 1 riprap with a median diameter of 0.3 m.
6) Sheet piles may provide some minor scour protection below the level of the footing but will not protect against ultimate scour levels.
7) Caisson or pile foundations which extend below the expected ultimate scour depth are the preferred method of protection against potential foundation failure due to scour.
8) Riprap aprons may also be used to protect the piers from scour. These aprons should consist of a 0.6 m thick layer of Class 1 riprap with a median diameter of 0.3 m and extend 11 m from the edges of the piers.
9) The second pier from the south bank may require special treatment for scour due to the presence of a sand lens 3.5 m below the surface. The effect of the development of a scour hole through this sand layer near the edge of the riprap apron and the associated wetting of the sand lens on the foundation stability should be investigated thoroughly; and, if necessary, piles which extend down below the sand lens should be included to support this pier.

The ice load and scour components of the pier design appear to have conflicting requirements as to the width of the piers. The ice load analysis indicates that wide sloping piers are preferred while the scour analysis indicates that narrow piers are preferred. Providing riprap scour protection to the wide piers as proposed herein provides a workable solution to the conflicting requirements; however, the performance of this riprap protection should be monitored to ensure that it continues to provide the required scour protection and that it is repaired time a timely fashion if it is found to be damaged.

If you have any questions or comments about the above analysis, please contact me at 780-496-7671.

Sincerely,

Gary Van Der Vinne, M.Sc., P.Eng.
Trillium Engineering and Hydrographics Inc.

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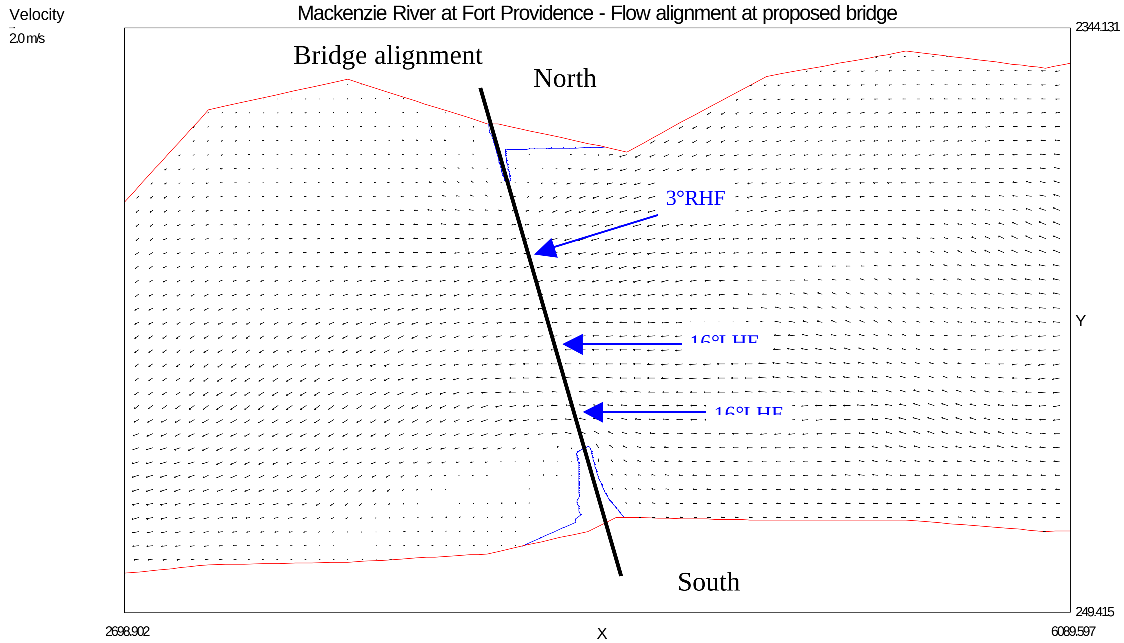


Figure 1 **Velocity vectors obtained from simulation of two-dimensional flow field in vicinity of proposed bridge.**

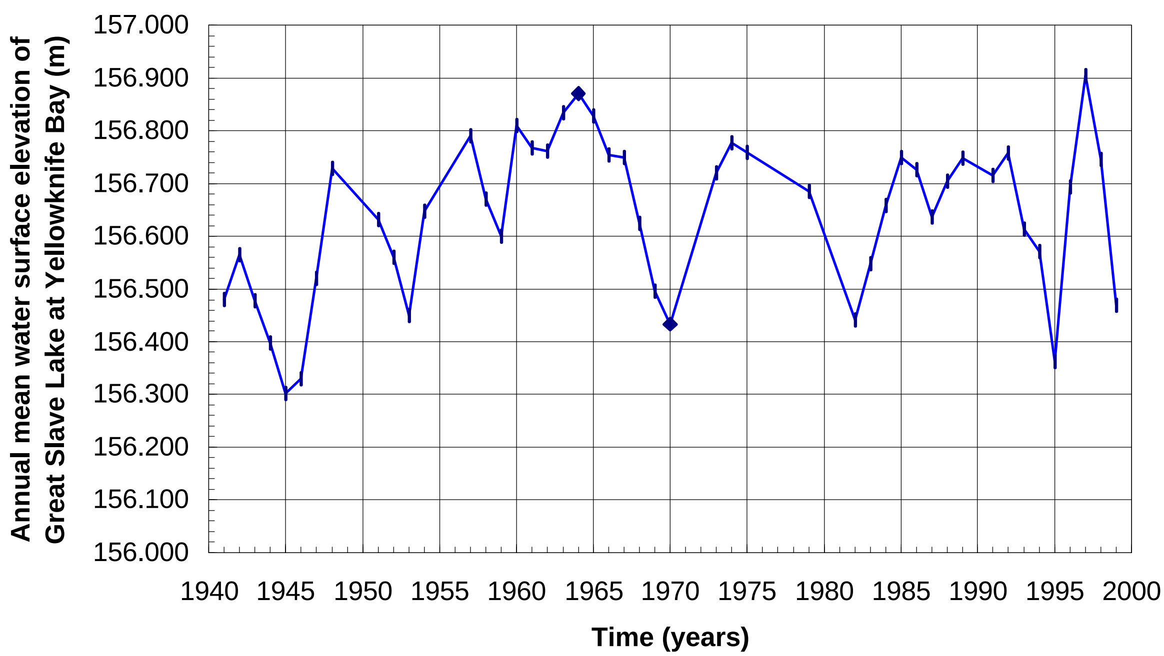
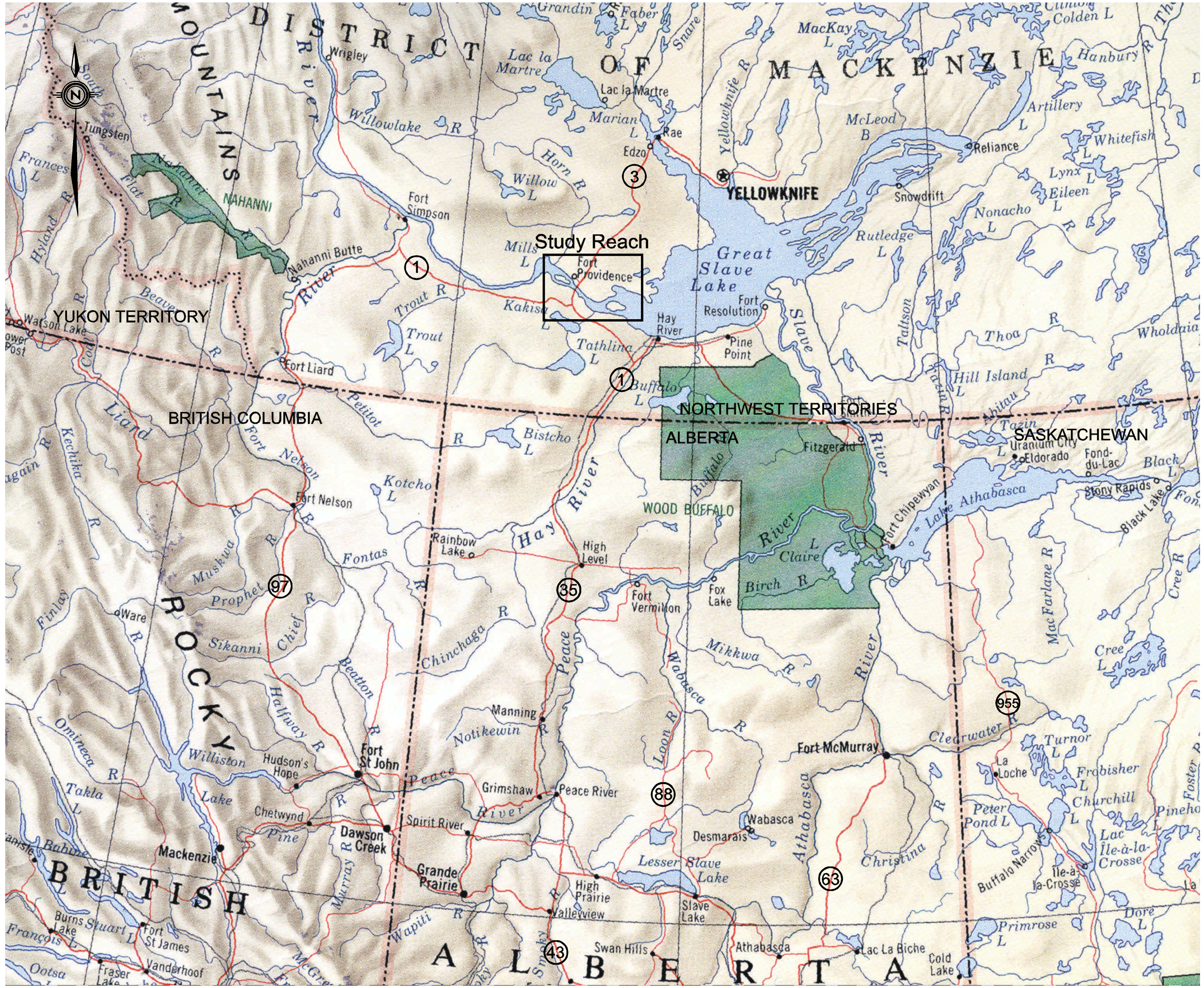


Figure 2 **Variation in annual mean water surface elevation of Great Slave Lake at Yellowknife Bay with time.**



Map: MCR 4032
Scale: 1:5,000,000

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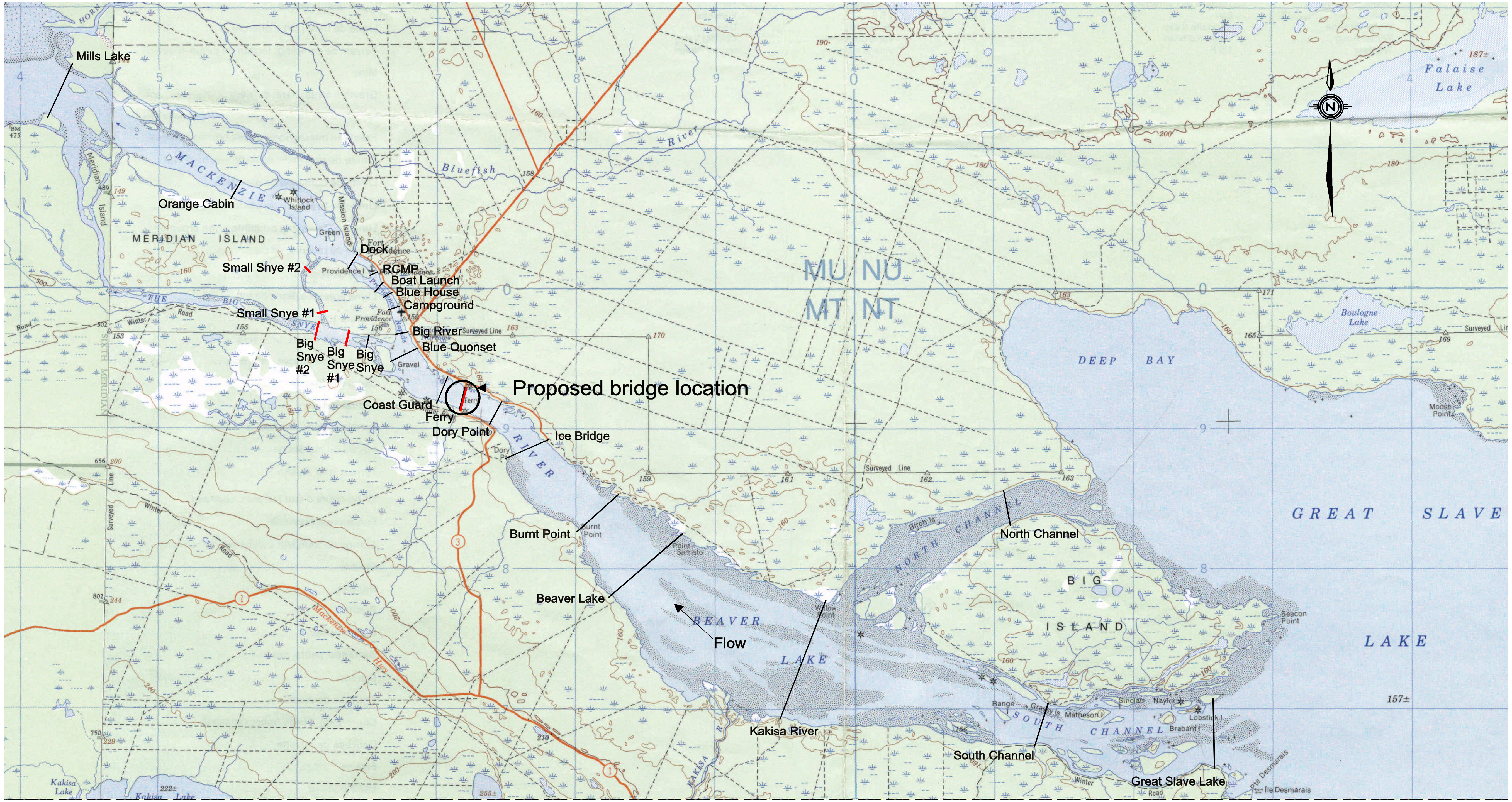
PROJECT: MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE		
LOCATION PLAN		
TRILLIUM FILE	DATE	FIGURE
02-519	OCT. 29, 2002	1.1



Maps: NTS-85E and 85F
Scale: 1:300,000

Previous surveyed sections
2002 Trillium surveyed sections

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CLIENT JIVKO ENGINEERING LTD.		
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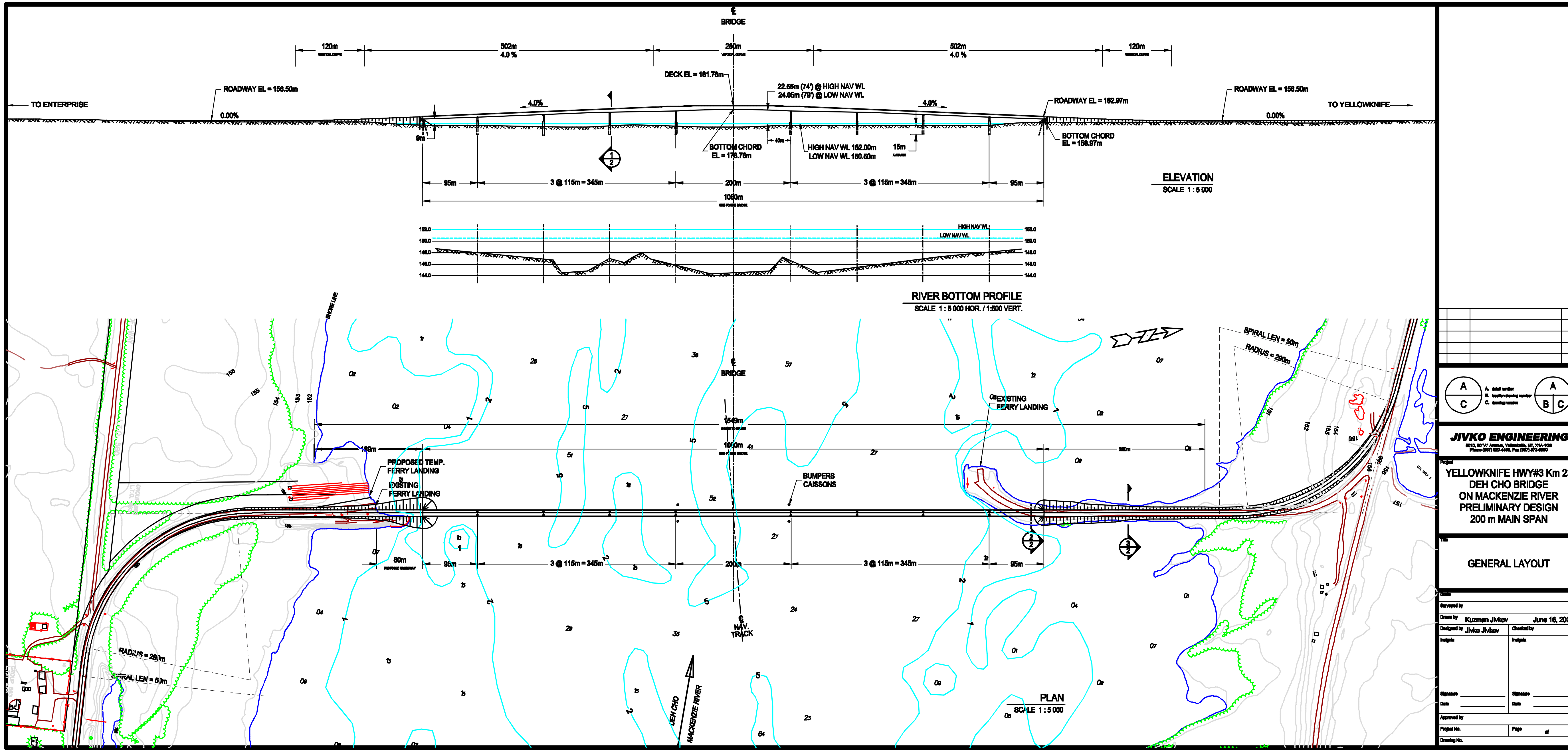
STUDY REACH		
TRILLIUM FILE 02-519	DATE OCT. 29, 2002	FIGURE 1.2



NOTE: Drawing provided by Jivko Engineering Ltd., reprinted at scale 1:7500

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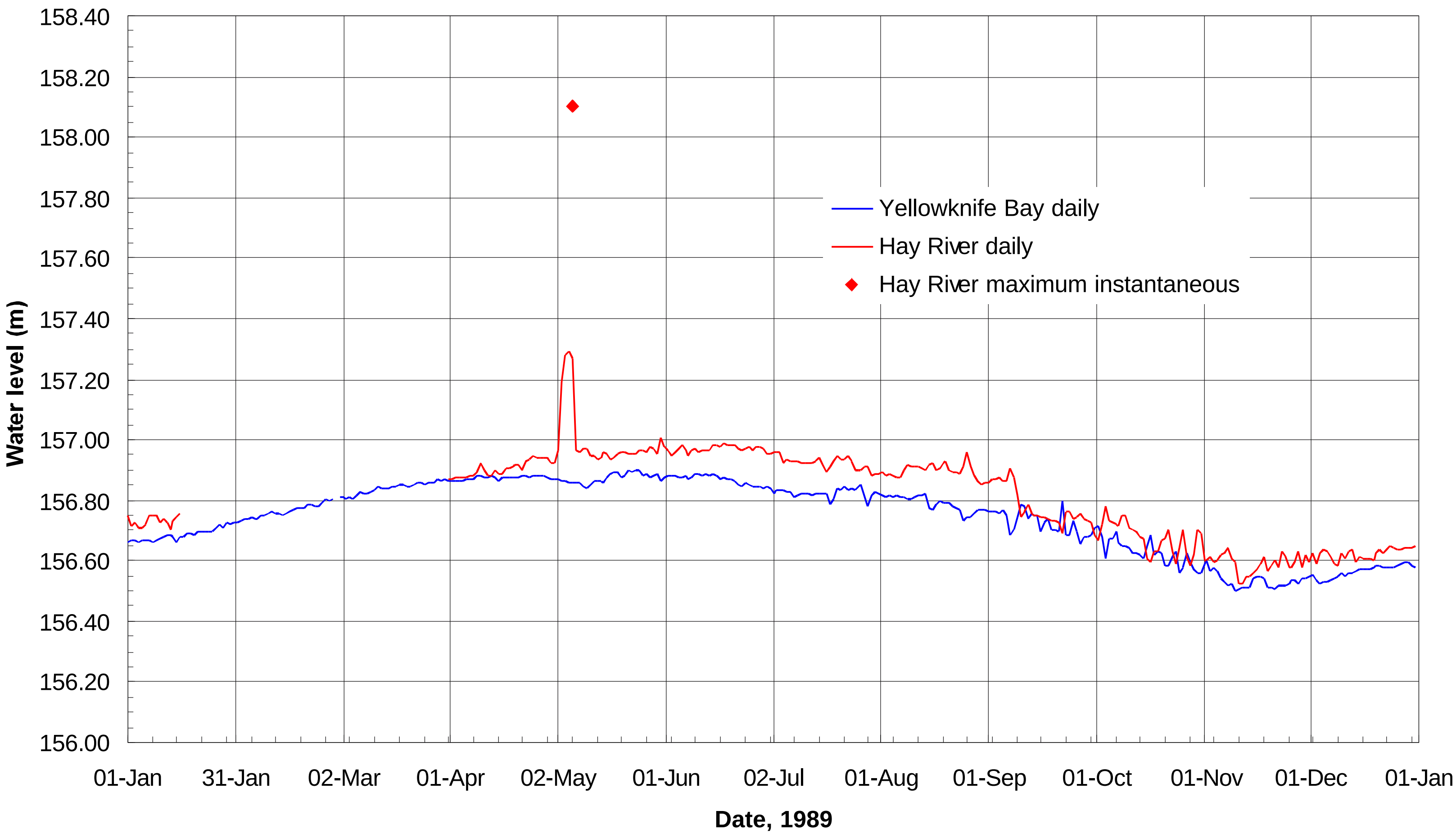
	ELEVATION AND PLAN OF PROPOSED BRIDGE AT FERRY

TRILLIUM FILE	DATE	FIGURE
02-519	OCT. 29, 2002	1.3



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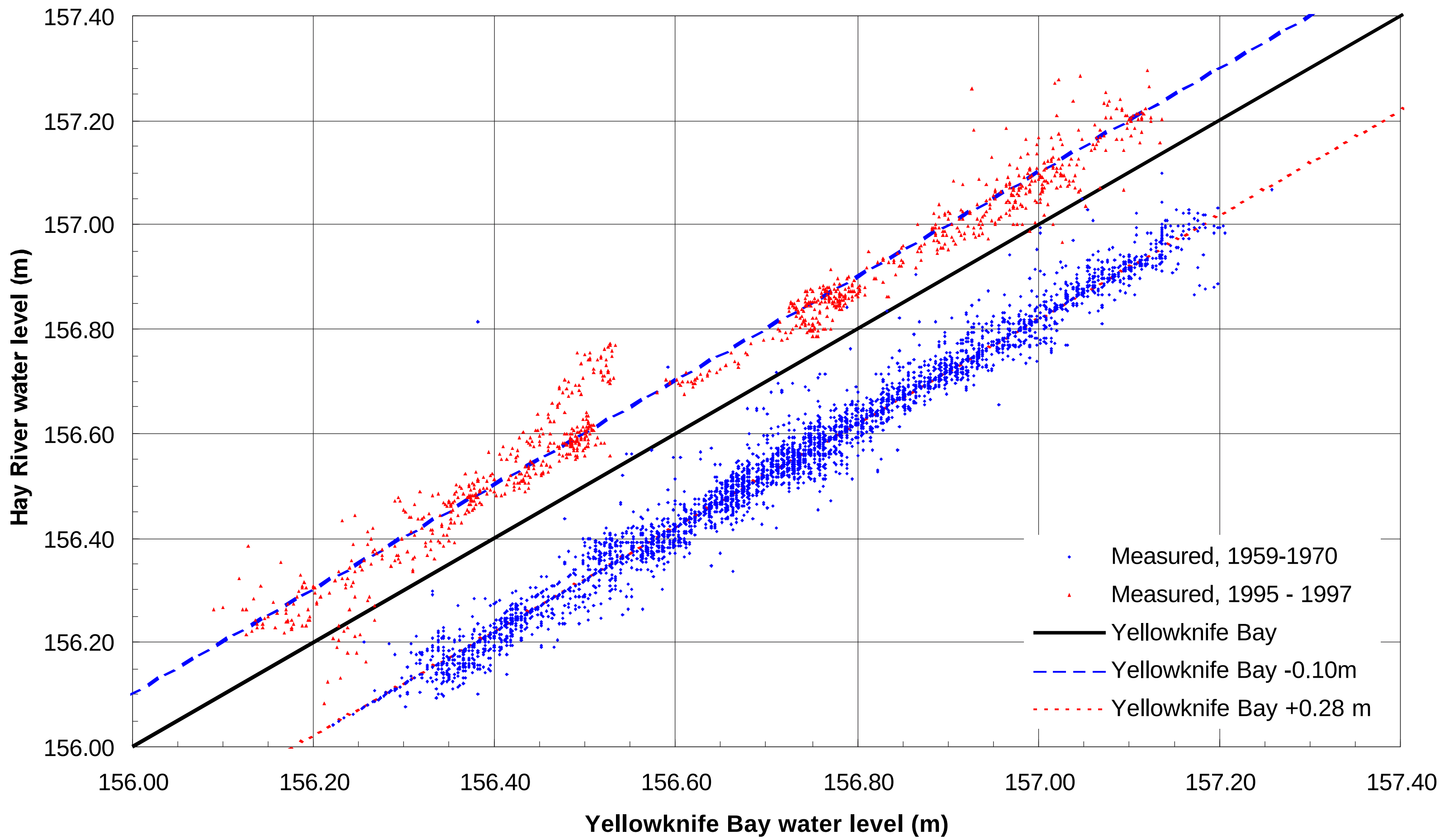
GREAT SLAVE LAKE WATER LEVELS AT HAY RIVER AND YELLOWKNIFE BAY, 1989

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	2.1



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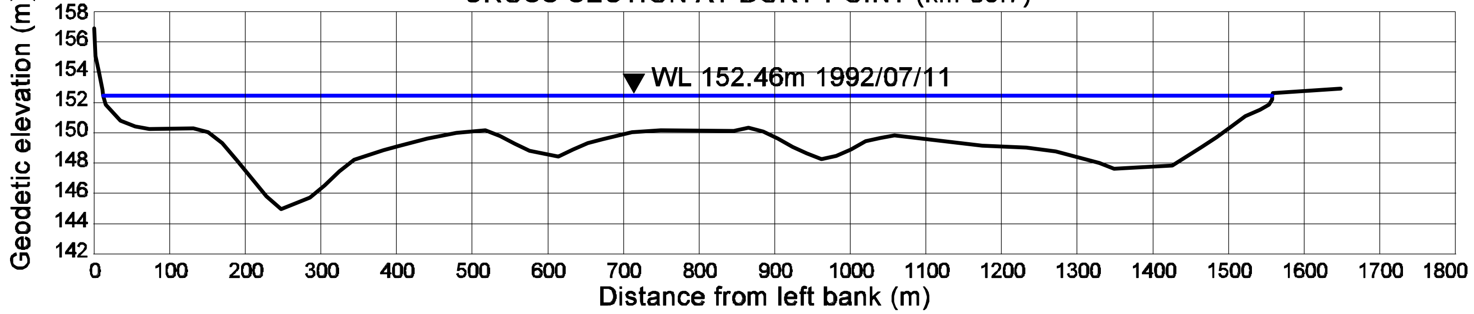
PROJECT
MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE

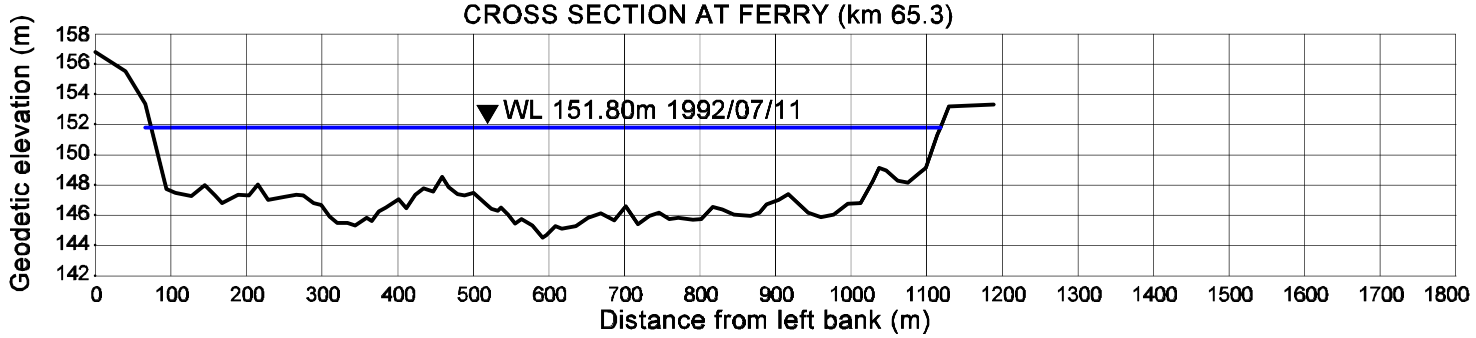
COMPARISON OF GREAT SLAVE LAKE WATER LEVELS AT HAY RIVER AND YELLOWKNIFE BAY

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	2.2

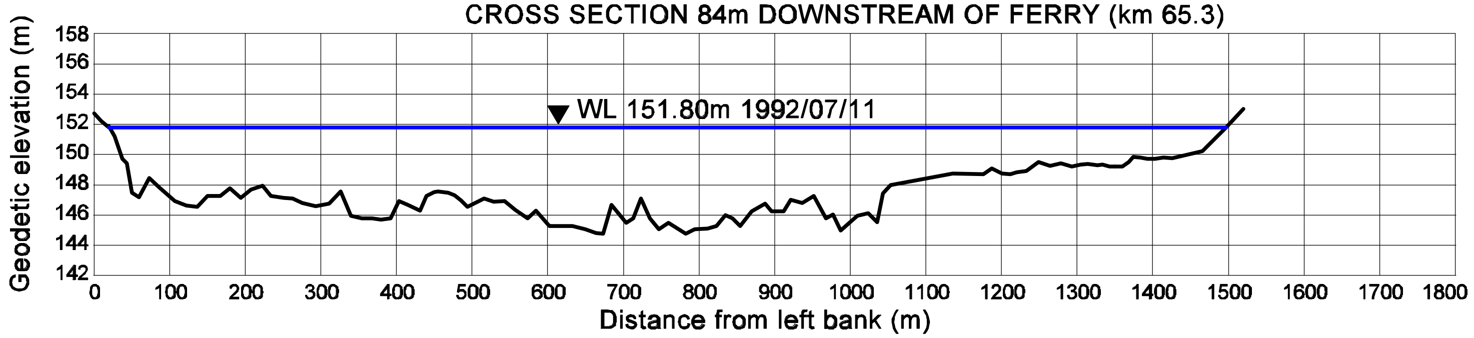
CROSS SECTION AT DORY POINT (km 63.7)



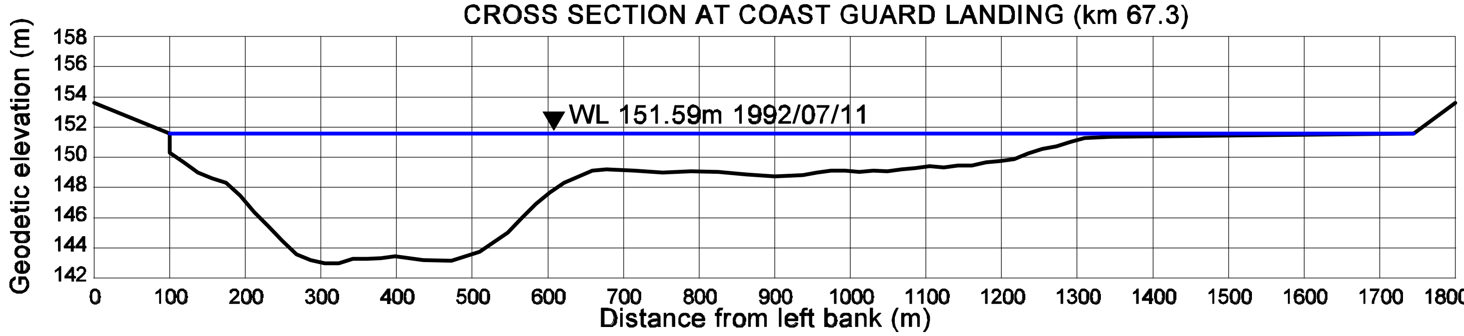
CROSS SECTION AT FERRY (km 65.3)



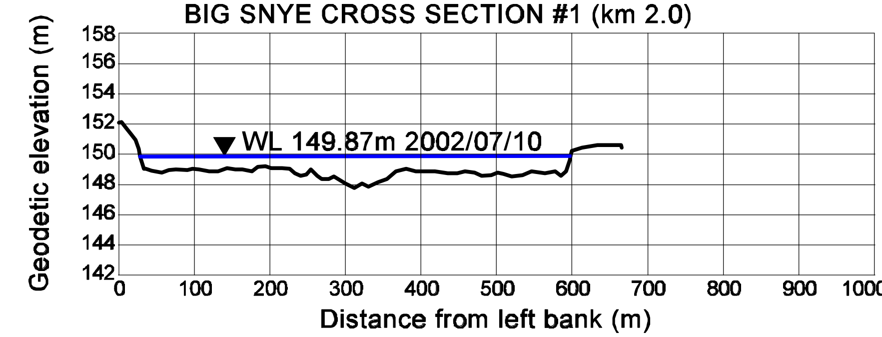
CROSS SECTION 84m DOWNSTREAM OF FERRY (km 65.3)



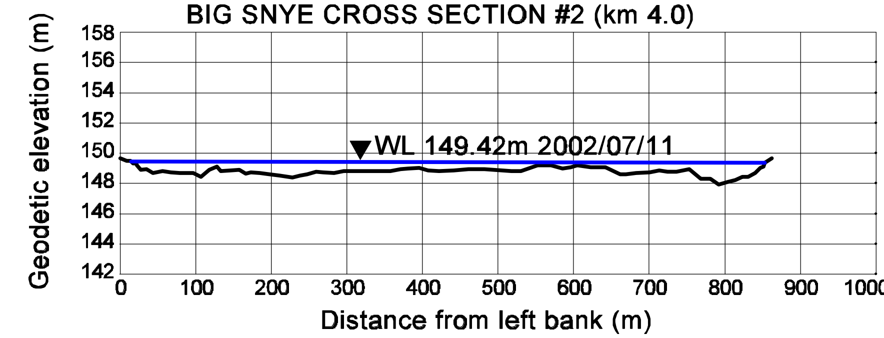
CROSS SECTION AT COAST GUARD LANDING (km 67.3)



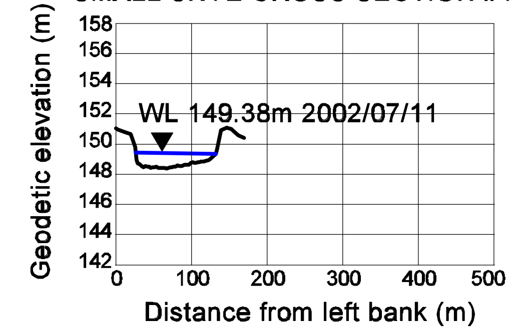
BIG SNYE CROSS SECTION #1 (km 2.0)



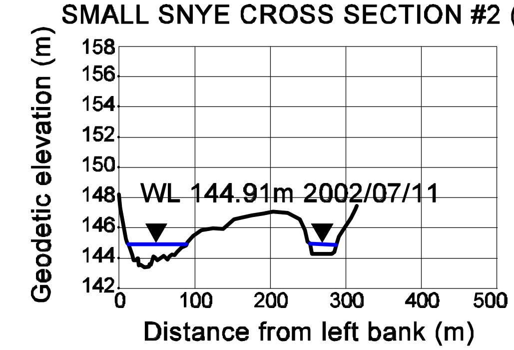
BIG SNYE CROSS SECTION #2 (km 4.0)



SMALL SNYE CROSS SECTION #1 (km 1.0)



SMALL SNYE CROSS SECTION #2 (km 5.0)



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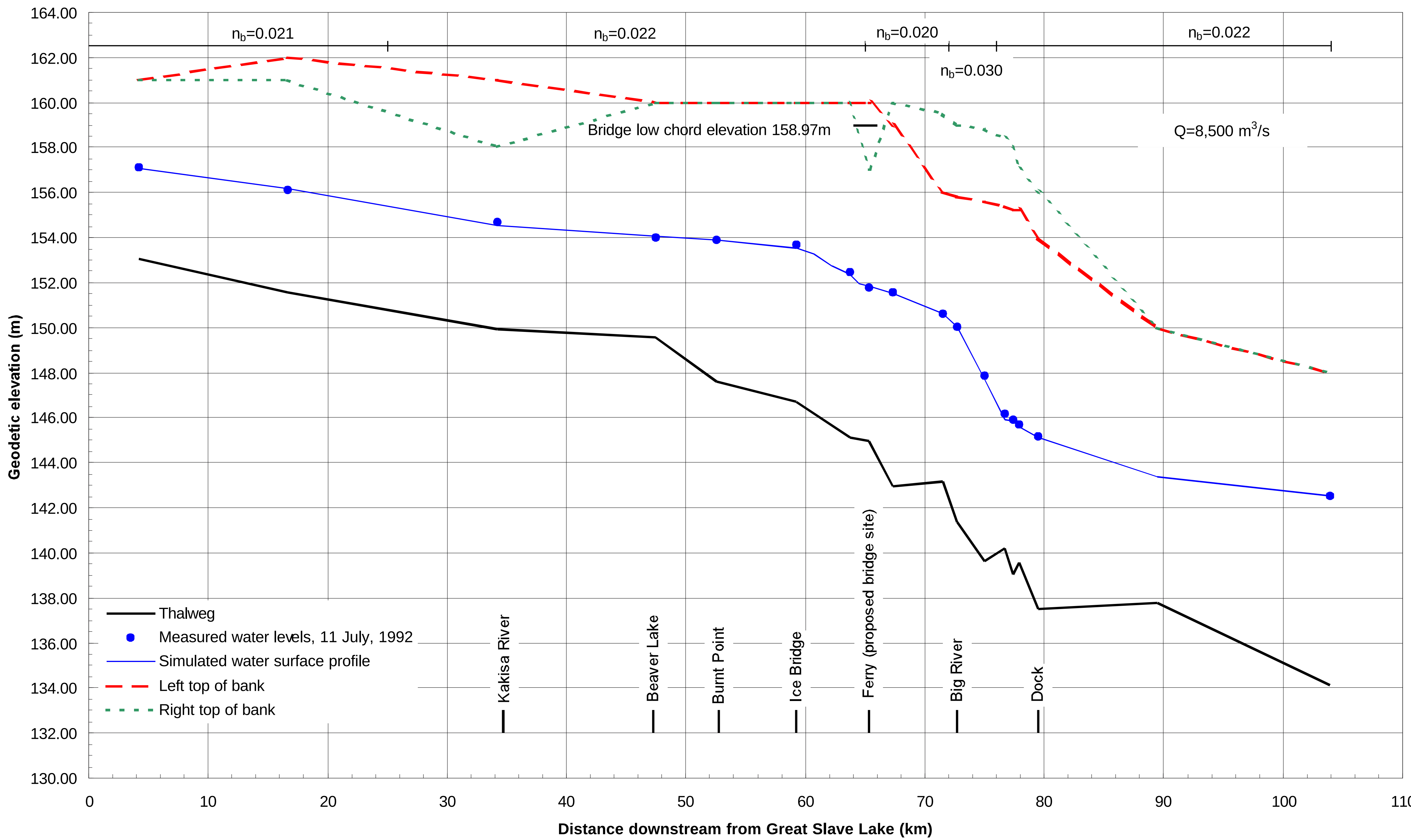
SCALE: H: 1:10,000
V: 1:500

CLIENT	JIVKO ENGINEERING LTD.
PROJECT	MACKENZIE RIVER AT FORT PROVIDENCE FERRY CROSSING
	CROSS SECTIONS IN FERRY REACH AND SNYE CHANNELS

TRILLIUM FILE	DATE	FIGURE
02-519 02-523	July 17, 2002	2.3



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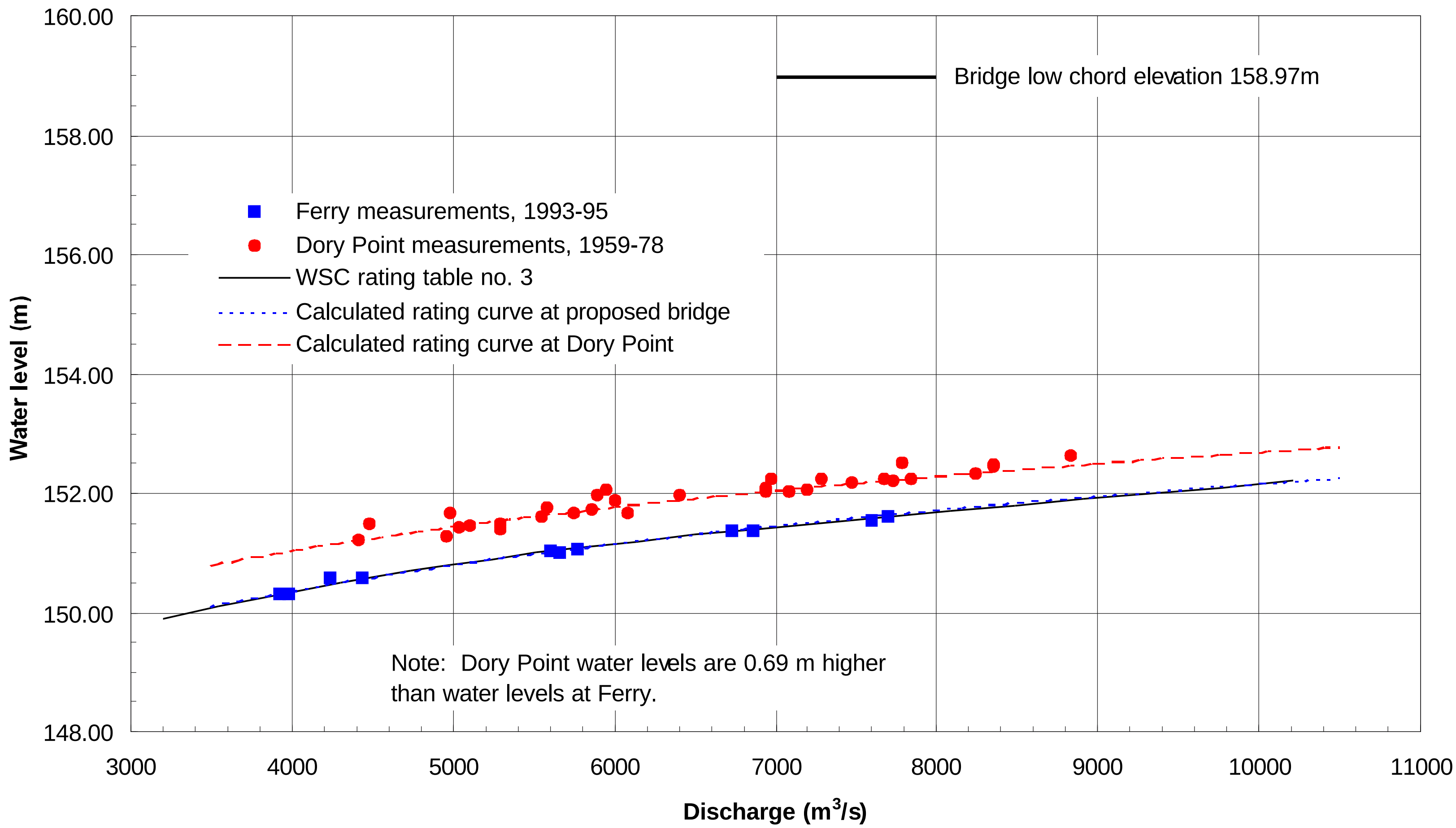
PROJECT MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE		
CALIBRATION OF OPEN WATER LEVEL SIMULATIONS		
TRILLIUM FILE 02-519	DATE OCT. 29, 2002	FIGURE 2.4



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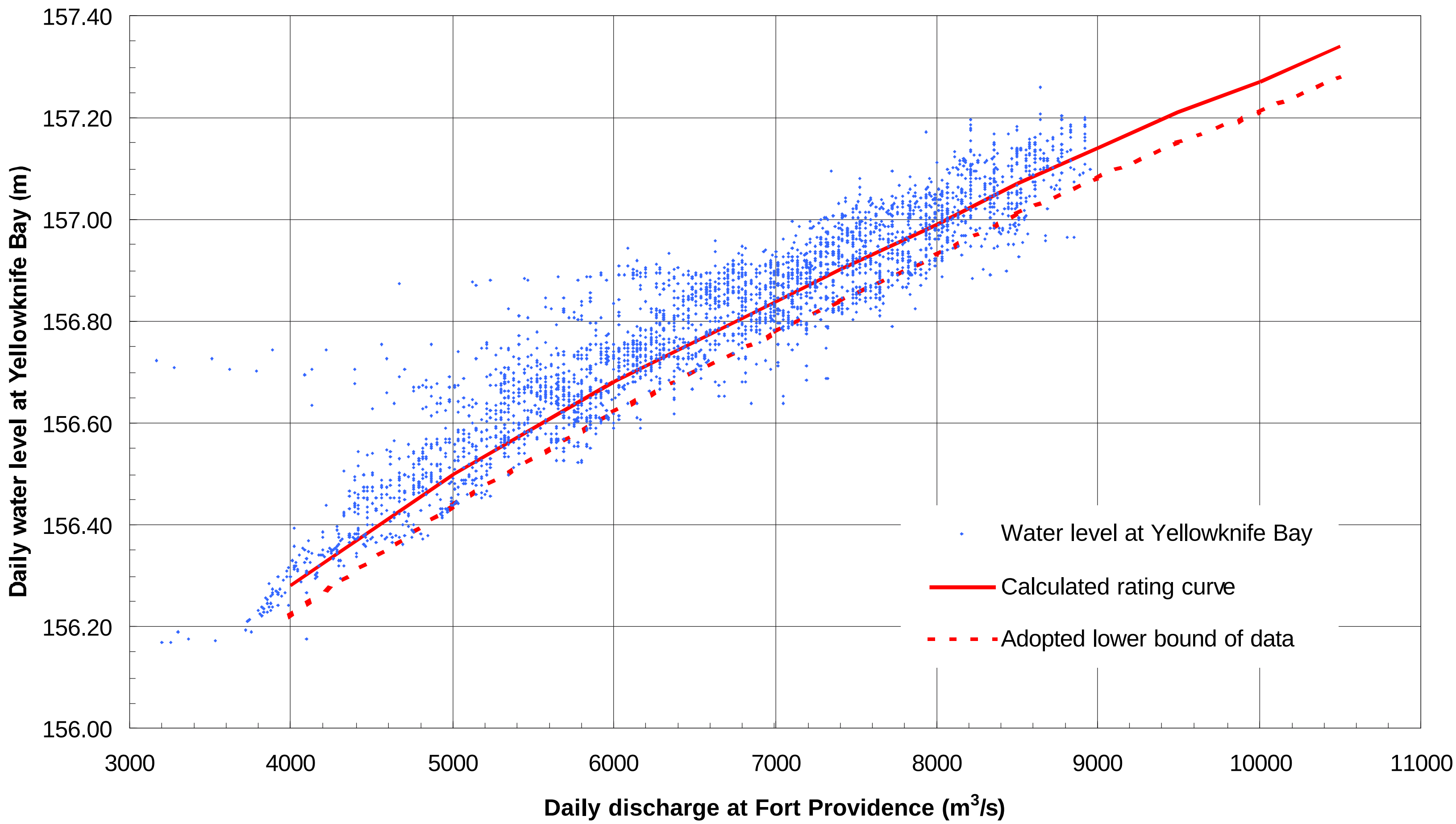
PROJECT
MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE

DISCHARGE RATING CURVES AT DORY POINT AND PROPOSED BRIDGE

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	2.5



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PROJECT
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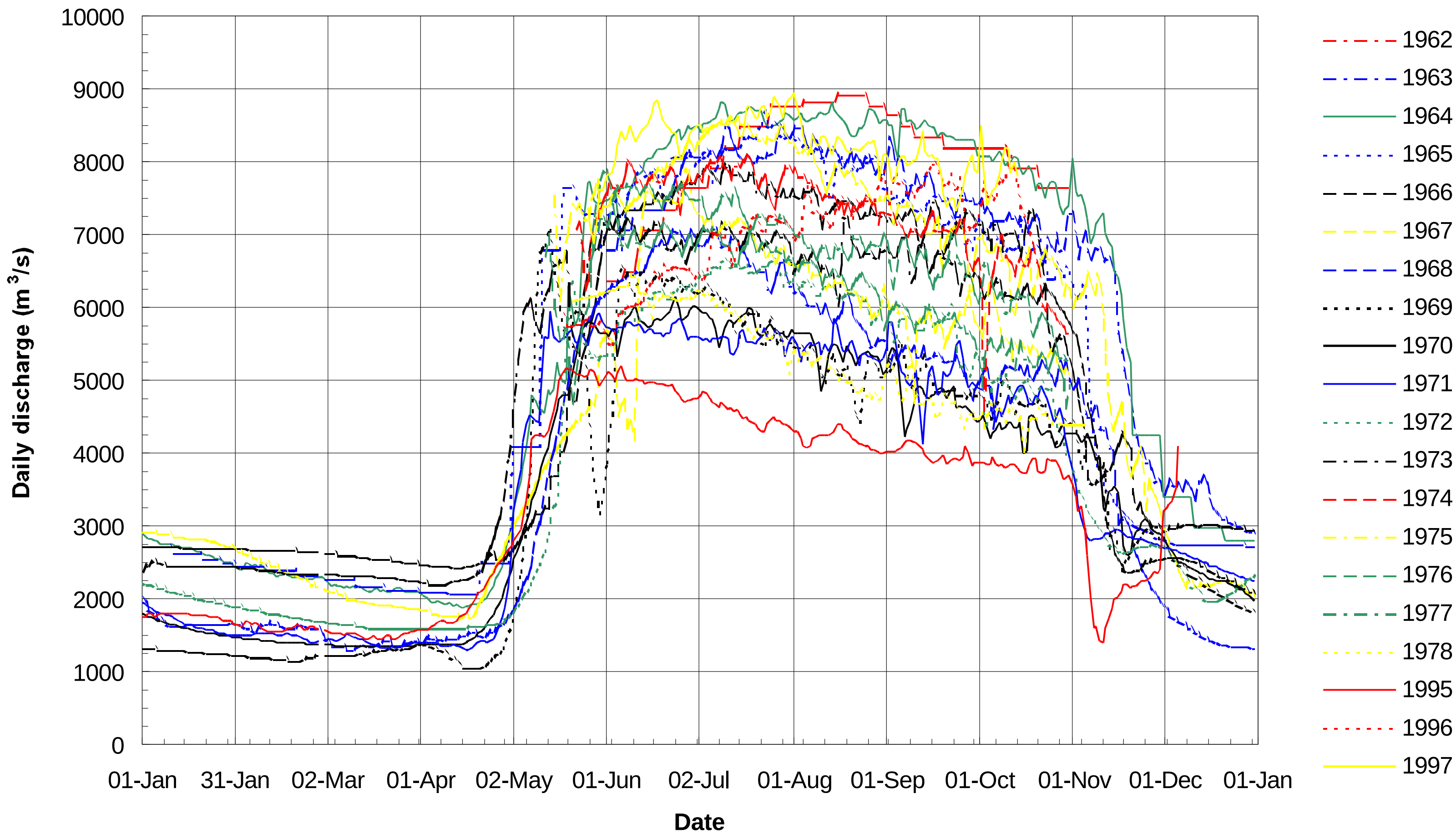
DISCHARGE RATING CURVES AT
OUTLET OF GREAT SLAVE LAKE

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	2.6



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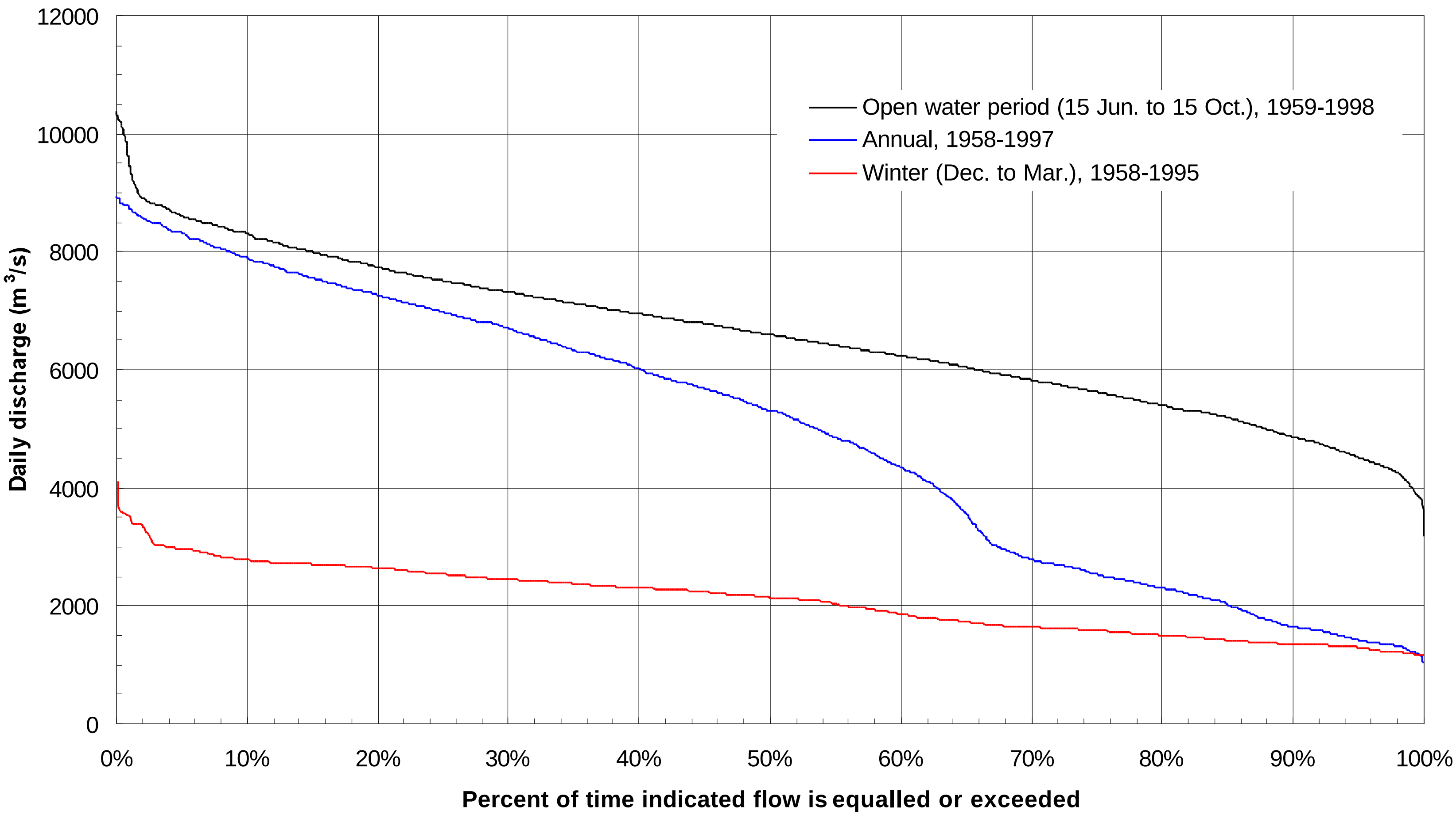
PROJECT	MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE
	ANNUAL DISCHARGE HYDROGRAPHS AT FORT PROVIDENCE

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	2.7



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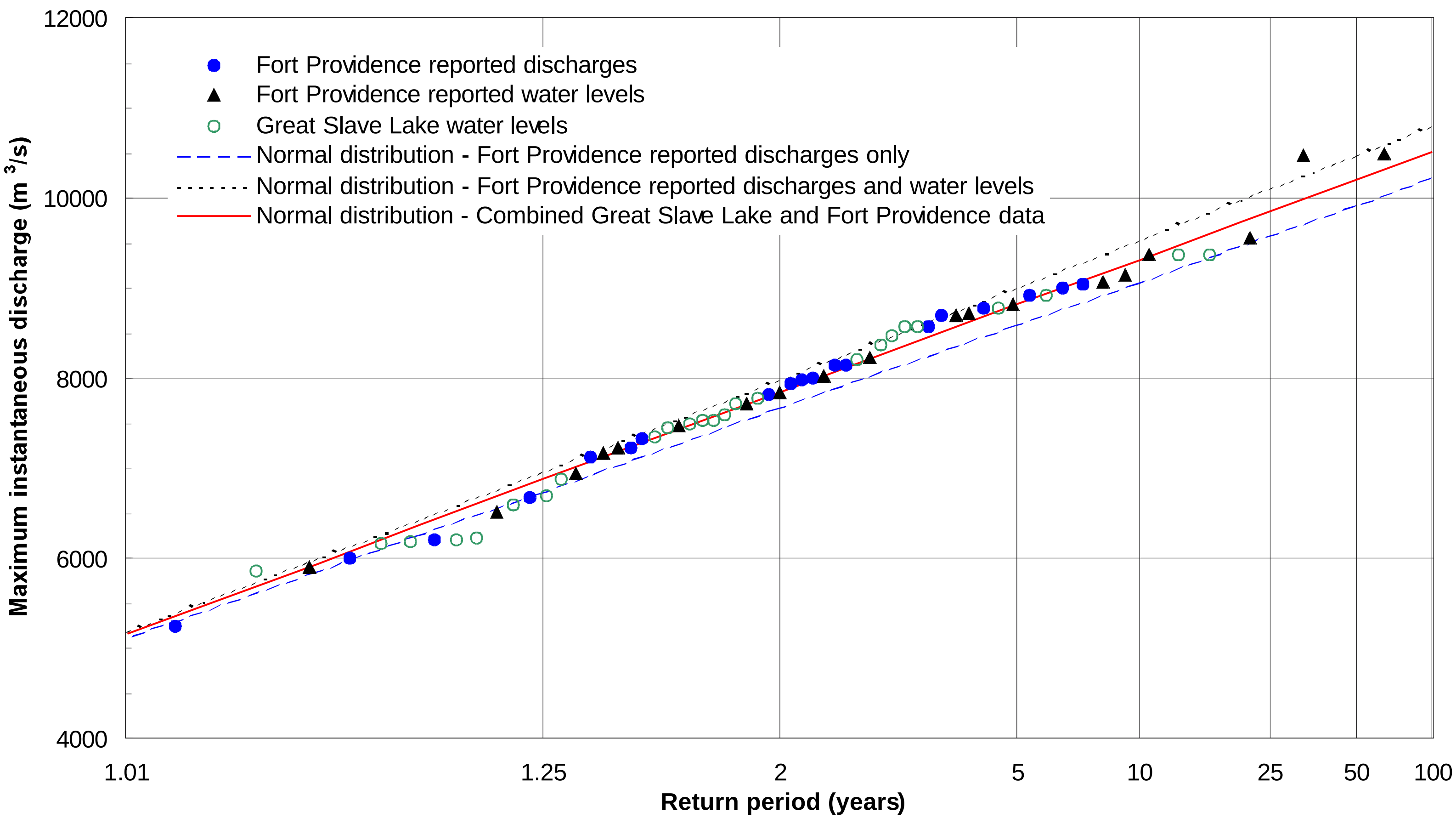
PROJECT MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE		
FLOW DURATION CURVES AT FORT PROVIDENCE		
PROJECT NUMBER 02-519	DATE OCT. 29, 2002	FIGURE 2.8



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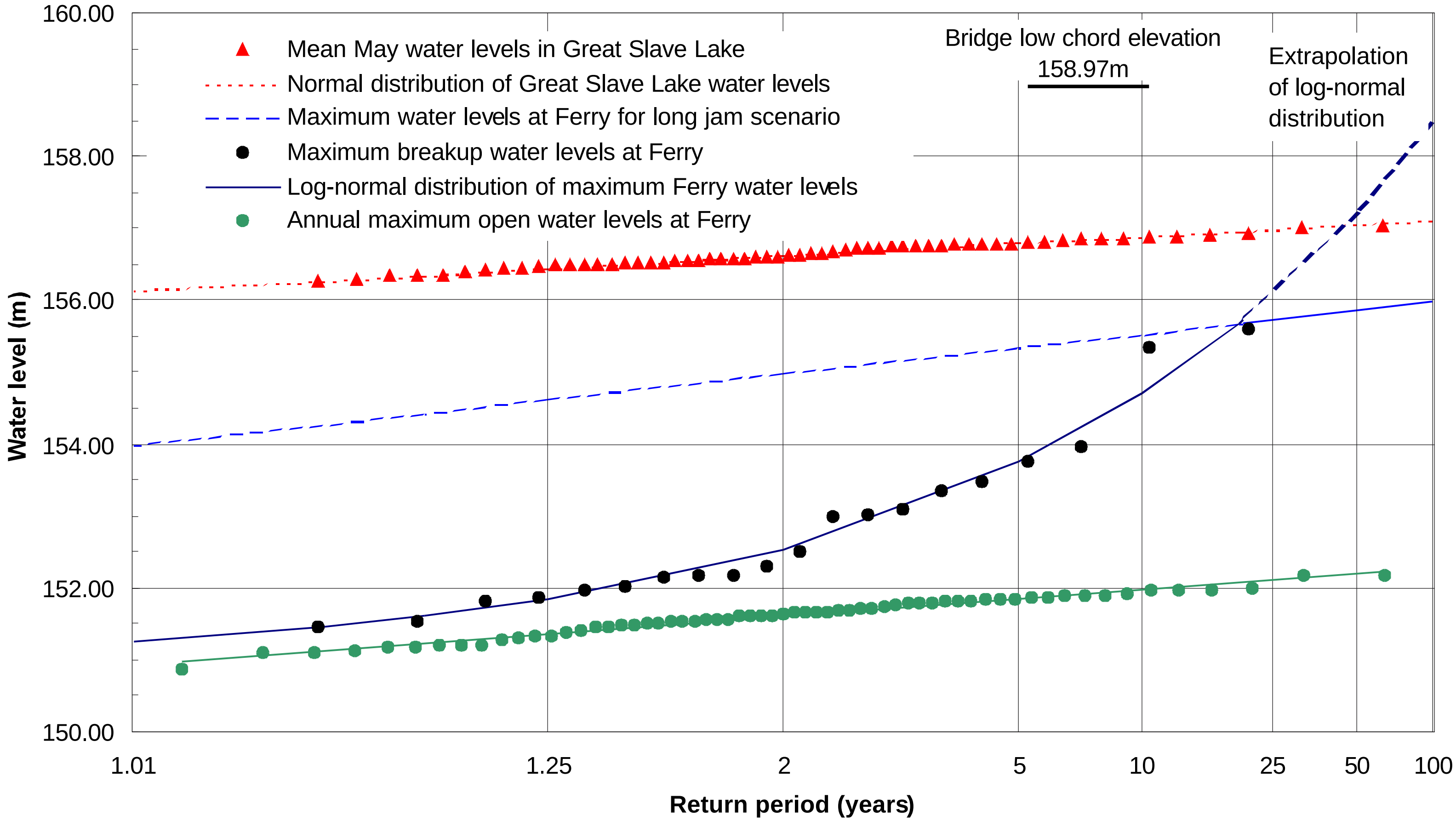
PROJECT
MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE

ANNUAL MAXIMUM DISCHARGE FREQUENCY CURVES AT FORT PROVIDENCE

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	2.9



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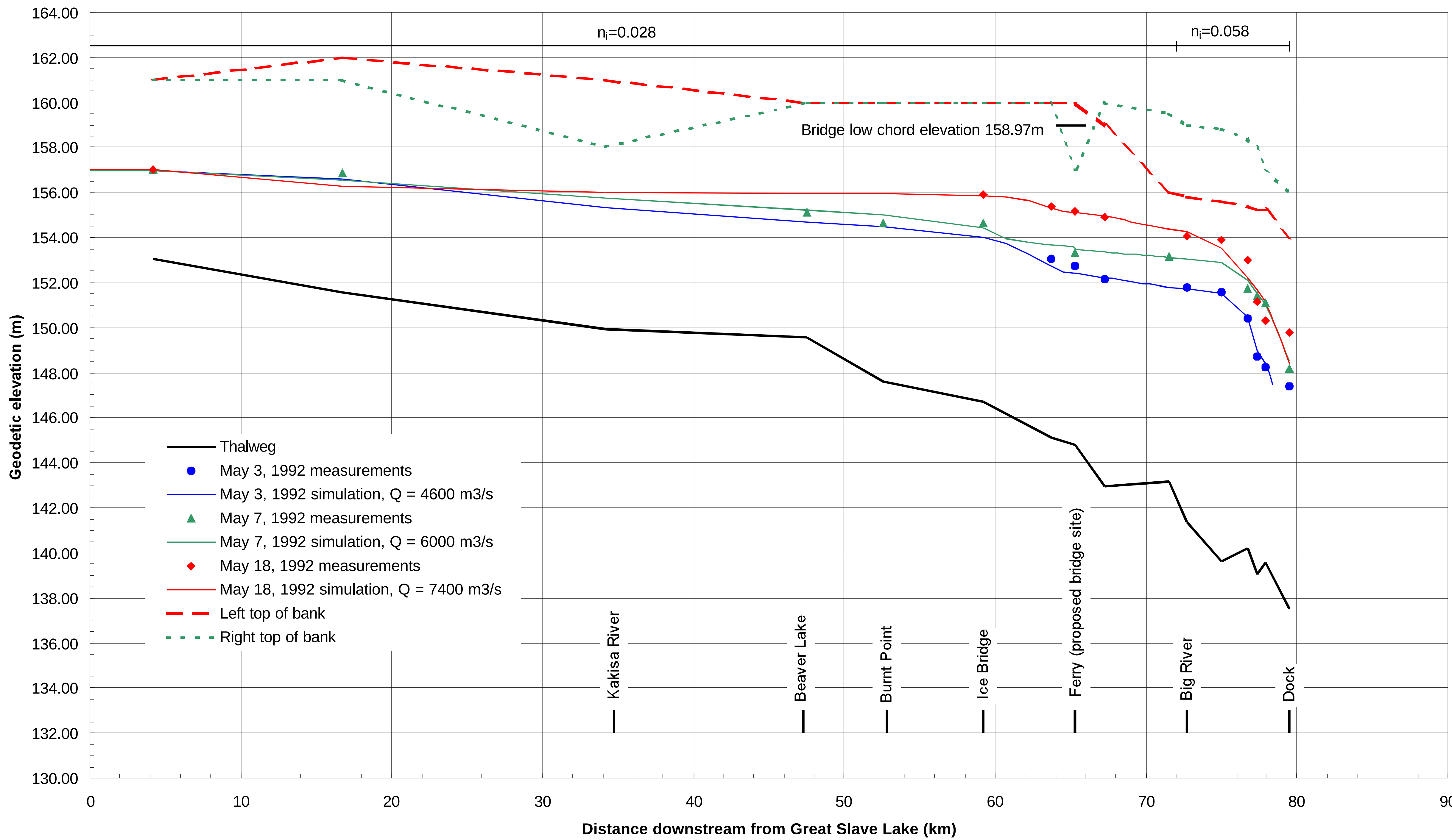
PROJECT	MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE

ANNUAL MAXIMUM BREAKUP WATER LEVEL FREQUENCY CURVES

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	3.1



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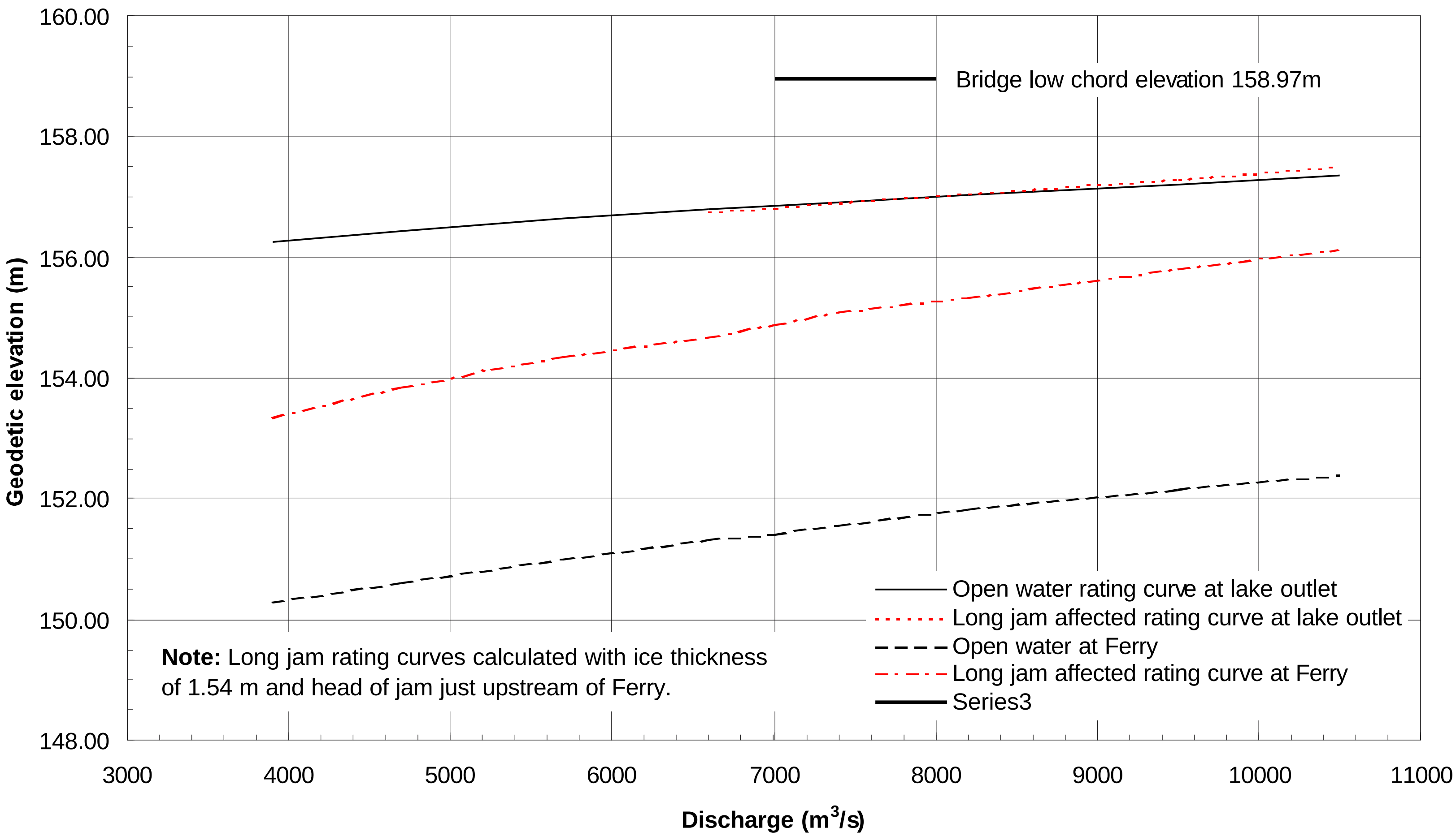
PROJECT MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE		
CALIBRATION OF ICE-AFFECTED WATER LEVEL SIMULATIONS		
TRILLIUM FILE 02-519	DATE OCT. 29, 2002	FIGURE 3.2



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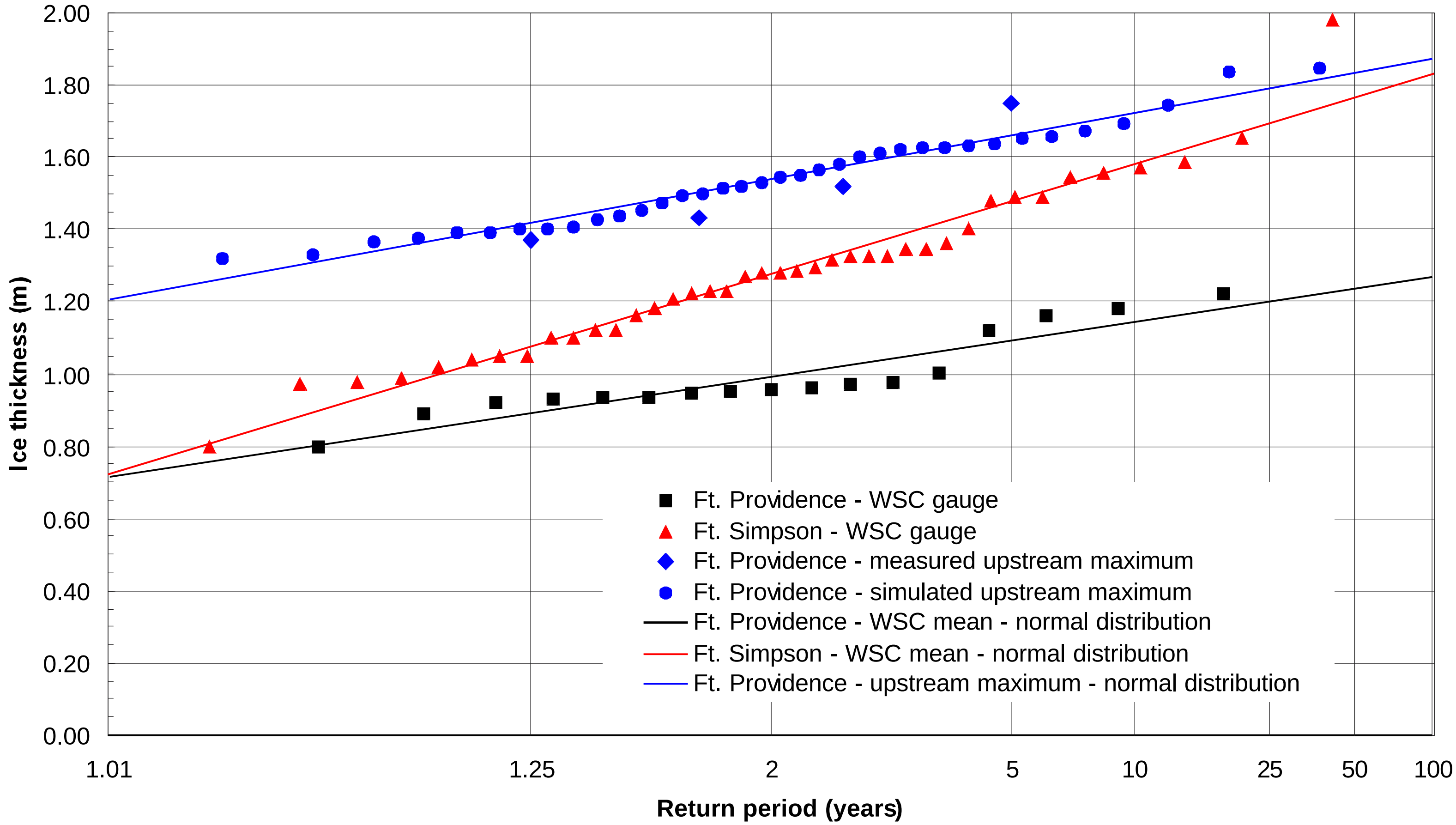
DISCHARGE RATING CURVES AT FERRY AND OUTLET OF GREAT SLAVE LAKE		
PROJECT NUMBER: 02-519	DATE: OCT. 29, 2002	FIGURE: 3.3



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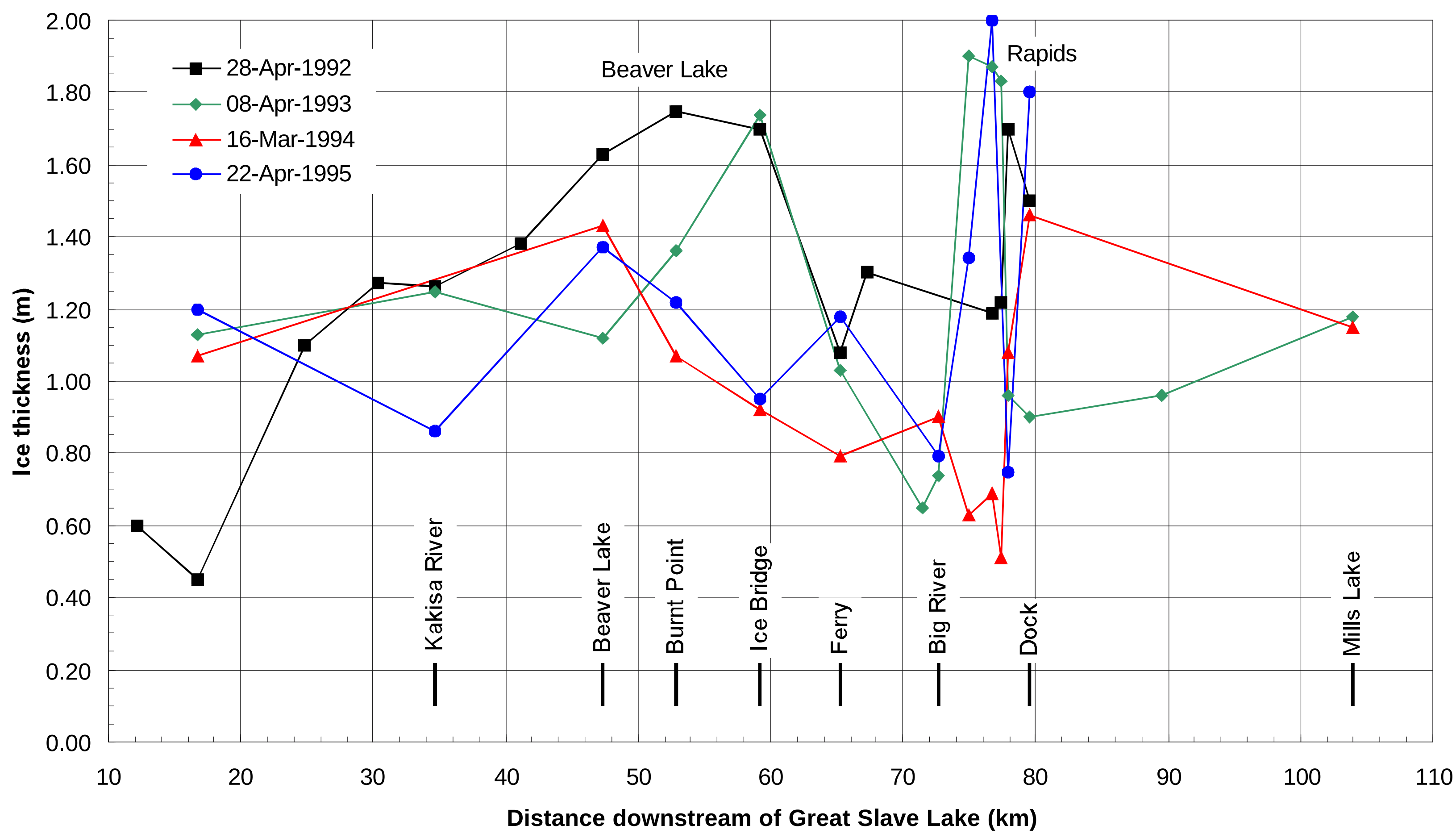
PROJECT
MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE

FREQUENCY CURVES OF REGIONAL ICE THICKNESS

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	3.4



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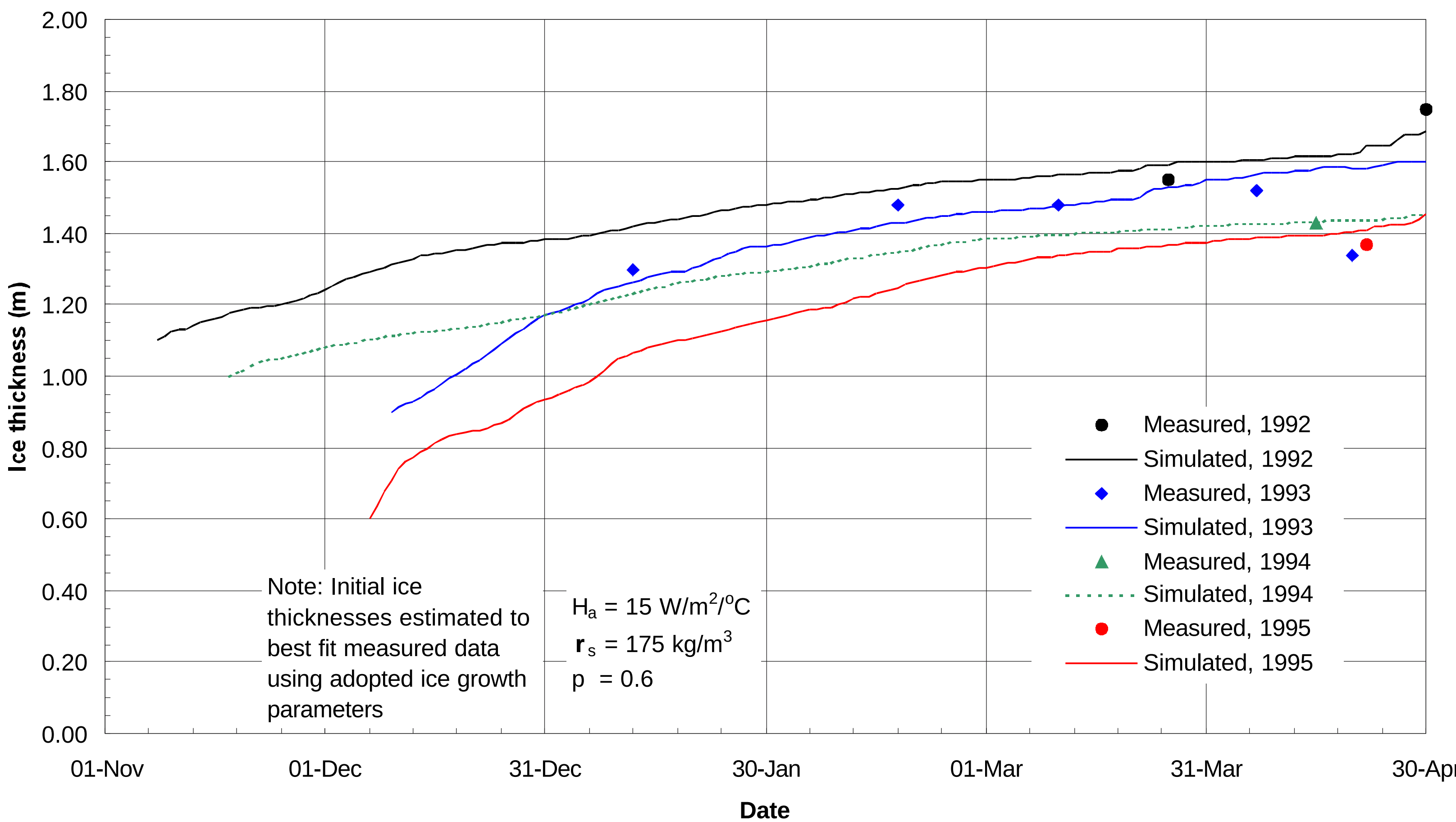
PROJECT	MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE

MEASURED ICE THICKNESSES IN VICINITY OF PROPOSED BRIDGE

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	3.5



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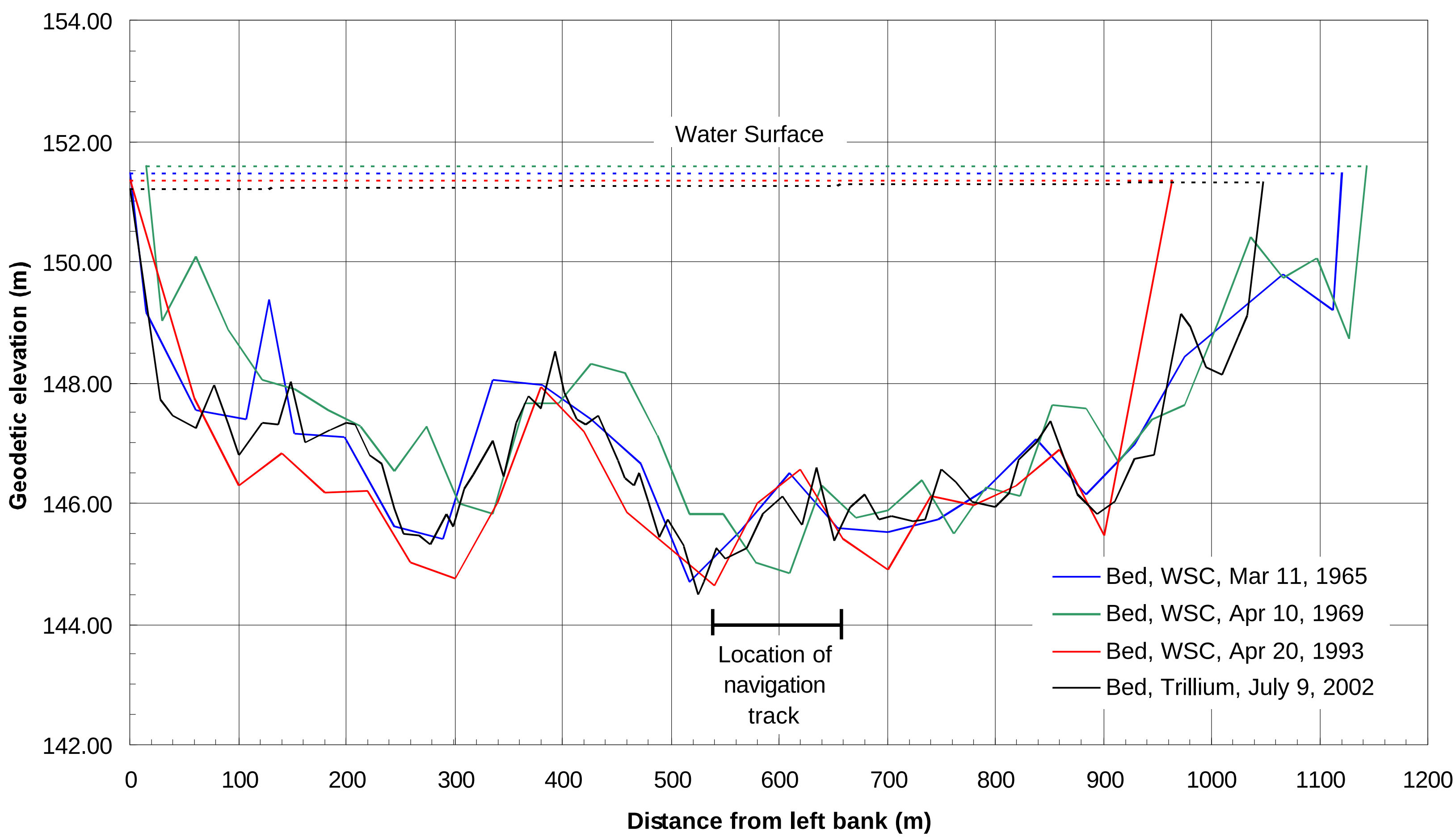
PROJECT
MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE

COMPARISON OF MEASURED AND SIMULATED ICE THICKNESSES UPSTREAM OF PROPOSED BRIDGE

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	3.6



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PROJECT	MACKENZIE RIVER BRIDGE AT FORT PROVIDENCE

COMPARISON OF CROSS SECTIONS MEASURED AT PROPOSED BRIDGE SITE FROM 1965-2002

PROJECT NUMBER	DATE	FIGURE
02-519	OCT. 29, 2002	4.1