ATHABASCA RIVER BRIDGE TO STEEPBANK MINE
RIVER HYDRAULICS AND
ICE STUDY

AGRA
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ATHABASCA RIVER BRIDGE TO STEEPBANK MINE RIVER HYDRAULICS AND ICE STUDY

Submitted To:

H. A. Simons Ltd. for Suncor Inc.

Submitted By:

AGRA Earth & Environmental Limited

Calgary, Alberta

In Association With

Trillium Engineering and Hydrographics Inc.

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EXECUTIVE SUMMARY

I. INTRODUCTION

Hydrologic, hydraulic and ice investigations of the Athabasca River were conducted to support preparation of a tender information package for design and construction of a proposed bridge at a site at the north edge of Suncor's Tar Island Dike (TID). The bridge is required to expand Suncor's oil sands extraction operation to the proposed Steepbank Mine on the east side of the Athabasca River. The investigations included the following tasks:

- predict the potential for increased ice jamming and associated affects on upstream river bank conditions, particularly along TID;
- estimate potential bridge pier, abutment and cofferdam scour and identify appropriate protection measures for each;
- estimate hydraulic loads, ice loads, flood levels and flow during peak open water flood conditions and ice jam conditions; and
- determine the potential impact of the bridge crossing alternatives on river flow patterns particularly with respect to navigation and siltation characteristics in the vicinity of Suncor's Fresh Water Inlet located approximately 400 metres downstream of the proposed bridge site.

AGRA Earth & Environmental Limited (AEE) conducted the work in association with Trillium Engineering and Hydrographics Inc. who were responsible for the ice assessments and scour analysis during winter and breakup.

The investigations considered three and four pier bridge crossing options with total widths between bridge abutments of 340 metres and 440 metres, respectively. Bridge piers are referred to by numbers 1 to 4 from left to right across the river.

The investigations also considered in-stream construction issues mainly for the four pier crossing for which H. A. Simons has identified two alternative cofferdam configurations. Cofferdam construction for the piers is expected to begin at the west side during the open water season and finish on the east side during ice cover. The "Base Case" cofferdam configuration assumes an earthfill cofferdam for the left pier and two circular sheet pile cofferdams for the centre two piers, all constructed during the open water season. This is followed by an earthfill cofferdam for the right side pier constructed in the winter. The alternative cofferdam configuration assumes the two left side piers are constructed from an earthfill cofferdam during the open water season followed by a similar cofferdam on the right for the other two piers constructed in the winter.

The investigations involved an office analysis of data from government sources, AEE files and H. A. Simons Ltd. Bridge design assumptions and bridge crossing layouts were provided by H. A. Simons Ltd.

II. DESIGN CRITERIA

The proposed bridge will provide the main access to the east bank of the Athabasca River to support initial construction and operation of a 105,000 BPCD oil sands hydro-transport facility. Therefore, the expected service life will be at least twenty-five years and possibly much longer. Accordingly, the investigations are based upon the following design parameters:

- Design Flood Water Level: 1:100 year open water flood level or 1:100 year ice breakup level, whichever is greater.
- Low Chord Elevation of Bridge at Highest Span: 15.2 metres above the 1:10 year flood level which is defined as the navigation flood level.
- Design River Bed Scour at Piers and Abutments: 1:100 year scour due to open water flooding or to ice jamming, whichever is greater.
- Maximum River Bed Scour at Piers and Abutments (with no additional safety factor allowances): 1:500 year flood event.
- River Bank Protection: 1:100 year flood and 1:100 year ice conditions, whichever is greater.

Construction planning and design measures will depend to a large extent upon the risk that Suncor and the design/build contractor are willing to accept. Expected discharges, water levels and ice conditions are therefore presented for a range of risks or probabilities of occurrence. In this manner, appropriate cofferdam levels and protection measures can be selected for a variety of planned construction periods.

III. HYDROLOGY

Hydrologic design discharges and seasonal discharges are estimated based upon gauged records at the Water Survey of Canada (WSC) gauge named Athabasca River Below McMurray (07DA001) located approximately 25 kilometres upstream of the bridge site. Inflows between the gauge and the bridge site are considered negligible because the increase in drainage area at the bridge site is less than one percent. Estimated maximum instantaneous discharges are summarized in Table ES-1.

TABLE ES-1
Flood Frequencies
Athabasca River Below McMurray (07DA001)

Return Period (Years)	Maximum Instantaneous Discharge (m^3/s)	Comments
2	2 450	Mean Annual Flood
10	3 900	Navigation Flood
20	4 440	-
100	5 950	Design Flood
500	7 700	Extreme Flood Check

IV. HYDRAULICS

The river is a stable, entrenched sand bed channel which does not exhibit obvious signs of aggradation or degradation. It narrows to 300 metres in width just upstream of the bridge site and is approximately 450 metres wide at the bridge. The potential for river channel migration or degradation is not considered a concern for bridge design.

Hydraulic conditions are analyzed using fourteen surveyed river cross-sections and the HEC-2 computer model. Model calibration was based upon a rating curve constructed from 1977 to 1995 water levels recorded at Suncor's Fresh Water Inlet.

Resulting mean channel flow velocities for existing conditions range from approximately 1.3 m/s during the mean annual flood to 2.0 m/s during the 1:100 year flood at the bridge section. Corresponding mean channel flow depths are 4.3 metres and 6.5 metres, respectively, for these flood events.

Predicted increases in water levels and flow velocities for the two open water cofferdam configurations are summarized in Table ES-2.

TABLE ES-2
Impact of Base Case and Alternative Cofferdams Configurations on Channel Velocity and Water Levels During the Open Water Construction Period

Return Period Flood Event (year)	Existing Mean Channel Velocity (m/s)	Base Case Cofferdams(1)			Alternative Cofferdams(2)		
		Mean Channel Velocity (m/s)	Increase in Velocity over Existing (m/s)	Increase in Water Level over Existing (m)	Mean Channel Velocity (m/s)	Increase in Velocity over Existing (m/s)	Increase in Water Level over Existing (m)
2	1.35	2.36	1.01	0.18	2.75	1.40	0.26
10	1.69	2.87	1.18	0.25	3.32	1.63	0.37
20	1.79	3.03	1.16	0.29	3.50	1.71	0.42
50	1.95	3.05	1.10	0.29	3.75	1.80	0.48
100	2.04	3.07	1.03	0.25	3.93	1.89	0.52

Note: 1. Construction period for Base Case Cofferdam Configuration for Piers 1, 2 and 3 in place is March - October. Refer to Figure 3.2 for details.
2. Construction period for Alternative Cofferdam Configuration is March to September. Refer to Figure 3.3 for details.

A much greater constriction in the river is required for the Alternative cofferdam than the Base Case cofferdam configuration. The channel topwidth at the 1:20 year flood level is constricted from 412 metres for existing conditions to 272 metres for the Base Case, to 245 metres for the Alternative cofferdam configuration.

The impact of the two bridge options on water levels and flow velocities is minimal as summarized in Table ES-3.

TABLE ES-3
Impact of Three and Four Pier Schemes on Channel Velocities and River Levels

Return Period Flood Event (year)	Existing Mean Channel Velocity (m/s)	Four Pier Scheme			Three Pier Scheme		
		Mean Channel Velocity (m/s)	Increase in Velocity over Existing Load (m/s)	Increase in Water Level over Existing Conditions (m)	Mean Channel Velocity (m/s)	Increase in Velocity over Existing Conditions (m/s)	Increase in Water Level over Existing Conditions (m)
2	1.35	1.40	0.05	0.02	1.78	0.43	0.07
10	1.69	1.72	0.03	0.04	2.20	0.51	0.11
20	1.79	1.83	0.04	0.05	2.34	0.47	0.13
100	2.04	2.08	0.04	0.07	2.67	0.63	0.18

V. ICE CONDITIONS

The ice-related analyses draws upon historical observations of ice jams and ice-related high water levels between Fort McMurray and Fort Mackay. The observed ice jam stage frequency curve is transposed to the proposed bridge site with appropriate adjustments for local conditions. The toe characteristics and the subsequent scour at the ice jam toe are calculated using the non-uniform ice jam model, **RIV JAM**. The ice loads are based on the CSA-S6-88 design code and numerous measurements and ice strength observations on the Athabasca River.

The earliest and latest reported ice breakup dates on the Athabasca River are April 11 and April 28. The velocity of the breaking ice front when ice jams have formed have been as great as 5.5 m/s for short periods of time, but it typically averages about 3.5 m/s. Ice floe velocities typically vary for 4 to 5 m/s during the ice run prior to jamming and are only about 2 m/s when the stable jam is collapsing. Stage increases can vary between 1 and 5 m/h and the drawdown rate is typically 0.5 m/h.

Ice jam observations indicate that the location of the toe can be anywhere between MacEwan Bridge at Fort McMurray and Suncor. Recorded durations of jams have varied substantially from two days up to fourteen days. The breakup stage-frequency curve at the bridge site is estimated by transposing the Fort McMurray ice jam rating curve using the channel hydraulics at the site and the 1979 ice jam profile which occurred just upstream of Suncor. The analysis indicates that grounding of the ice jam toe is a necessary condition for ice jams in this reach of the river. Predicted severe ice jam levels at frequencies greater than the 1:20 year jam are derived by reducing the frequencies of larger events at Fort McMurray by 30 percent to account for the reduction in severe ice jams observed in the reach downstream of Fort McMurray. Table ES-4 summarizes the estimated ice breakup stage frequencies at the bridge site.

TABLE ES-4
Summary of Breakup Stage Frequencies and Representative Discharges at the Proposed Bridge Site

Exceedence Probability (%)	Return Period (years)	Representative Discharge (m^3/s)	Water Elevation at the Proposed Bridge Site (Suncor Datum - m)
1	100	3160	242.0
2	50	2450	241.2
5	20	1530	240.2
10	10	1050	239.0
20	5	680	238.0
50	2	320	236.4

Impacts of the two winter cofferdam configurations on ice jam levels are minimal, as summarized in Table ES-5.

TABLE ES-5
Summary of Ice-jam Related Water Elevations with a Cofferdam in Place

Calculated Water Elevation (Suncor Datum - m)			Return Period (years)
No Constriction	Base Case	Alternative Case	
241.40	241.41	241.44	100
240.11	240.12	240.15	20
239.03	239.04	239.06	10
236.37	236.38	236.39	2

Ice jam scour is computed assuming most of the flow occurs under the grounded toe of the jam with very little seepage. Predicted scour elevations for the two bridge options and two cofferdam configurations are summarized in Table ES-6.

TABLE ES-6
Scour Depths Below the Toe of an Ice Jam at the Bridge Site for Various Bridge and Construction Scenarios

Return Period (years)	Scour Elevations in Bridge Waterway (Suncor Datum - m)					
	Four-Pier Bridge			Three-Pier Bridge		
	Bridge Without Cofferdam	Base Case Cofferdam	Alternative Case Cofferdam	Bridge Without cofferdam	Base Case Cofferdam	Alternative Case Cofferdam
100	225.1	224.2	222.3	224.5	223.8	221.3
50	225.7	224.9	223.1	225.1	224.4	222.1
20	226.7	226.0	224.5	226.2	225.7	223.7
10	227.8	227.3	226.1	227.4	227.0	225.5
5	228.8	228.4	227.5	228.5	228.2	227.1
2	229.9	229.7	229.2	229.7	229.5	228.9

Either bridge option is not expected to increase the likelihood of ice jamming in the reach because the bridge opening is greater than the 300 metre narrow section in the river immediately upstream.

Calculated ice loads are based on a 1:20 year ice strength of 1200 kPa with a 1:20 year ice thickness of 1.3 metres applied at a 1:100 year elevation. Various dynamic ice loads based on these characteristics are summarized in the conclusions.

VI. DESIGN SCOUR

Two criteria are recommended for scour at the piers and bridge abutments. The first criteria is the 100 year scour level caused by an open water flood or ice jam, whichever is greater. The bridge should be stable based on conservative calculations with normal safety factors for this condition. The second criteria is the 500 year scour level based on the worst case of open water flood or ice jam. The bridge should be stable for this condition based on reduced safety factors.

Maximum potential open water scour is computed as the sum of general scour (which develops due to the orientation or constriction of flow during flood conditions) plus bedforms (dunes in the case of the Athabasca river) plus local scour conditions at piers or at the nose of cofferdams.

Riverbed scour is based upon local hydraulic characteristics and the riverbed material. The riverbed material is assumed to consist of uniformly graded fine sand (average bed material size (D_{50}) is 0.3 mm) based upon available drillhole logs.

Recommendations pertaining to scour at cofferdams and bridge piers are discussed below.

<u>**Cofferdams**</u>
Open water scour depths computed for the cofferdam configurations are presented graphically in Section 5 of the main report. Because local velocities at the nose of the cofferdam can exceed 3.6 m/s during the 1:20 year flood event, the minimum armour protection for open water season earthfill cofferdams should consist of a 0.4 metre thick layer of cobbles (i.e., 2 x D_{50}, where D_{50} = 200 mm) at a 3:1 slope. An apron 0.8 metres thick extending out from the toe of the cofferdam is needed to allow for anticipated local scour.

Flow velocities and local scour under ice conditions before breakup are expected to be minor. Therefore, no armour protection is deemed to be necessary if all construction is completed before breakup. If the cofferdam is expected to be left in service during breakup, a larger Class I size riprap (D_{50} = 300 mm) would be required.

<u>**Bridge Piers**</u>
The piers of the current proposed bridge alignment will be skewed about 8 degrees to the direction of flood flow if the piers are aligned perpendicular to the axis of the bridge. The direction of flow through the bridge during floods is estimated by drawing a line tangent from the upstream projecting east bank to the west side of the island just downstream of the bridge. Because of the shifting bed conditions observed on the Athabasca River in this reach, the actual local direction of flow at any pier may easily vary by ±5 degrees from this estimated flow direction. The impact of skew on local pier scour increases the scour depth from 4.6 metres with no skew to 7.9 metres with 13 degrees of skew to the direction of flow.

The general open water scour envelopes for the two bridge options are as follows:

Bridge Option	1:100 Year Return Period Open Water Scour Level[1]	1:500 Year Return Period Open Water Scour Level[1]
Four Pier	226.0 m	224.6 m
Three Pier	223.9 m	222.2 m

[1] Excluding local pier scour effects.

Local pier scour would extend below the above elevations unless the pile cap foundation is designed to limit local pier scour. Because downward vortex currents responsible for local scour typically do not extend out more than two times the pier width or 2 times 2.5 metres in this case, the proposed pile cap foundation supplemented with riprap around the pile cap at the same level as the pile cap is proposed to limit pier scour. The top of pile cap level is therefore based on the deepest 1:100 year return period scour whether due to ice or open water scour.

At the 1:500 year extreme flood condition, general scour is below the proposed top of the pile cap. Under these conditions, pier and foundation scour relations predict scour could extend to bedrock (elevation 217± metres), assuming no increase in natural bed armouring with increasing scour depth. To limit the extent of scour during extreme floods, additional riprap around the pile cap is proposed. The specified amount of additional riprap proposed is sufficient to allow settlement of the riprap around the pile cap as the river scours but prevents scouring under the pile cap foundation. A maximum additional scour of 5 metres below the top of the pile cap is a conservative estimate for these conditions.

Proposed riprap protection around the pile cap is based on a 14 metre wide by 22 metre long pile cap with the pier located near the front of the pile cap (for stability purposes). A minimum horizontal projection of 6 metres of Class I (D_{50} = 300 mm) riprap, 1.2 metres thick is proposed around the upstream end of the pier. This allows for the fact that local scour is greatest upstream of the direction of flow. A 3 metre wide by 1.2 metre thick layer is proposed all around the rest of the pile cap to keep any local scour removed from the edge of the pile cap. Riprap is also proposed at the downstream end, because wake vortex currents and potential bed material transport will be stronger here due to the extent of riprap protection and lack of material movement allowed upstream and on the sides of the piers.

<u>Abutment Scour</u>

Design abutment scour conditions are the same as defined by the 1:100 year open water flood or ice jam general scour envelope, whichever is deepest. A 0.6 metre thick layer of Class I (D_{50} = 300 mm) riprap abutment protection is required to provide protection against ice shove and the potential high velocities which develop during ice jam scouring. Bank protection to the 1:100 year maximum ice level is proposed for both banks.

Because maximum ice jam scour could develop equally anywhere across the channel section, a 2 metre thick rock apron is proposed to protect against scour to the same design scour level on both abutments.

VII. CONCLUSIONS AND RECOMMENDATIONS

1. Recommended bridge design parameters for the three and four pier bridge options are summarized in Table ES-6 below.

TABLE ES-7
Summary of Bridge Design Parameters

Item	Design Parameter	Unit	Four Pier Bridge	Three Pier Bridge
1	Mean Annual Water Level (655 m^3/s)	m	234.9	234.9
2	Mean Annual Flood Level (2450 m^3/s)	m	237.0	237.0
3	Mean Annual Ice Jam Level	m	236.4	236.4
4	1:10 Year Flood/Navigable Flood Level (3900 m^3/s)	m	238.2	238.2
5	Bridge Low Chord Elevation (15.2 m + Item 4) at highest span	m	253.4	253.4
6	1:100 Year Flood Level (5950 m^3/s)	m	239.6	239.7
7	1:100 Year Ice Jam Level/Top of Abutment Riprap Protection	m	242.0	242.0
8	Design Scour Level = Minimum top of foundation and abutment scour protection level (1)	m	225.0 (I)	223.9 (F)
9	Design Maximum Ice Thickness	m	1.3	1.3
10	Ice Strength	kPa	1200	1200

(1) Riprap protection around the pile cap foundation is also required to protect against local scour and extreme flood conditions
I = 1:100 year ice jam scour
F = 1:100 year flood scour

Dynamic ice loads for 1:20 year ice strength with a 1:20 year ice thickness applied at a 1:100 year elevation are summarized in Table ES-7.

TABLE ES-8
Ice Loads on the Bridge Piers for 20 Year Ice Strength, 20 Year Ice Thicknesses and 100 Year Ice Elevation

Load Type	Nose angle (°)		Magnitude	Elevation
	Vertical	Horizontal	(kN)	(m)
Longitudinal[1]	90	-	6500	241.5
	60	-	3800	241.5
Transverse	-	100[2]	1100[3]	241.5
Thermal[4]	-	-	2700	235.0
Ice jam	-	-	120	237.0
Vertical[5]	-	-	900	235.0

[1] Refers to load parallel to direction of flow in the river.
[2] Equivalent to round pier nose.
[3] Assumes pier nose inclined at 60 degrees, refer to Table 4.12 for effect of pier skew.
[4] Assumes a pier length of 10 metres.
[5] Assumes a pier circumference of 25 metres.

2. The bridge alignment and abutment locations for either the three or four pier bridge options do not result in excessive constrictions to flow in the Athabasca River. The maximum backwater affect is 17 cm due to the three pier bridge option during a 1:100 year flood. Similarly, either bridge option is not expected to increase the likelihood of ice jamming or bank erosion in this reach. Rapid drawdown rates of 0.5 m/h are usually associated with an ice run or a jam of very short duration. Rapid drawdown is therefore not a major concern affecting bank stability. Additional bank erosion protection along TID is therefore not required as a result of the proposed bridge.

3. The Alternate cofferdam configuration results in constrictions of over 40 percent of the channel width. The left side cofferdam would cause a large flow shift. The constriction caused by the right side cofferdam may be enough to initiate an ice jam. Therefore, this cofferdam should be removed before the breakup period.

4. The cofferdams are subject to extensive river bed scour because of the highly mobile sand bed river conditions in this reach of the Athabasca River. Extensive riprap aprons are required to accommodate this scour. Alternatively, the cofferdams could be protected by deep sheet piling.

5. An ice bridge is expected to be used during construction. The ice bridge should be broken up into ten or more units to reduce the risk of creating an ice jam during breakup.

6. The bridge alignment is estimated to be skewed 8 degrees from perpendicular to the direction of flow during floods. This is an upper limit of the tolerable skew for the design conditions discussed in this report because actual local skew on some piers may

be even greater during a flood. If possible, the bridge pier alignment should be turned counterclockwise by up to 8 degrees to minimize the risk of scour and to reduce ice loading.

7. The present location of the left abutment projects into the river from the existing bank by about 75 metres. Moving the abutment further into the river is not recommended because the main flow of the river is presently aligned along the left bank side of the channel. Moving the four pier bridge configuration 50 to 75 metres westward is recommended because this would reduce the risk of excessive scour, reduce the risk of ice damage and reduce the risk of sedimentation at the Fresh Water Intake.

   If the three pier bridge option is selected, the left abutment and Piers 1, 2 and 3 should be in the same locations as piers 1, 2 and 3 of the four pier bridge option. The right abutment of the three pier option should be located at about the location of Pier 4 of the four pier bridge option.

8. The hydraulic investigations of the three pier bridge option are based on vertical abutments. These abutments would have to be designed for dynamic ice loads that are similar to those experienced by the piers, except applied to a larger width of about 10 metres. To minimize the exposure of the abutment to high ice loads, a sloped abutment could be constructed. A slope of 45 degrees or steeper would have a minor impact on the hydraulic results presented in this report.

9. The recommended minimum opening from a hydraulic and ice jam perspective is as illustrated on Figure ES-1 below. Coordinates for the corresponding abutments are indicated as provided by H. A. Simons.

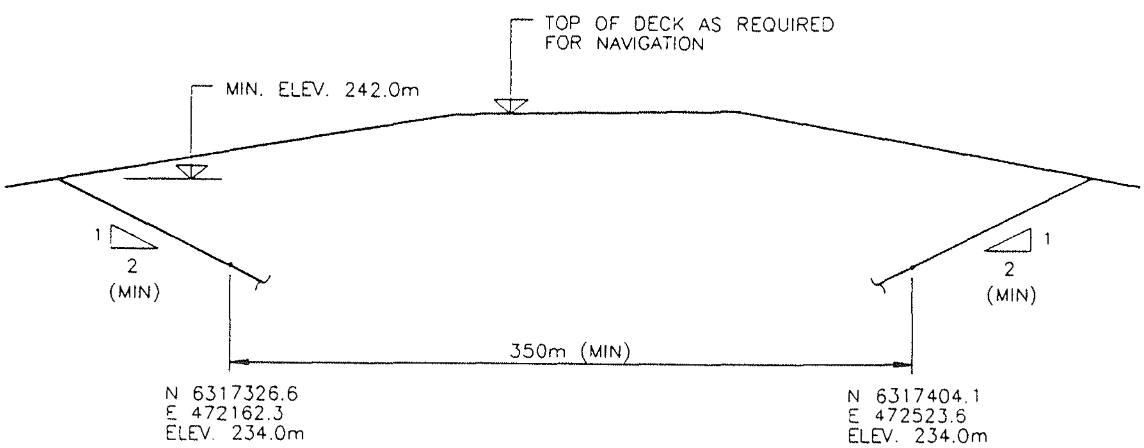


FIGURE ES–1 MINIMUM HYDRAULIC OPENING

10. Historical ice jams observed in the reach upstream of the bridge site can be expected to occur in the vicinity of the bridge. As a result, ice jam levels are higher than open water flood levels for similar return periods.

11. Protection against local pier scour at the piers is provided by the foundation pile cap which extends nearly 6 metres on both sides of the piers. This protection for local scour should be supplemented by riprap extending out from the pile cap as shown on Figure 6.2 and keyed into the bed so that the top of the riprap is level with the top of the pile cap. The riprap protection should be sufficient to provide protection in the event of an extreme flood, up to and including the 1:500 year flood.

12. Bed levels across the channel between piers 1 and 3 are expected to remain relatively uniform. Either of these sections could serve as the navigation channel section. River training works should not be used to control the location of the thalweg in the centre bridge section.

13. The proposed bridge will cause some deposition along the left bank. This may extend downstream as far as Suncor's Fresh Water Inlet. The potential impact of such sedimentation is expected to be minor. If deposition does become a major problem, it may be possible to modify the existing inlet to remedy the situation.

14. All recommendations in this report are based on the bridge alignment and configuration provided by H. A. Simons. If any changes occur to the bridge alignment or configuration, including piers and abutments, AEE and Trillium Engineering and Hydrographics Inc. should be advised of these changes and given an opportunity to revise these recommendations.

1.0 INTRODUCTION

1.1 BACKGROUND

Suncor Inc. is planning to expand its existing oil sands extraction operation at Lease 86/17 located 35 kilometres north of Fort McMurray. The expansion will include opening of the Steepbank Mine on Lease 97 located on the east side of the Athabasca River across the river from the existing Suncor operation. A study conducted by H. A. Simons Ltd. identified that a bridge across the Athabasca River at the north edge of Suncor's Tar Island Dike (TID) will provide the best means of accessing the oil sand reserves on Lease 97 and adjacent leases. The proposed bridge site is shown on Figure 1.1.

H. A. Simons Ltd., on behalf of Suncor Inc., commissioned AGRA Earth & Environmental Limited (AEE) to conduct river and ice engineering investigations to provide criteria for planning, detailed design and construction planning of the proposed bridge. AGRA Earth & Environmental Limited (AEE) conducted the work in association with Trillium Engineering and Hydrographics Inc. who are specialized in river ice engineering. AEE managed the project and conducted the hydrologic studies, open water hydraulic studies and open water scour analyses. Trillium Engineering and Hydrographics Inc. was responsible for ice assessments and scour analysis during winter and breakup.

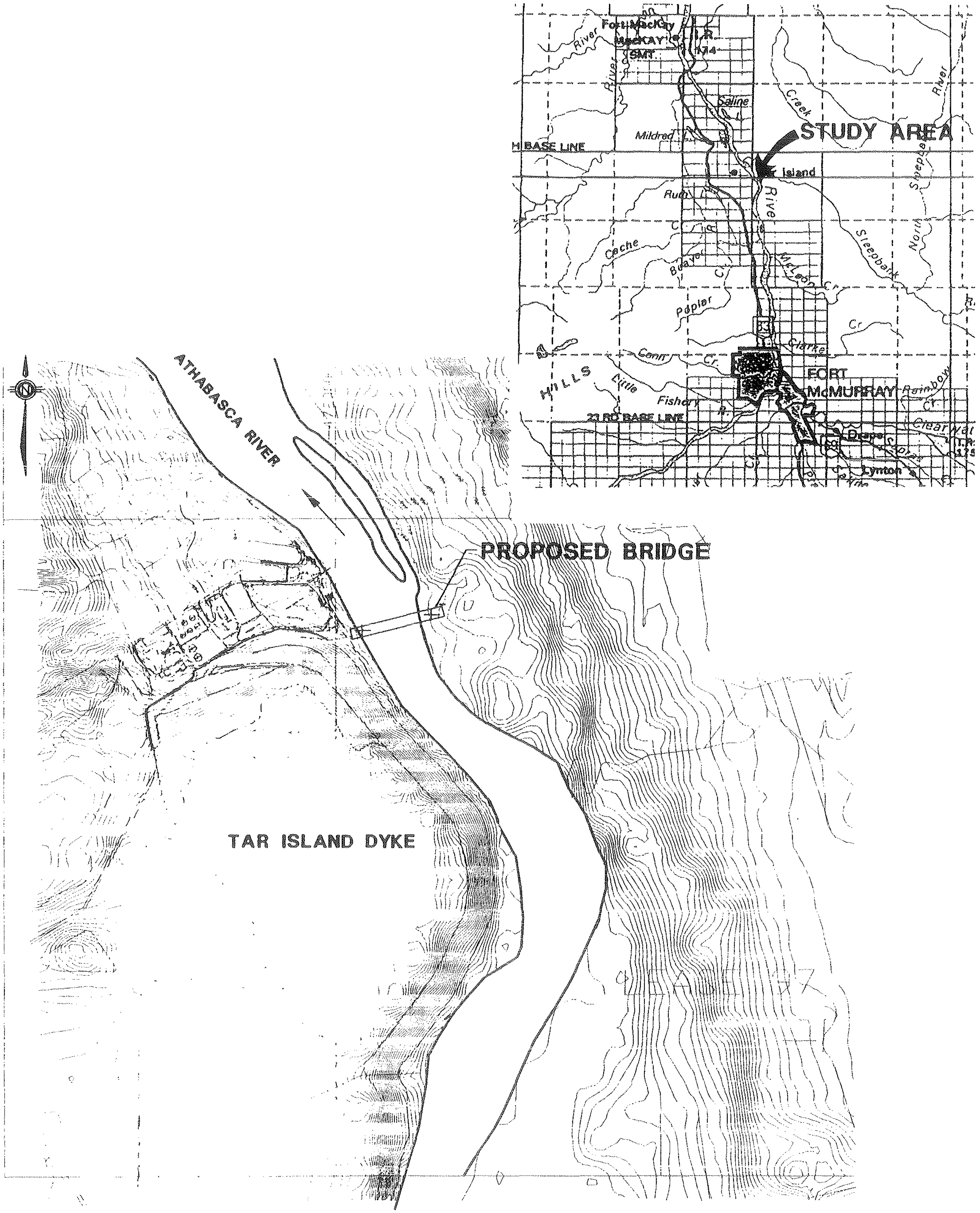
1.2 OBJECTIVES/SCOPE OF WORK

Suncor Inc. requires this river and ice engineering study to supply hydraulic loads, ice loads, flood levels, flow velocities and scour depths in a tender information package for design and construction of the proposed bridge across the Athabasca River. The specific objectives established for these investigations are given in a letter from H. A. Simons Ltd. dated November 2, 1995 as follows:

1. *Review the provided river data and ascertain what additional background data will be required to complete the study.*

2. *Determine the impact of the bridge piers on the river. The study shall evaluate upstream and downstream river profiles prior to bridge construction, during bridge construction, and after bridge construction. The effect on erosion of the toe of Tar Island Dike shall be evaluated and erosion protective design measures shall be evaluated and recommended (if any measures required). Detail preparation of erosion protection drawings and design will not be required at this time. The influence of the bridge on the river surface profile shall be evaluated for mean flow conditions, annual average peak flow conditions, 1:10 year flood conditions, navigational flood conditions, and 1:100 year flood conditions.*

3. *Determine the depth of scour at each of the proposed river pier locations and provide recommendations for erosion protection measures for the fill at each abutment. The consultant will review the scour depth predictions under peak flood conditions and ice jam conditions.*



AGRA
Earth & Environmental Limited
ENGINEERING & ENVIRONMENTAL SERVICES

CLIENT:	H.A. SIMONS LTD.	DATE:	JAN 1996
PROJECT:	SUNCOR–ATHABASCA RIVER BRIDGE	JOB No.	CW1466.00
	LOCATION PLAN	CAD FILE:	CW\1466\1466–502
		FIGURE 1.1	REV. —

4. *Determine the extent of ice jamming at the bridge and predict depth of ice anticipated at the bridge pier location. Evaluate the climatic conditions at the Fort McMurray area and provide recommendations for fracture strength and pier ice forces. Evaluate the rate of ice jamming and jamming breakup for determination of rate of rise/fall in river levels upstream of the bridge with particular emphasis of rapid drawdown and resulting destabilization of Tar Island Dyke. Evaluate the ice scour effects along the toe of Tar Island Dike and work with recommendations of Item 3 above for erosion protective measures.*

5. *Review river geometry and changes in river flow pattern for determination of impact to the Suncor Fresh Water Inlet. Evaluation will include determination of changes in siltation characteristics of river inlet area.*

The investigations consider three and four pier bridge crossing options with total widths of 340 metres and 440 metres, respectively between bridge abutments. Two cofferdam configurations are evaluated.

The evaluation of the ice regime of the Athabasca River at this location is particularly important because most of the large historical flood events in this area have been as a result of the formation of ice jams. Thus, the quantification of the ice jam-related flood frequencies and scour is necessary to determine the appropriate level of protection. Given a tight construction schedule, and the fact that the east side access berm for the piers may be required to withstand a spring breakup event, the risks of cofferdam overtopping and scouring need to be identified.

The investigations involved an office analysis of data available from government sources, AEE files and H. A. Simons Ltd. Bridge design assumptions and bridge crossing layouts were provided by H. A. Simons Ltd.

1.3 SUPPORTING INFORMATION

The investigations are based upon the following data.

(a) historical aerial photographs of the local reach of the river (1950, 1971, 1980, 1981, 1994);

(b) river cross-section surveys by Alberta Research Council (ARC) in 1977 and 1982; by AEE in 1992 and July 1995; and by Suncor Inc. in September 1995;

(c) previous HEC-2 (computer model for calculating backwater hydraulics) set-up and run results for various return period events;

(d) historical river levels at the Suncor intake from 1977 - 1995;

(e) design and construction files for Highway No. 63 bridge near Fort Mackay obtained from Alberta Transportation, Bridge Branch;

(f) Athabasca River discharge data from Water Survey of Canada;

(g) river bed material samples and drill hole logs along TID obtained from AEE files;

(h) drill hole logs by Klohn Crippen (1995) at the pier locations of the proposed bridge crossing obtained from H. A. Simons Ltd.;

(i) historical ice jam and ice scar data (ARC, Alberta Environmental Protection [AEP] and others); and

(j) numerous hydrologic, hydraulic, navigation, ice, geologic and geomorphic reports for the local reach of the Athabasca River.

1.4 DESIGN CRITERIA AND APPROACH

The proposed bridge will provide the main access to the east bank of the Athabasca River to support initial construction and operation of a 105,000 BPCD oil sands hydro-transport facility. The bridge will accommodate mine haul trucks plus various pipelines (hydro-transport, hot water, recycle and tailings). The expected service life of the mine is twenty-five years but the service life of the bridge may be much longer. Accordingly, the investigations are based upon the following design parameters:

(a) Design Flood Water Level: 1:100 year flood level or ice breakup level, whichever is greater.

(b) Low Chord Elevation of Bridge at Highest Span: 15.2 metres above the 1:10 year flood level which is defined as the navigation flood level by the Canadian Coast Guard.

(c) Design River Bed Scour at Piers and Abutments: 1:100 year scour due to open water floods or ice jamming, whichever is greater.

(d) Maximum River Bed Scour at Piers and Abutments (with no additional safety factor allowances): 1:500 year flood event.

(e) River Bank Protection: 1:100 year flood and 100 year ice conditions, whichever is greater.

Construction planning and design measures will depend to a large extent upon the risk that Suncor and the contractor is willing to accept. Expected discharges, water levels and ice conditions are therefore presented for a range of risks or probabilities of occurrence. In this manner, appropriate cofferdam levels and protection measures can be selected for a variety of planned construction periods.

The ice-related analyses that is undertaken herein will draw heavily on the historical observations of ice jams and ice-related high water levels between Fort McMurray and Fort Mackay. The observed ice jam stage frequency curve (and its associated discharges) will be transposed to the proposed bridge site using the conventional ice jam stability theory and the non-uniform ice jam model, **RIVJAM**. From this analyses, risks of cofferdam overtopping will be determined. The toe characteristics and the subsequent scour at the toe will be calculated from the results provided by **RIVJAM** and modified to reflect a range of possible channel flow constrictions at the toe. The ice loads will be based on the CSA-S6-88 design code, interpreted within the framework of measured ice loads at Hondo (Athabasca River near Smith) and numerous measurements and ice strength observations in the vicinity of the proposed bridge.

1.5 PROPOSED BRIDGE CROSSING LAYOUT AND CONSTRUCTION PLANS

Details of the four pier and three pier bridge crossing options are as described below.

Four Pier Bridge
Piers are numbered 1 to 4 from left to right (west to east) across the river. Abutments are at 2H:1V slopes facing the river. Spans between piers are 70 metres from the abutment to the first pier and 100 metres for the three centre spans from Pier 2 to 4 to provide a total width of 440 metres between abutments.

Three Pier Bridge
Vertical abutments are assumed with spans of 70 metres between the abutments and the first piers and 100 metres between the two centre sections to provide a total width of 340 metres between abutments.

The same pier and foundation dimensions are assumed for each bridge option. Piers are 2.5 metres wide at the foundation pile cap tapering to 2.0 metres wide above the river bed level. A wedge shaped steel pier nose is assumed with a nosing angle of 15 degrees from the vertical. Pier length varies from approximately 18 metres (±) at the foundation to 9 metres (±) at the narrowest to 15 metres at the top. The pile cap foundation is assumed to be 14 metres wide by 22 metres long by 1.5 metres thick with the pier located near the front of the pile cap for stability purposes.

Construction Plans
In-stream construction considerations are based on the four pier crossing assuming two alternative cofferdam configurations. Cofferdam construction for the piers is expected to begin from the west side during the open water season and finish on the east side during ice cover. The "Base Case" cofferdam configuration assumes an earthfill cofferdam for the left (west) pier and two 34.5 metre diameter circular sheet pile cofferdams for the centre two piers during the open water season. This is followed by an earthfill cofferdam for the right (east) side pier in the winter. The "Alternative Case" cofferdam configuration assumes the two left side piers are constructed from an earthfill cofferdam in the open water season followed by a similar cofferdam on the right for the other two piers in the winter.

The earthfill cofferdams are expected to be dozed into the river using clean overburden typically ranging from 150 mm minus down to number 200 sieve. A 3H:1V completed slope is assumed for these earthfill cofferdams.

2.0 HYDROLOGY

2.1 FLOOD FREQUENCY

The location of the proposed Athabasca River bridge crossing is approximately 25 kilometres downstream of the Water Survey of Canada (WSC) gauge named Athabasca River Below McMurray (07DA001). This gauge has flow records since 1958. The drainage area above this WSC gauging station is 133 000 km^2. Inflows between the WSC gauge and the bridge site are from less than one percent of the total drainage area and have therefore been omitted in the analysis. Longer term streamflow records dating back to 1913 are available further upstream at the Athabasca River at Athabasca (07BE001) gauge where the drainage area of the Athabasca River is 74 600 km^2.

Based upon the above data, flood frequency estimates for the bridge site were prepared by:

(1) analyzing the recorded discharges at the McMurray gauge alone; and

(2) extending the period of record at the McMurray gauge by correlating its concurrent period of record with the Athabasca (07BE001) gauge, deriving a relationship between the two gauges and using the relationship to extend records at the McMurray gauge back to 1913.

Flood frequency estimates were prepared by analyzing recorded maximum annual daily discharges of the Athabasca River Below McMurray. Daily discharges were used instead of peak instantaneous discharges because more daily discharge data are available than instantaneous. A number of flood frequency distributions were evaluated to determine the frequency distribution which provides the best fit to the data. The Log Pearson III distribution provided the best fit to the data and resulted in a 1:100 year maximum daily discharge estimate of 5740 m^3/s.

Evaluation of the concurrent period of record of the McMurray (07DA001) and the Athabasca (07BE001) gauges indicate that the annual flood peak at McMurray typically occurs one to two days after the flood peak at Athabasca. A regression analysis of the two gauges was undertaken for the concurrent period of record (1958-93) based on a one and two day lag. The two day lag provided a better fit of the data and the relationship derived between the two gauges is provided below.

$$Q_2 = 0.916Q_1 + 761 \qquad (R^2 = 0.754) \qquad (1)$$

Where:

Q_1 = Annual Maximum Daily Discharge At Athabasca Gauge (m^3/s)

Q_2 = Annual Maximum Daily Discharge At McMurray Gauge (m^3/s)

The annual maximum daily discharges at the McMurray gauge were estimated for the period 1913 to 1957 based on the above relationship. Flood frequency estimates were then prepared for the extended period of record at the McMurray gauge. The Log Pearson III distribution provided the best fit to the extended data.

The best fit distributions, for both the recorded and extended analyses, provided similar 1:100 year maximum daily discharge estimates, in the range of 5 740 m^3/s to 5 770 m^3/s. The Log Pearson III distribution for extended discharges is marginally preferable over the other distributions since it provides a good fit, it is based on a long-term period of record and gives a conservative estimate. Therefore, the Log Pearson III distribution based on the extended period of record as shown in Figure 2.1 was selected to provide flood frequency estimates for the bridge site.

The ratios of maximum instantaneous discharges to maximum daily discharges for the McMurray station were analyzed. The following relationship was developed to obtain the maximum instantaneous discharge:

$$Q_{Inst} = 1.041\,Q_{Daily} - 47 \qquad (R^2 = 0.997) \qquad (2)$$

Where:

Q_{Inst} = Maximum Instantaneous Discharge At McMurray Gauge (m^3/s)

Q_{Daily} = Maximum Daily Discharge At McMurray Gauge (m^3/s)

The maximum daily and instantaneous discharges, at the McMurray gauge and the proposed bridge crossing, for various return periods, are listed in Table 2.1.

TABLE 2.1
Flood Frequencies
Athabasca River Below McMurray (07DA001)

Return Period (Years)	Maximum Daily Discharge (m^3/s)	Maximum Instantaneous Discharge (m^3/s)	Comments
2	2 400	2 450	Mean Annual Flood
10	3 790	3 900	Navigation Flood
20	4 310	4 440	-
100	5 760	5 950	Design Flood
500	7 430	7 700	Extreme Flood Check

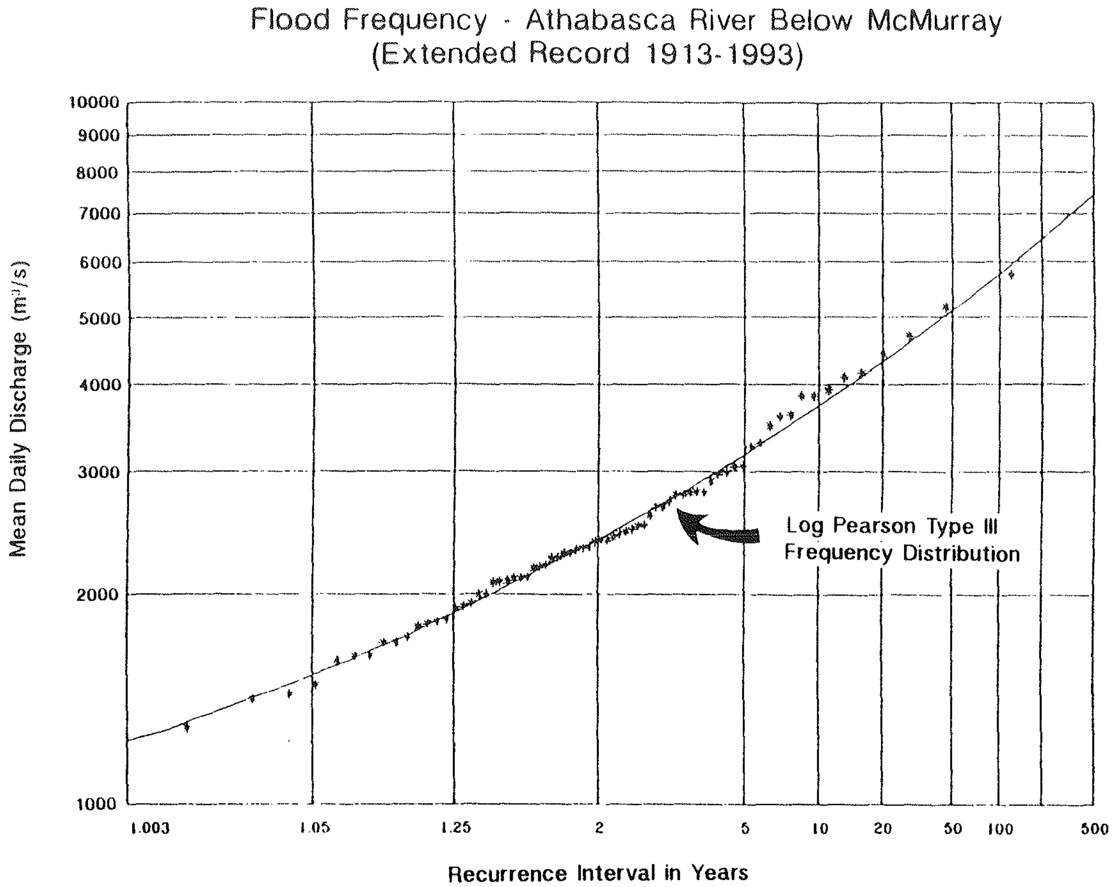


Figure 2.1.

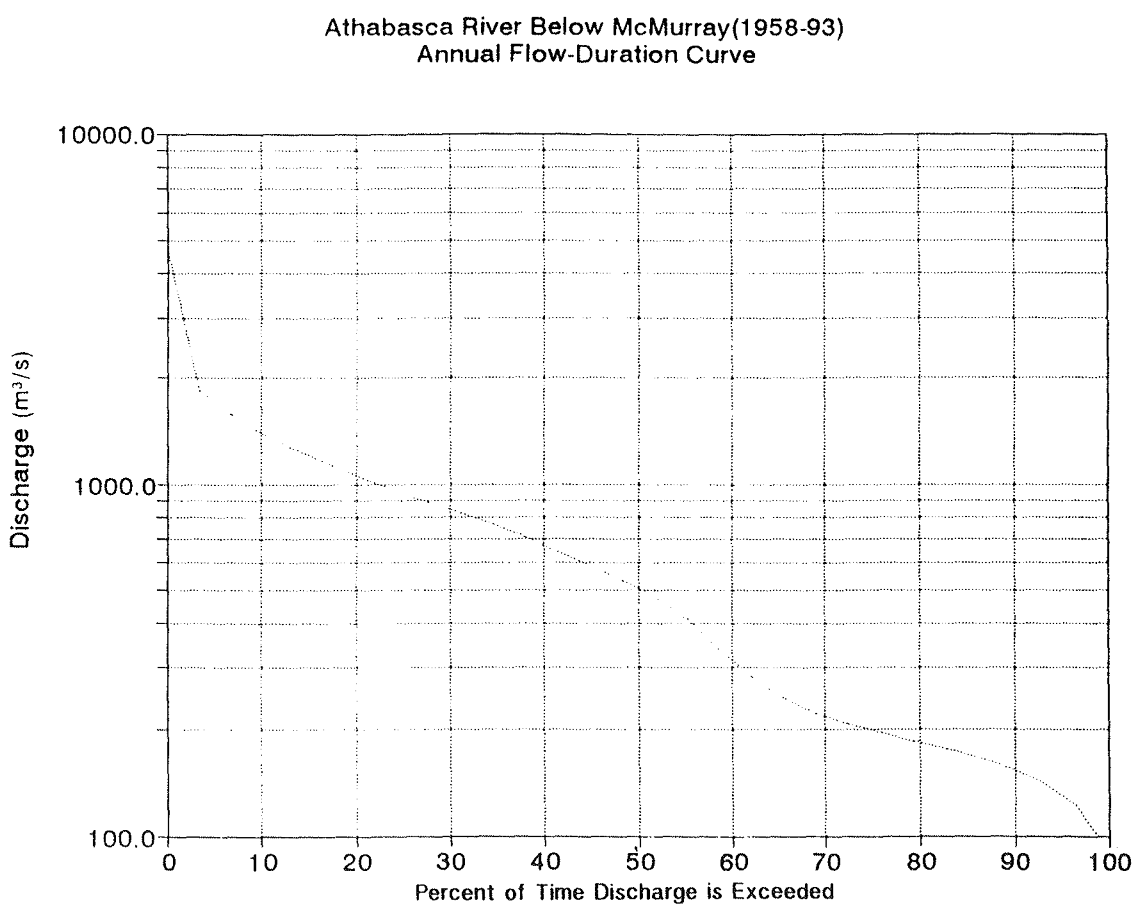


Figure 2.2.

2.2 FLOW DURATION CURVES

Annual and monthly flow duration curves are provided on Figures 2.2 to 2.6 based upon the records at the McMurray gauge.

The mean annual discharge is approximately 655 m^3/s and the median discharge is 500 m^3/s. The median discharge is exceeded fifty percent of the time as indicated by the annual flow duration curve in Figure 2.2. Figures 2.3 to 2.6 provide an indication of the probability of a given discharge being equalled or exceeded in any given month.

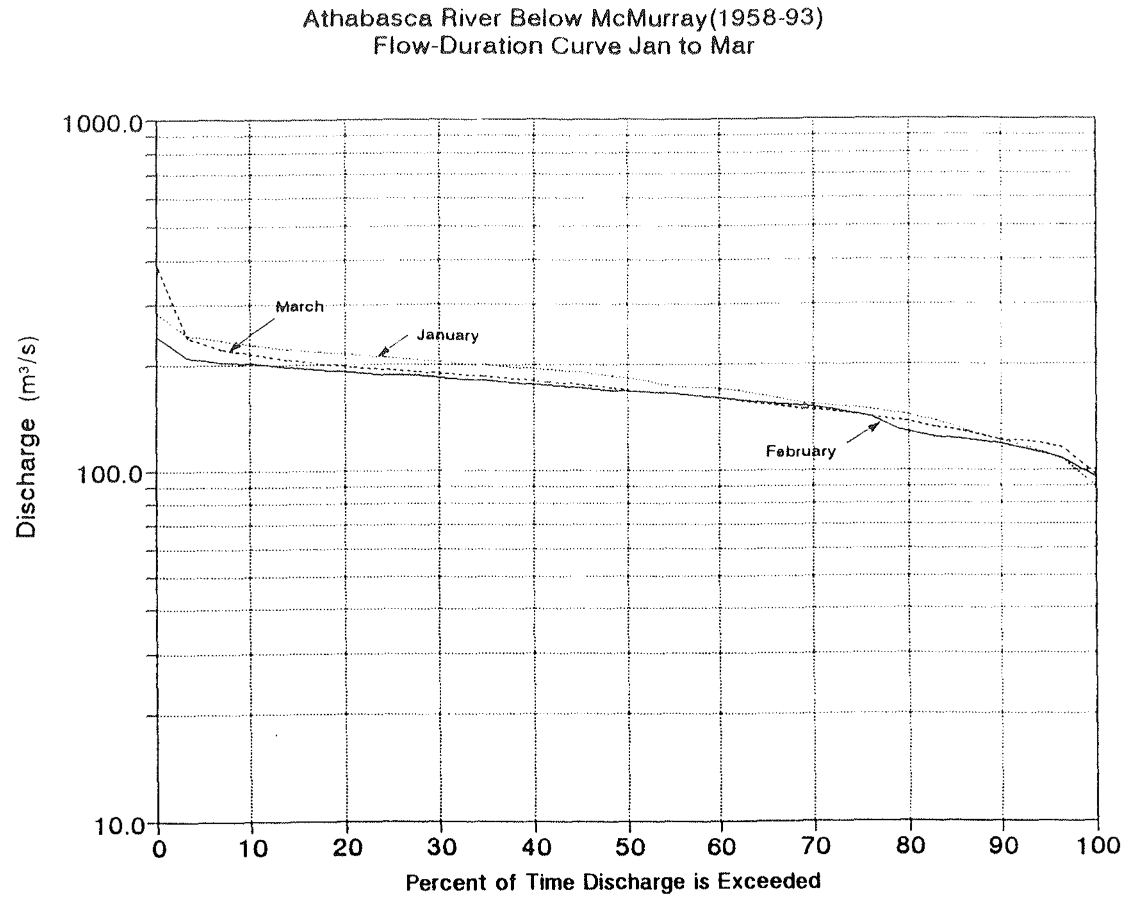


Figure 2.3.

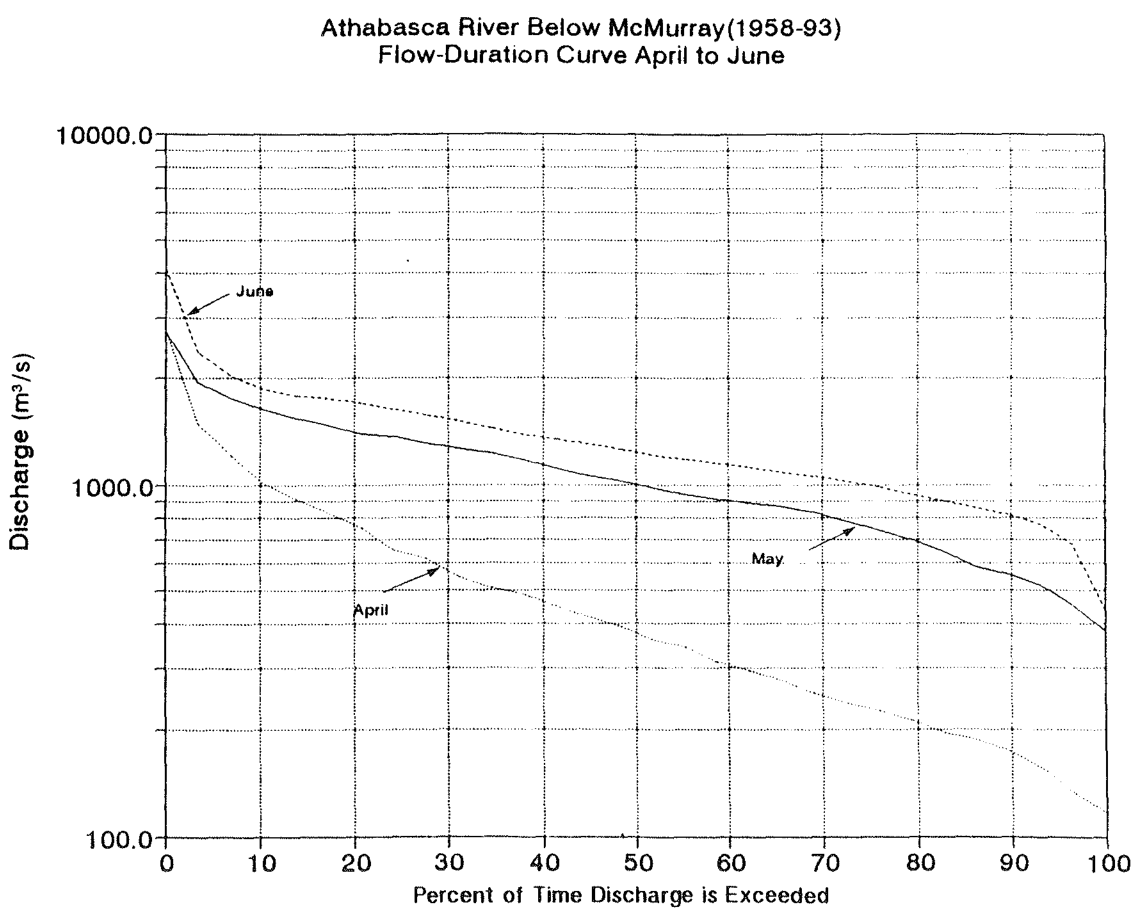


Figure 2.4.

Athabasca River Below McMurray(1958-93)
Flow-Duration Curve July to Sept

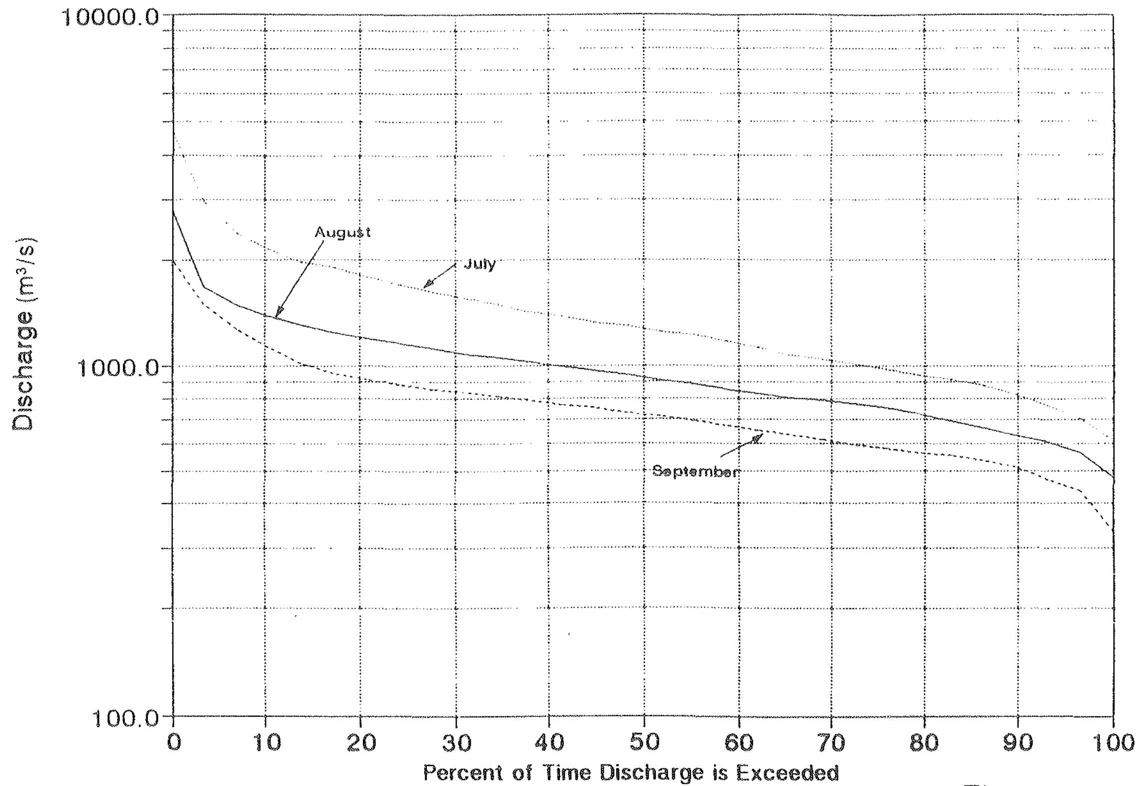


Figure 2.5.

Athabasca River Below McMurray(1958-93)
Flow-Duration Curve Oct to Dec

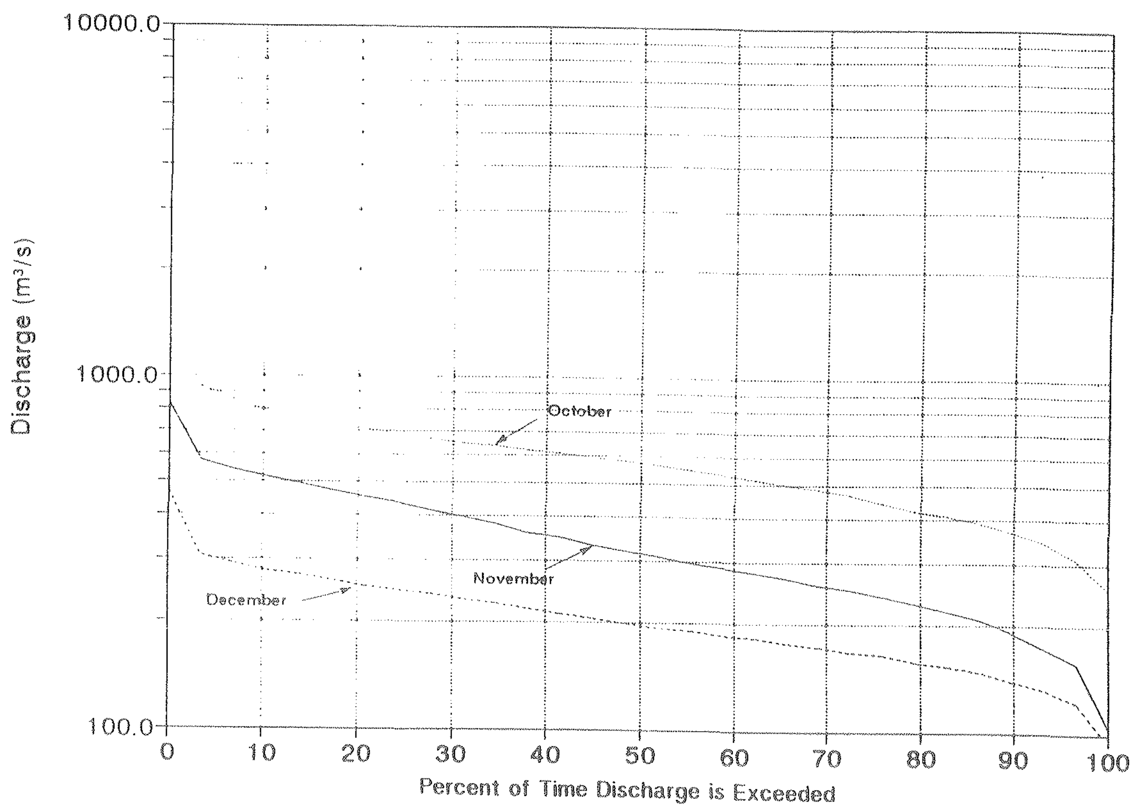


Figure 2.6.

3.0 RIVER REGIME AND OPEN WATER HYDRAULICS

3.1 ATHABASCA RIVER REGIME

The Athabasca River is classed as a weakly meandering stream with wandering elements constrained in a fairly narrow river valley. The river is characterized by meandering channels, which form point bars at ends and longitudinal bars at the inflection points of the meandering thalwegs. The river bed material consists of fine sand (D_{50} = 0.3 mm). The channel configuration is dynamic due to the slow downstream migration of the sand bars or large sand dune formations.

The channel is laterally confined in a relatively straight valley. Valley walls extend approximately 85 metres through the Cretaceous McMurray and Clearwater formations into the underlying Devonian limestone. The Devonian limestone bedrock is frequently exposed along both river banks. Limestone outcrops in the vicinity of the bridge site are located on the right bank extending from 350 to 750 metres upstream of the bridge opposite TID and on the left bank immediately downstream of the bridge site at Suncor's Fresh Water Inlet.

The valley bottom, including the river channel, occasional islands, floodplain and higher level remnant channels in the vicinity of the bridge site is approximately 1750 metres wide. The width of the river channel ranges from 300 to over 800 metres. The river narrows to 300 metres just upstream of the bridge site as it is confined by the limestone outcrop on the right and the TID on the left. At the bridge crossing site, the channel widens to approximately 450 metres. The river plan is shown on Figure 3.1.

The Athabasca River channel in the vicinity of the proposed bridge is a stable, entrenched sand bed channel which exhibits regime conditions. It does not exhibit obvious signs of aggradation or degradation. It is currently evolving as a laterally confined, meandering system. Any tendency for downcutting is restricted by the low channel gradient which is 0.00014 at the bridgesite and downstream of the bridge site and 0.00026 upstream of the bridge site. Any tendency for downcutting is also constrained by level of Lake Athabasca to the north. There is little potential for riverbed degradation or major channel migration at the bridge site.

The current bridge alignment and location of the proposed bridge abutments may have been based upon the direction of flow during low flow conditions and the presence of a low level point bar on the left bank immediately upstream of the left abutment. This bar was removed by erosion in 1995 indicating possible entrenchment and shifting of the thalweg towards the left bank. At high flows the direction of flow tends to straighten. As a result, the current proposed bridge alignment is skewed approximately 8 degrees from perpendicular to the direction of flow during flood conditions. The implications of this are discussed further in Section 5.

PHOTO DATE: AUGUST 18, 1993

APPROX. SCALE 1:8600 0 100 200 300m

AGRA
Earth & Environmental Limited
ENGINEERING & ENVIRONMENTAL SERVICES

CLIENT:	H.A SIMONS LTD.	DATE:	JAN 1996
PROJECT:	SUNCOR–ATHABASCA RIVER BRIDGE	JOB No.	CW1466.00
	ATHABASCA RIVER AT TAR ISLAND DYKE	CAD FILE:	
		FIGURE 3.1	REV. –

3.2 HYDRAULIC ANALYSIS

Hydraulic conditions in a 5 kilometre reach of the river were defined based upon 14 surveyed river cross-sections which are shown on graphs in Appendix A and provided on diskette also in Appendix A. These sections are based on surveys in July 1995 by AEE and surveys in September 26-29, 1995 by Suncor. Both surveys are based on the Suncor datum which is estimated to be 0.95 metres higher than Geological Survey of Canada (GSC) datum. Most of the sections in Appendix A are based on the surveys by AEE because the Suncor sections do not extend from bank to bank. All elevations referred to in this report are based on the Suncor datum unless otherwise noted.

Open water hydraulic conditions were analyzed using the HEC-2 computer model (an industry standard one-dimensional backwater program developed by the U.S. Corps of Engineers) with the above cross-sections. Water surface elevations and velocities of flow through the study reach were computed for a range of discharges from 250 m^3/s to 7700 m^3/s. Model calibration was based upon a rating curve constructed from 1977 to 1995 water levels recorded at Suncor's Fresh Water Inlet and corresponding discharges recorded at the WSC gauge below Fort McMurray. Model runs start downstream at section 13 assuming a starting water level based on the slope-area method and an assumed channel roughness coefficient (Manning's n) of 0.025. The constructed stage versus discharge rating curve and resulting water levels computed for Section 12 located approximately 130 metres upstream of the inlet are shown on Figure 3.2. Predicted water levels at Section 12 are 0.62 metres higher than observed for the mean annual discharge (655 m^3/s) but were within 0.1 metres for most other discharges analyzed as shown in Figure 3.2. Observed water levels along TID during the course of other studies by AEE for Suncor in 1995 also confirm predicted water levels are comparable to observed levels for equal discharges.

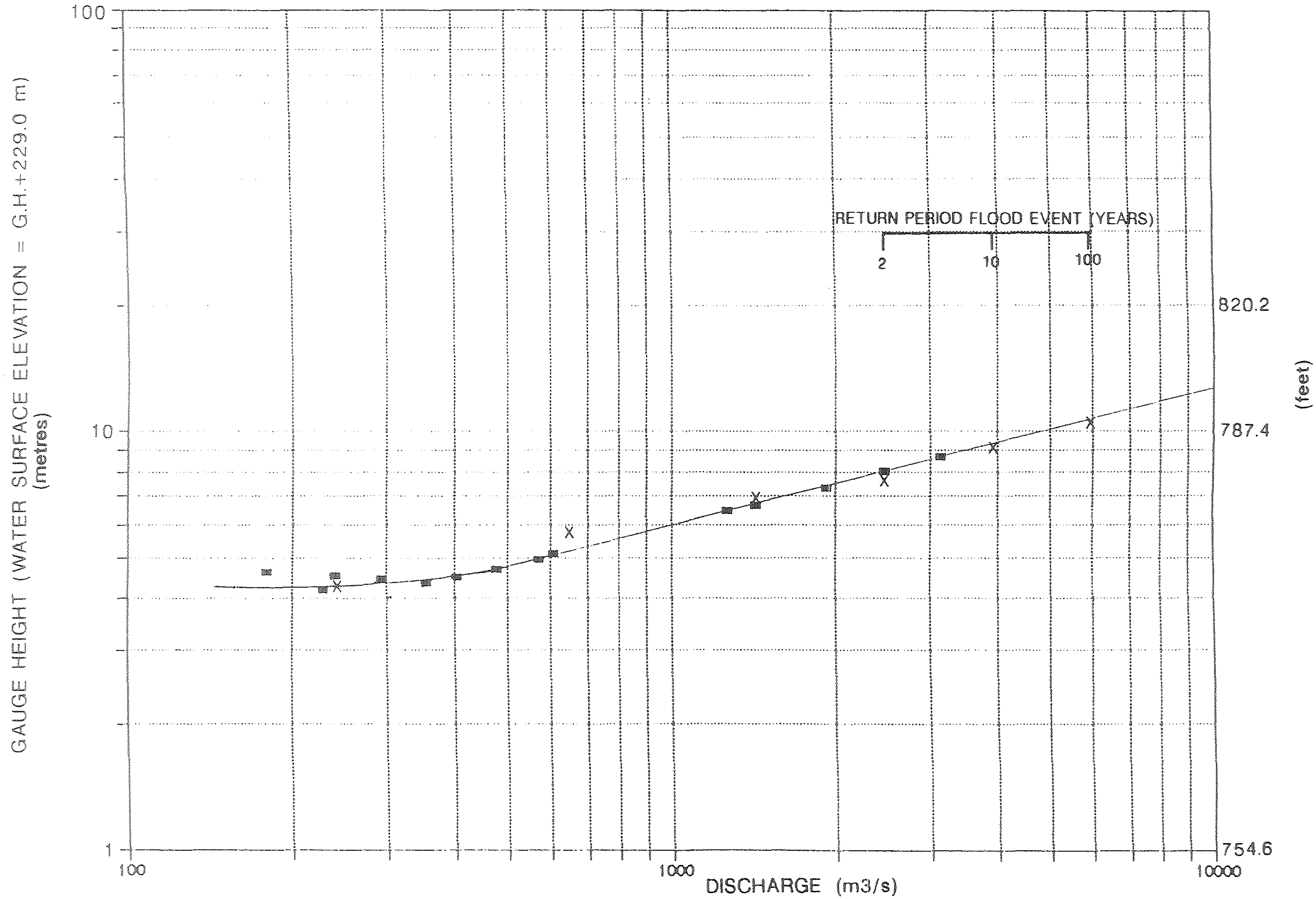
The resulting mean channel flow velocities for existing conditions range from approximately 1.3 m/s during the mean annual flood to 1.9 m/s during the 1:100 year flood at the bridge section. Corresponding mean channel flow depths are 4.3 metres and 6.5 metres, respectively, for these flood events.

3.3 HYDRAULIC DESIGN PARAMETERS

3.3.1 Impact of Cofferdams on River Water Levels

Predicted water levels and flow velocities during cofferdam construction and operation were evaluated for the Four-Pier Bridge design scheme. However, similar cofferdams may be used for construction of the Three-Pier Bridge.

Two cofferdam configurations have been evaluated, as defined below. Right and left bank abutments are also assumed to be in-place prior to pier construction. These abutments have minimal impact on the flow constriction beyond that assumed for the cofferdams.



RECORDED LEVELS AT SUNCOR'S FRESH WATER INLET (1977-1995)

X PREDICTED HEC-2 LEVELS

AGRA
Earth & Environmental Limited
ENGINEERING & ENVIRONMENTAL SERVICES

CLIENT: H.A. SIMONS LTD.	DATE: JANUARY 1996
PROJECT: SUNCOR–ATHABASCA RIVER BRIDGE	JOB No. CW1466.00
RATING CURVE FOR ATHABASCA RIVER AT SECTION 12	FIGURE: FIGURE 3.2. REV. 0

Base Case Cofferdams Configurations: Cofferdams for construction of Piers 1, 2 and 3, will be in place during the open water season and a cofferdam for construction of Pier 4 will be in place during the winter as follows:

Pier 1: earthfill cofferdam from March to September;
Piers 2 and 3: 34.5 metre diameter sheetpile cofferdams from June to October; and
Pier 4: earthfill cofferdam from freeze-up in December to break-up in March.

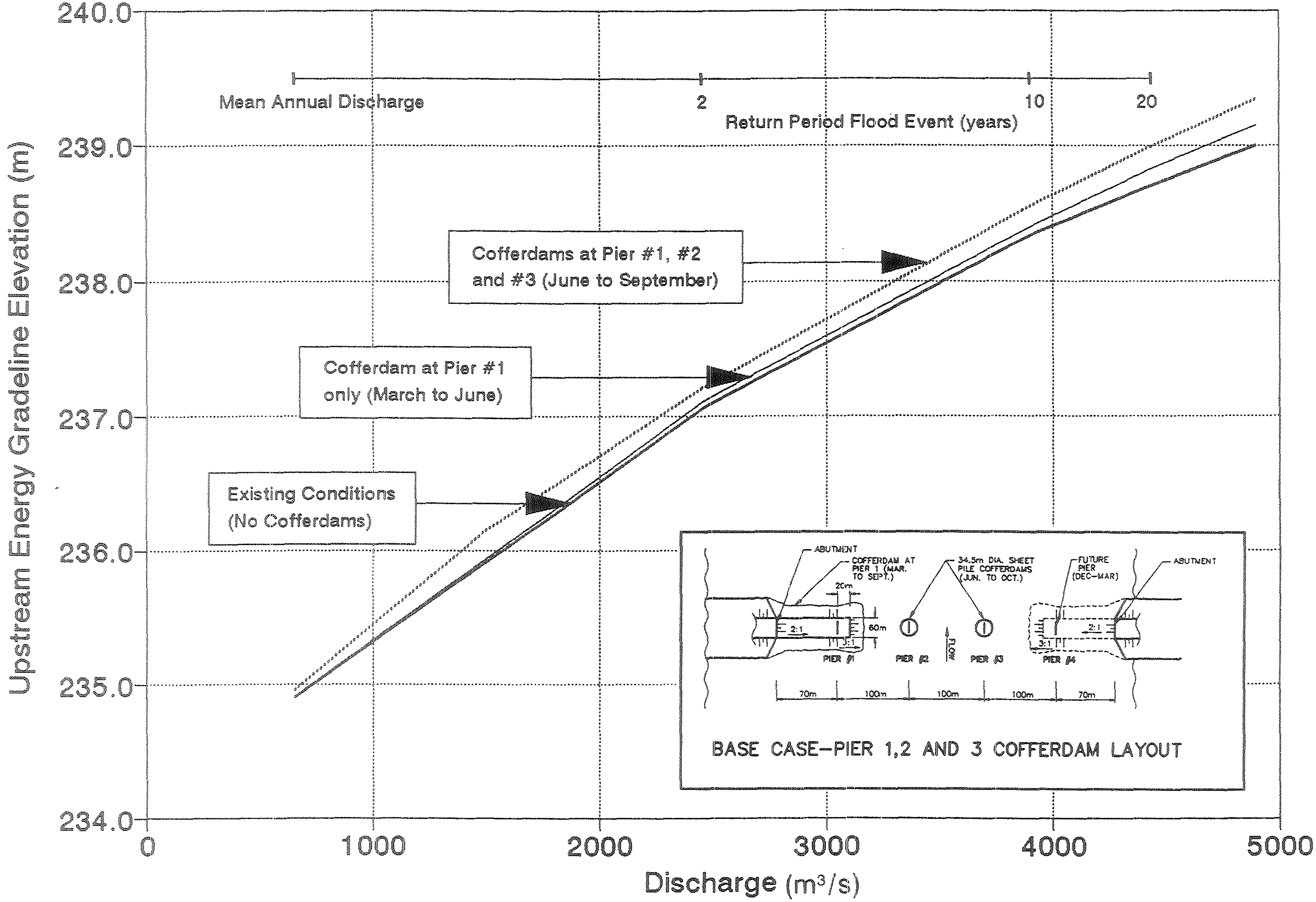
The assumed cofferdam configurations and resulting water levels for construction of Piers 1, 2, and 3 are shown on Figure 3.3. Figure 3.3 shows the water level impacts of two cofferdam configurations, one from March to June when there is only one cofferdam at Pier 1 and another for June to September when there are three cofferdams in place for each of Piers 1, 2 and 3. Since annual peak discharges can occur after June as frequently as they occur during or before June, cofferdam sizing and protection should be based on the case where all three cofferdams are in-place during the selected design flood event. Water levels are only increased by approximately 0.29 metres at the 1:20 year flood as a result of the three cofferdams. Impacts of the three cofferdams on mean channel flow velocities and water levels are summarized in Table 3.1.

TABLE 3.1
Impact of Base Case and Alternative Cofferdams Configurations on Channel Velocity and Water Levels During the Open Water Construction Period

Return Period Flood Event (year)	Existing Mean Channel Velocity (m/s)	Base Case Cofferdams[1]			Alternative Cofferdams[2]		
		Mean Channel Velocity (m/s)	Increase in Velocity over Existing (m/s)	Increase in Water Level over Existing (m)	Mean Channel Velocity (m/s)	Increase in Velocity over Existing (m/s)	Increase in Water Level over Existing (m)
2	1.35	2.36	1.01	0.18	2.75	1.40	0.26
10	1.69	2.87	1.18	0.25	3.32	1.63	0.37
20	1.79	3.03	1.16	0.29	3.50	1.71	0.42
50	1.95	3.05	1.10	0.29	3.75	1.80	0.48
100	2.04	3.07	1.03	0.25	3.93	1.89	0.52

Note: 1. Construction period for Base Case Cofferdam Configuration for Piers 1, 2 and 3 in place is March - October. Refer to Figure 3.2 for details.
2. Construction period for Alternative Cofferdam Configuration is March to September. Refer to Figure 3.3 for details.

PREDICTED WATER LEVELS FOR BASE CASE
COFFERDAM CONFIGURATION DURING JUNE TO SEPTEMBER



NOTE: Based on assumed cofferdam level of 238.5 m at Pier #1.

Figure 3.3.

A cofferdam level of 239.0 metres is expected to be the minimum design level for the Pier 1 Base Case Cofferdam. This provides approximately 0.5 metres freeboard during a 1:10 year flood event and zero freeboard during the 1:20 year flood event. Varying the cofferdam level by ± 1 metre from the 238.5 metre level assumed in the derivation of the curves in Figure 3.2 has negligible impact on the rating curve and results presented.

Alternative Cofferdams Configuration: An alternative cofferdam configuration and resulting water levels are compared with existing conditions in Figure 3.4. This alternative arrangement assumes that cofferdams at Piers 1 and 2 are in place during the March to September open water season and that cofferdams at Piers 3 and 4 are in place during the December to March winter period.

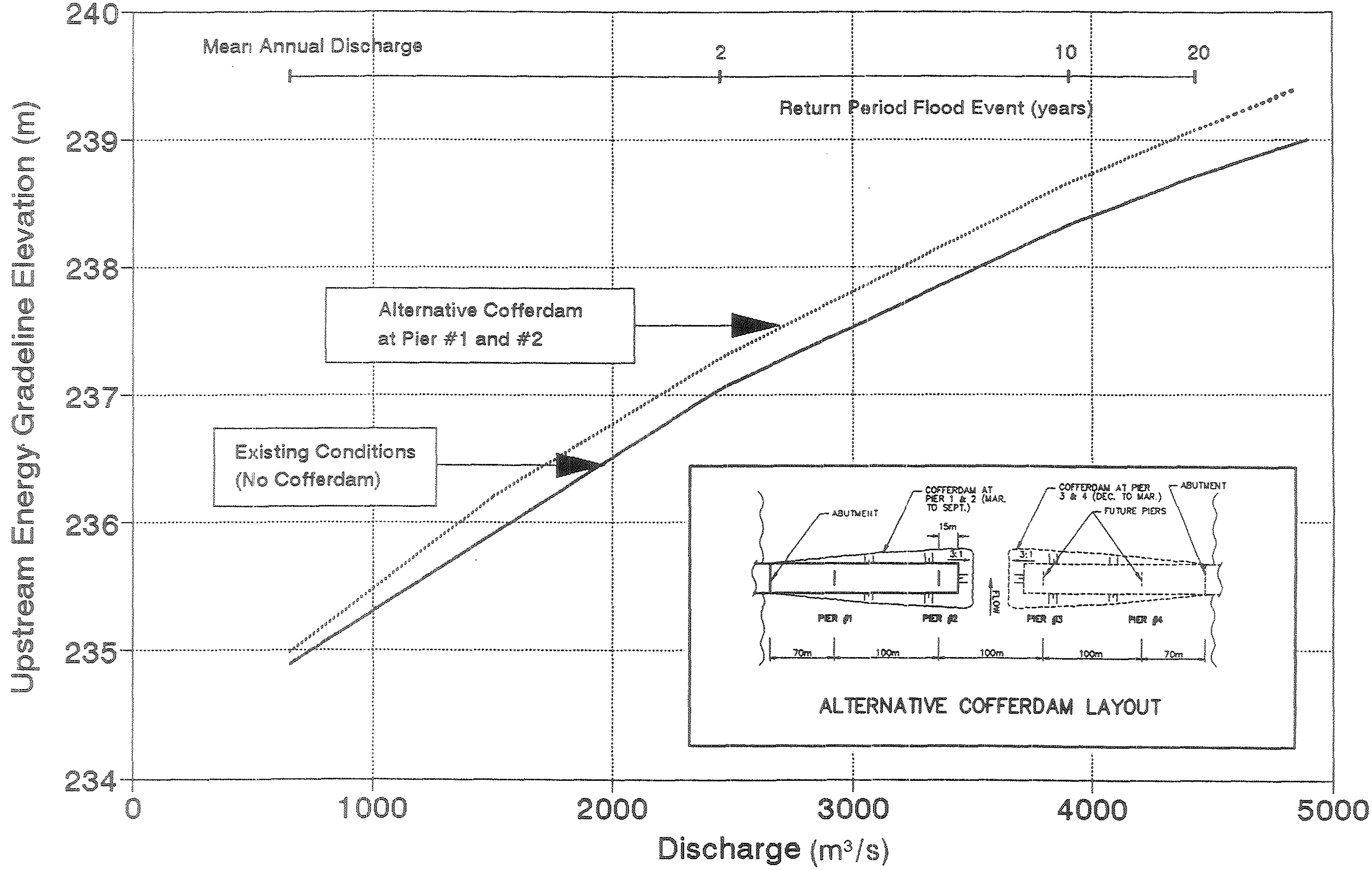
The alternative cofferdam configuration causes a much greater constriction in the river than the Base Case cofferdam configuration. The channel topwidth at the 1:20 year flood level is reduced from 412 metres for existing conditions to 272 metres for the Base Case configuration and to 245 metres for the Alternative cofferdam configuration. The water levels for the alternative cofferdam configuration are increased 0.42 metres above existing conditions during the 1:20 year flood event. Other backwater levels and mean channel flow velocity impacts are summarized in Table 3.1.

3.3.2 Impact of Bridge Abutments and Piers on Bridges

The assumed four pier bridge scheme and resulting water levels are shown on Figure 3.4. The design low chord elevation is 253.4 metres based on a minimum clearance of 15.2 metres from the 1:10 year navigation flood level of 238.2 metres. The minimum clearance of 15.2 metres above the 1:10 year flood level was established by the Canadian Coast Guard.

Based on the monthly flow duration curves and river level rating curve shown in Figure 3.5, the variations in expected water levels at the bridge through the open water season are summarized in Table 3.2.

PREDICTED WATER LEVELS FOR ALTERNATIVE COFFERDAM CONFIGURATION DURING MARCH TO SEPTEMBER



NOTE: Based on assumed cofferdam level of 239.5 m at Pier #1 and #2.

Figure 3.4.

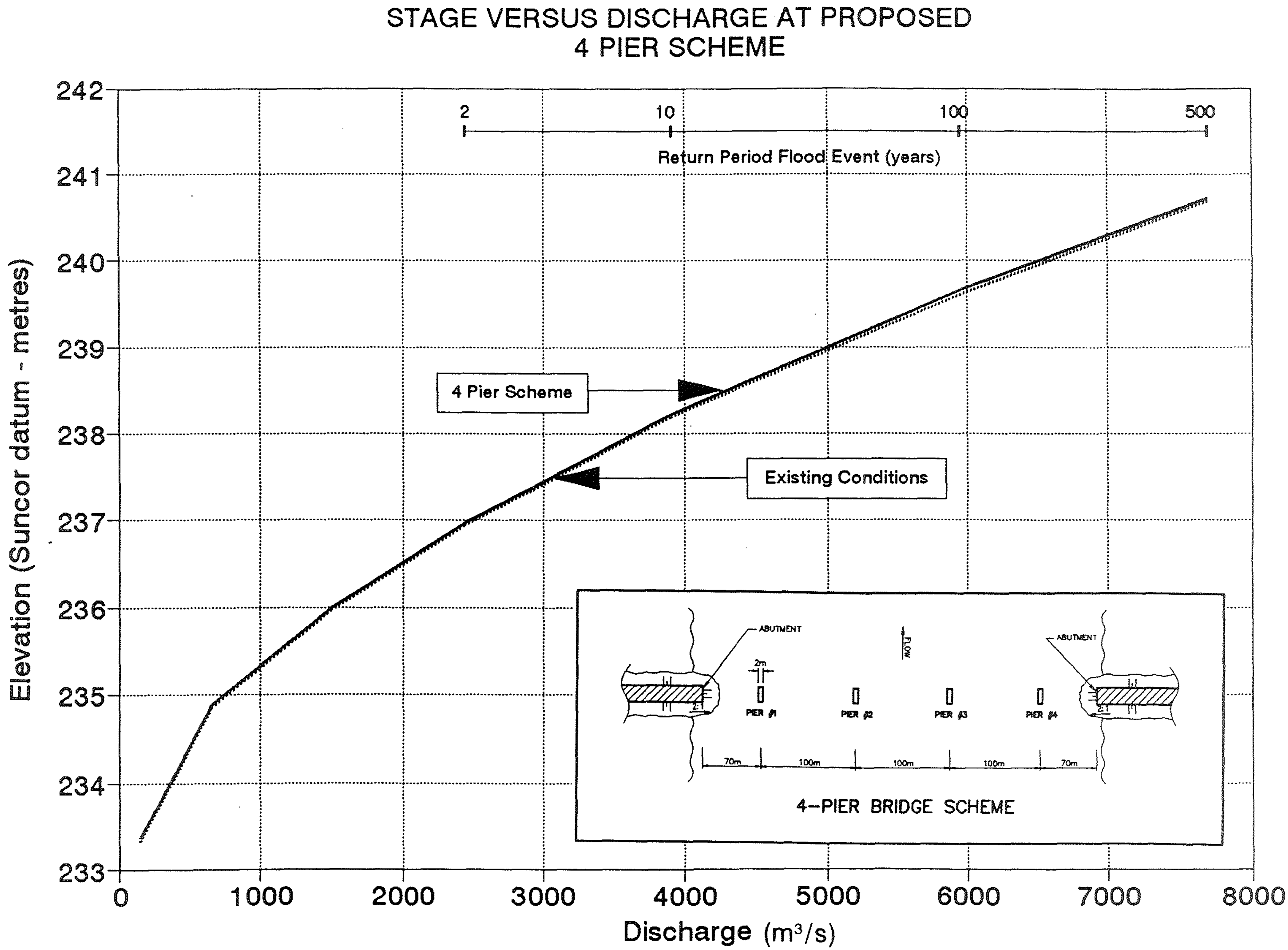

STAGE VERSUS DISCHARGE AT PROPOSED
4 PIER SCHEME
Elevation (Suncor datum - metres)
Discharge (m³/s)
Return Period Flood Event (years)
2
10
100
500
4 Pier Scheme
Existing Conditions
ABUTMENT
FLOW
2m
PIER #1
PIER #2
PIER #3
PIER #4
70m
100m
100m
100m
70m
4-PIER BRIDGE SCHEME
242
241
240
239
238
237
236
235
234
233
0
1000
2000
3000
4000
5000
6000
7000
8000

Figure 3.5.

TABLE 3.2
Estimated Ice Free Water Levels at the Bridge

Month	Percent of Time Level is Equaled or Exceeded (Metres - Suncor Datum)		
	75%	50%	25%
April	233.59	234.03	234.90
May	234.95	235.34	235.81
June	235.27	235.66	236.14
July	235.27	235.70	236.20
August	234.98	235.23	235.52
September	234.62	234.97	235.17
October	234.21	234.64	234.91
November	233.59	233.86	234.21

A three pier bridge scheme with vertical abutments (instead of 2H:1V abutments for the four pier bridge scheme) would cause minor increases in the water levels as indicated on Figure 3.6. Increases in water levels and velocities as a result of both the three pier and four pier schemes are minor, as indicated in Table 3.3.

TABLE 3.3
Impact of Three and Four Pier Schemes on
Channel Velocities and River Levels

Return Period Flood Event (year)	Existing Mean Channel Velocity (m/s)	Four Pier Scheme			Three Pier Scheme		
		Mean Channel Velocity (m/s)	Increase in Velocity over Existing Load (m/s)	Increase in Water Level over Existing Conditions (m)	Mean Channel Velocity (m/s)	Increase in Velocity over Existing Conditions (m/s)	Increase in Water Level over Existing Conditions (m)
2	1.35	1.40	0.05	0.02	1.78	0.43	0.07
10	1.69	1.72	0.03	0.04	2.20	0.51	0.11
20	1.79	1.83	0.04	0.05	2.34	0.47	0.13
100	2.04	2.08	0.04	0.07	2.67	0.63	0.18

BACKWATER IMPACTS FOR 3 AND 4
PIER SCHEMES

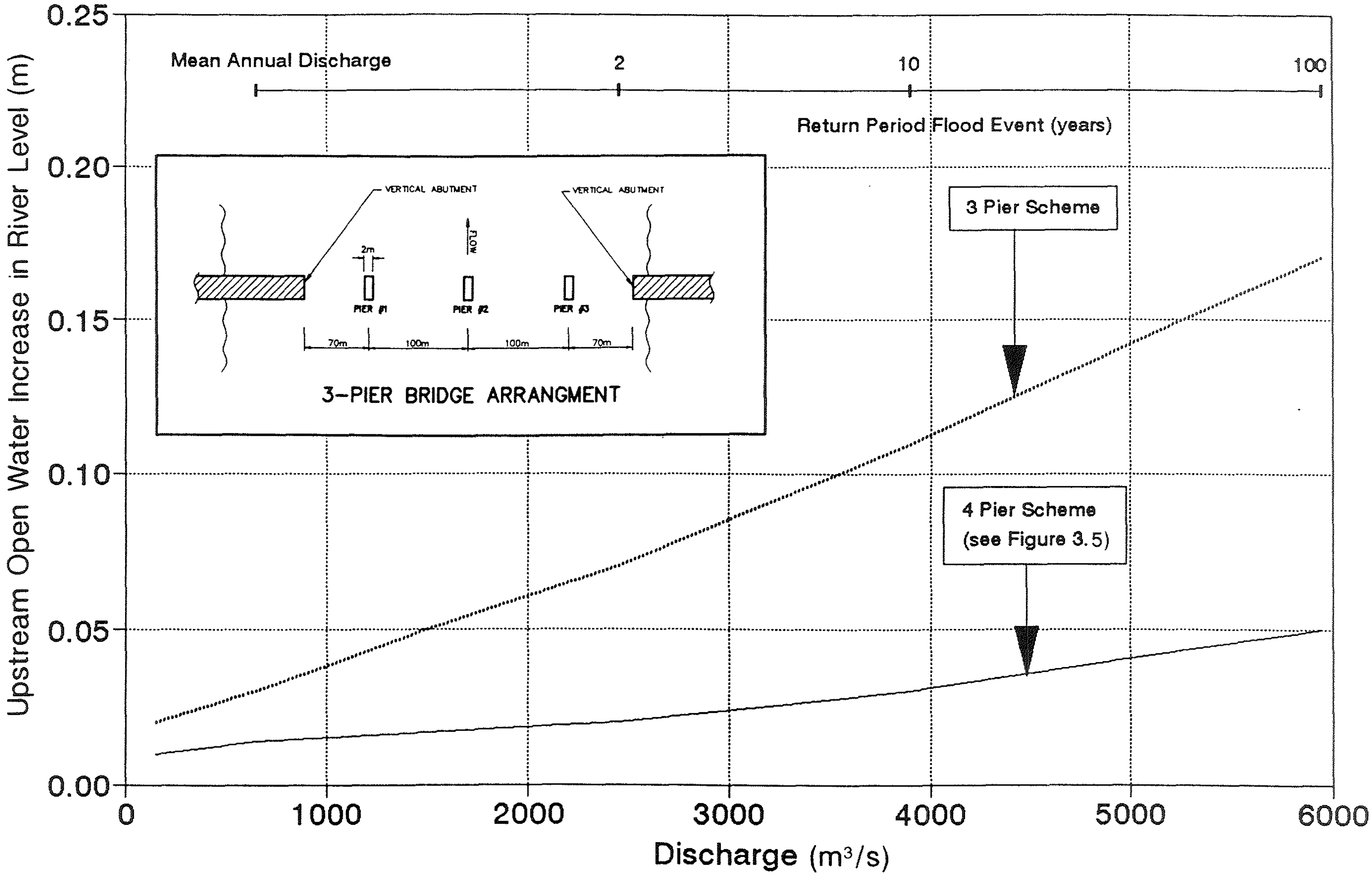


Figure 3.6.

4.0 ICE CONDITIONS

4.1 ICE JAM FLOOD FREQUENCIES

4.1.1 Historical Observations and Ice Jam Processes

Observations of breakup and measurements of ice jams have been carried out in a systematic way since the early 1970's (Gerard, 1975; Andres *et al.*, 1984, 1985a, 1985b; and Malcovish *et al.*, 1988). This has provided a good understanding of the breakup processes in the river reach between Fort McMurray and Suncor and has also provided a good quantitative description of the characteristics of the ice jams.

Prior to this work, Blench (1964) undertook a review of the historical data of peak spring breakup elevations, quoting Hudson Bay Company data that went as far back as 1875. From this work it was possible to produce frequency curves of breakup elevations at Fort McMurray (Andres in Watt *et al.* (1989) and AEP (1993)). With appropriate adjustments, these frequency curves are applicable to the bridge site at Suncor.

Breakup on the Athabasca River progresses in a downstream direction from the mouth of the Pembina River. Runoff from the plains area in the middle portion of the Athabasca River Basin first initiates breakup upstream of the Town of Athabasca. The breakup front moves downstream, through a process that alternates between the accumulation of ice floes, the creation and destruction of jams, and the development of surges that fracture the ice cover. Large jams typically form in the rapids areas between Grande Rapids and Fort McMurray as the ice front works its way downstream through the thickened ice cover in this part of the river. The rising discharge, narrow channel, and the steep channel gradients upstream of Fort McMurray maintain the breakup momentum.

Once the ice front enters the reach downstream of Fort McMurray, the reduced channel slope, the increased channel width, and the numerous islands reduce the transport capacity and the breakup process becomes much less dynamic. A jam of some magnitude and duration forms annually, but its characteristics depend on the combination of meteorological and hydrologic characteristics that prevail in each winter and spring, and on the ice jamming sequence upstream. The severity of the ice run and the location of the toe of the jam in the Suncor reach depends on the discharge in the river, the location of the toe of the upstream jam that precipitated the ice run, and the strength of the ice cover downstream of Fort McMurray. Thus, it is a purely stochastic process that is amenable to frequency analyses.

<u>Breakup Dates</u>

A historical record of the date of breakup (date of ice out) is somewhat incomplete because of the different ways in which breakup is defined. Water Survey of Canada (WSC) defines the date of ice out as the last date for which there are no backwater effects. On the other hand, from direct observations, the date of breakup is defined as the day on which the ice is fractured with a significant rise in stage, due either to a surge of ice or the creation of an ice jam. Often the date of last backwater can lag the date of breakup by about 5 to 10 days.

Breakup typically occurs in the month of April. The WSC definition of last ice, as reported in Kellerhals, Neill, and Bray (1972) for only 12 years of record suggests that the earliest recorded breakup was on April 16, the latest breakup date was May 7, and the average breakup date is April 28. On the other hand, from actual observations of breakup between 1977 and the present, the earliest date of breakup was April 11 (note that some of the data suggests that March 27 may be the earliest recorded breakup date), the latest breakup date was April 28 (note also that some data suggests that May 3 is the latest breakup date), and the average breakup date is April 19. In nine out of ten years breakup occurs between April 15 and April 25 (T. Winhold, personal communication).

<u>Ice Run Characteristics</u>

Ice runs occur when ice jams that have formed upstream of Fort McMurray release due to a rising carrier discharge (the background discharge in the river, independent of the surge-related discharge). This ice and water then moves into the study reach. The stage of these ice runs is very close to that of the equilibrium jam stage of the source jam as long as the surge is moving into a channel with solid ice. There is very little attenuation in the stage because of the forces of the solid ice cover on the moving ice jam. The velocity (celerity) of the breaking ice front is a function of the height of the surge and has been measured to be as great as 5.5 m/s for short periods of time, but it typically averages about 3.5 m/s (Andres and Doyle, 1984).

The velocity of the ice floes in the centre of the channel have been measured during an ice run at the MacEwan Bridge at Fort McMurray on a number of occasions. Typically, the floe velocity varies between 4 and 5 m/s during the ice run prior to jamming and about only 2.0 m/s when the stable jam is collapsing. It should be noted that short-term peak velocities of between 7 and 8 m/s have been measured on one occasion. In view of the stage of the ice run for that particular event and the magnitude of the other measured ice run velocities, this high velocity is very unusual and its measurement may be in error.

The rate of stage increase during the ice run can vary substantially. Measurements at MacEwan Bridge indicate that the rate of rise can vary between 1 and 5 m/h. Drawdown rates would be similar if the surge moves by without jamming or if a jam should fail mechanically. In the event of thermal destruction, the drawdown rate is typically 0.5 m/h. It should be noted that a rapid drawdown usually is associated with either an ice run or a jam of very short duration.

The floe sizes in the surges and in the jams are quite variable in space, usually getting smaller as one moves upstream from the toe of either the running ice or the jam. Typically, the maximum ice thickness is in the order of 1.5 metres. This is considerably greater than the in situ ice measured at Fort McMurray (see later discussion) probably because the ice source is the very thick ice from the rapid areas upstream. The sizes of the floes are typically in the range of 4 to 5 metres in diameter, but can be as large as 20 to 30 metres in diameter. The dominant floe size of 4 metres is similar to the thicknesses of the jams as deduced from measurements of the shearwalls. These shearwalls are thickened ice left attached to the bank on the shore side of the shear lines (boundary between the moving ice and the shorefast ice) after the jam or the ice run. The height of the shear walls are thought to reflect the thickness of the jam.

In situations when an ice jam forms, stabilizes, and releases into very deteriorated ice or open water (as may exist downstream of the Fort McMurray Water Treatment Plant and downstream of Suncor) the surge attenuates very rapidly. The peak stage is reduced by about 50 percent by the time the surge moves downstream a distance equal to the length of the jam.

Toe Locations

As mentioned earlier, the location of the toe of the jam can vary from year to year, depending on the location of the upstream jam, the discharge, the ice strength, etc. Observations over the last 20 years or so (early 1970's to the present) suggest that the location of the toe can be anywhere between MacEwan Bridge and Suncor. In those years when no jams formed in the reach, a jam typically formed at or just upstream of MacEwan Bridge. This jam did not release until the ice downstream of MacEwan Bridge deteriorated to the point where the surge released into open water and hence attenuated very rapidly, or the ice jam simply melted in place and breakup downstream was very mild. Table 4.1 summarizes the observed locations of toes of stable ice jams in the study area.

TABLE 4.1
Summary of Ice Jam Toe Locations in the Vicinity of the Suncor Bridge

Year	Location of Toe (km) [1]	Distance Upstream of Proposed Bridge (km)	Comments
1977	286	23	Most downstream toe lodged against solid ice.
	291	28	Thickened part of jam located at a change in channel geometry.
1978	296	33	Toe at MacEwan Bridge, lodged against solid ice.
1979	269	6	Most downstream extent of jam, lodged against solid ice.
	280	17	Thickened part of jam located at a change in channel geometry.
1986	288	25	Jam toe lodged against solid ice with open water downstream.
1987	284	21	Toe is lodged against solid ice in a partially opened channel.
	290	27	Thickening of jam due to internal collapse of ice jam.

[1] Measured from mouth of Athabasca River at Lake Athabasca

It should be noted that the observation program changed after 1987, however, significant jams have not been reported after that date. From the above table it is apparent that most of the stable jams form upstream of the proposed bridge. In 1979 a stable jam formed within six kilometres of the proposed bridge. It should be noted that this jam was also the second most severe in recent history.

The likelihood of a severe jam forming at both Fort McMurray and the proposed bridge site in the same year is low. Only in five years out of the 15 years of measurements did a severe jam form that could have affected Suncor. That is, from the historical record, it is apparent that severe jams at Fort McMurray are severe (with a return period greater than 20 years) because the ice front stalls at the City without moving downstream to Suncor. This tends to insulate Suncor from the very high ice jam levels that have occurred at Fort McMurray. Thus, the probability of Suncor experiencing ice jams greater than the 1 in 20 year event at Fort McMurray is about only 30 percent of the probability of occurrence of a jam of the same characteristics at Fort McMurray.

The lower ice jam stages (with a return period less than 20 years) at the City result from the formation of temporary jams and/or surges that are also experienced at Suncor. Thus the probability of the City and Suncor both experiencing an ice jam stage that is less severe than the 1 in 20 year event at Fort McMurray is the same.

Over the 15 years that observations have been carried out, stable jams were observed in this reach only five times. In the other years the ice runs moved through the reach and only transient jams were produced, ie. they did not exist long enough to be measured. These temporary jams have been observed as far downstream as Fort Mackay. This suggests that even though stable jams were not observed at the bridge site, the potential is high for transient jamming and for the toes of such jams to occur within the bridge waterway. Fortunately, the transient nature of the jams will limit the time available to develop a maximum size of scour hole.

Duration of Jams

As mentioned above, the duration of the jams can vary substantially. In some years the ice run has sufficient momentum to move through the reach without developing jams. In other years, the jams may last only a few hours, and as the carrier discharge increases, the jams remain unstable and continue to move downstream. In other years, such as those identified in Table 4.1, a jam forms in the reach, the carrier discharge decreases, the jam gains stability, and it remains in place until it is destroyed by melting. In those cases the duration of the jam can be up to 14 days, depending on the weather conditions. The durations of the 1977, 1978, and 1979 jams were 8, 9, and 5 days respectively. The 1986 jam produced high water levels for five days and the 1987 jam only lasted two days.

4.1.2 Stage-Frequency Curves

Historical evidence indicates that ice jams are the dominant flood producing mechanism in this reach. Ice jams levels on the Athabasca River at the confluence of the Clearwater River (located some 30 kilometres upstream of the proposed bridge site) have exceeded 250 metres (GSC) twice since 1875 and 248 metres five times since that date. The most recent high ice jam level was 247.9 metres (GSC) in 1977. Alberta Environmental Protection (AEP) has carried out frequency analyses of the observed ice jam levels using the perception stage method proposed by Gerard and Karpuk (1979) and described by Andres in Watt *et al.* (1989). The AEP analyses corroborates a similar type of analysis that was carried out by Andres, also found in Watt *et al.* (1989).

Figure 4.1 illustrates the open water rating curve and the theoretical ice jam rating curve as compared with the measured data for both open water and ice jam events for the reach between the mouth of the Clearwater River and MacEwan Bridge. The ice jam rating curve was calculated using the equilibrium ice jam approach.

The reach-average hydraulic characteristics in the Fort McMurray reach of the river are represented by the cross sections at MacEwan Bridge. The channel is almost rectangular in shape, with an active width of 425 metres. The average slope is 0.00047 (Malcovish, Andres, and Mostert, 1988). The ice jam and open water levels are referenced to the mouth of the Clearwater River, to be consistent with the frequency analyses.

The measured data suggests that the bed roughness n_b is about 0.025 for the kinds of flow depths that one might experience under ice jams. The ice jam data can be reasonably well reproduced using an ice roughness n_i of 0.060 and a coefficient of internal friction μ of 1.0. With these values the calculated ice thicknesses for the observed jams vary between 3.9 metres and 4.7 metres. This is similar to the estimated ice thickness from the measurements of the shear walls.

Table 4.2 summarizes the stage frequency curve at the mouth of the Clearwater River and the carrier discharge that would be responsible for such a water elevation. The discharge was scaled off the ice jam rating curve shown in Figure 4.1.

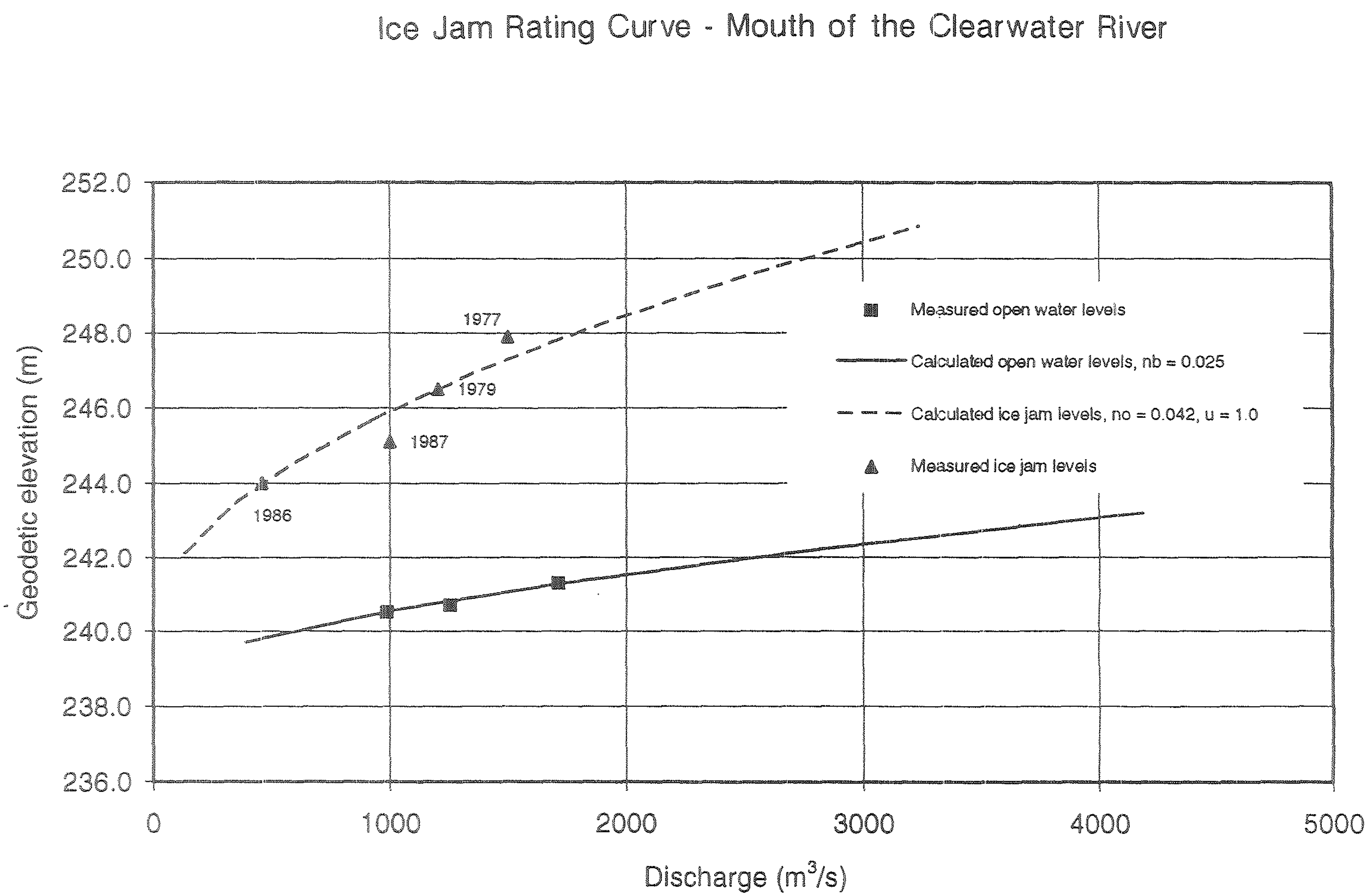

Ice Jam Rating Curve - Mouth of the Clearwater River
Geodetic elevation (m)
252.0
250.0
248.0
246.0
244.0
242.0
240.0
238.0
236.0
0
1000
2000
3000
4000
5000
Discharge (m³/s)
1977
1979
1987
1986
Measured open water levels
Calculated open water levels, nb = 0.025
Calculated ice jam levels, no = 0.042, u = 1.0
Measured ice jam levels

Figure 4.1.

TABLE 4.2
Summary of Breakup Stage Frequencies and Representative Discharges at the Mouth of the Clearwater River

Return Period (years)	Geodetic Elevation (m) at the Mouth of the Clearwater River	Representative Discharge[1] (m^3/s)
100	250.5	3160
50	249.2	2450
20	247.2	1530
10	246.0	1050
5	244.8	680
2	243.3	320

[1] Calculated from equilibrium ice jam theory using channel hydraulic characteristics at the mouth of the Clearwater River.

Two factors must be considered when transferring the frequency curve from the mouth of the Clearwater River to the proposed Suncor bridge site: (1) the hydraulic conditions that define the jam characteristics and (2) the representativeness of the ice jam frequencies at the Clearwater River of those at Suncor.

<u>Ice Jam Characteristics at Suncor</u>

The characteristics of the ice jams at Suncor are somewhat more complicated than those at the Clearwater River. First, the bed is much more mobile at Suncor, thereby making the arguments about the relationships between the bed and ice roughness more complicated. That is, upstream of the mouth of the Clearwater River the channel is composed of coarse material which does not transport easily under the discharge regime evident during breakup. On the other hand, the median bed material size of 0.3 mm at the proposed bridge site is mobile to various degrees under most discharges. Thus the bed forms and the subsequent bed roughness are coupled to the discharge itself. Second, the ice jam levels are a function of the toe conditions (and the likelihood of grounded toes in this reach) because of the general proximity of the historical jam toes to the bridge site.

Fortunately, the lower portion of the 1979 jam is a good representation of jams in the area of the proposed bridge site. By running the **RIVJAM** model to reproduce the observed jam levels, it was possible to quantify the hydraulic characteristics of the jam and the characteristics of the toe and its effect on the ice jam configuration. By matching the water levels in the vicinity of the toe it was deduced that grounding was a necessary condition to produce the measured ice jam profiles. From that deduction it was then possible to determine the appropriate composite and ice roughnesses. Table 4.3 summarizes the measured 1979 ice jam characteristics and the calibrated parameters necessary to reproduce the measured ice jam profile shown in Figure 4.2.

Measured and Modelled 1979 Ice Jam Profile
at a Location 6 km Upstream of Bridge Site

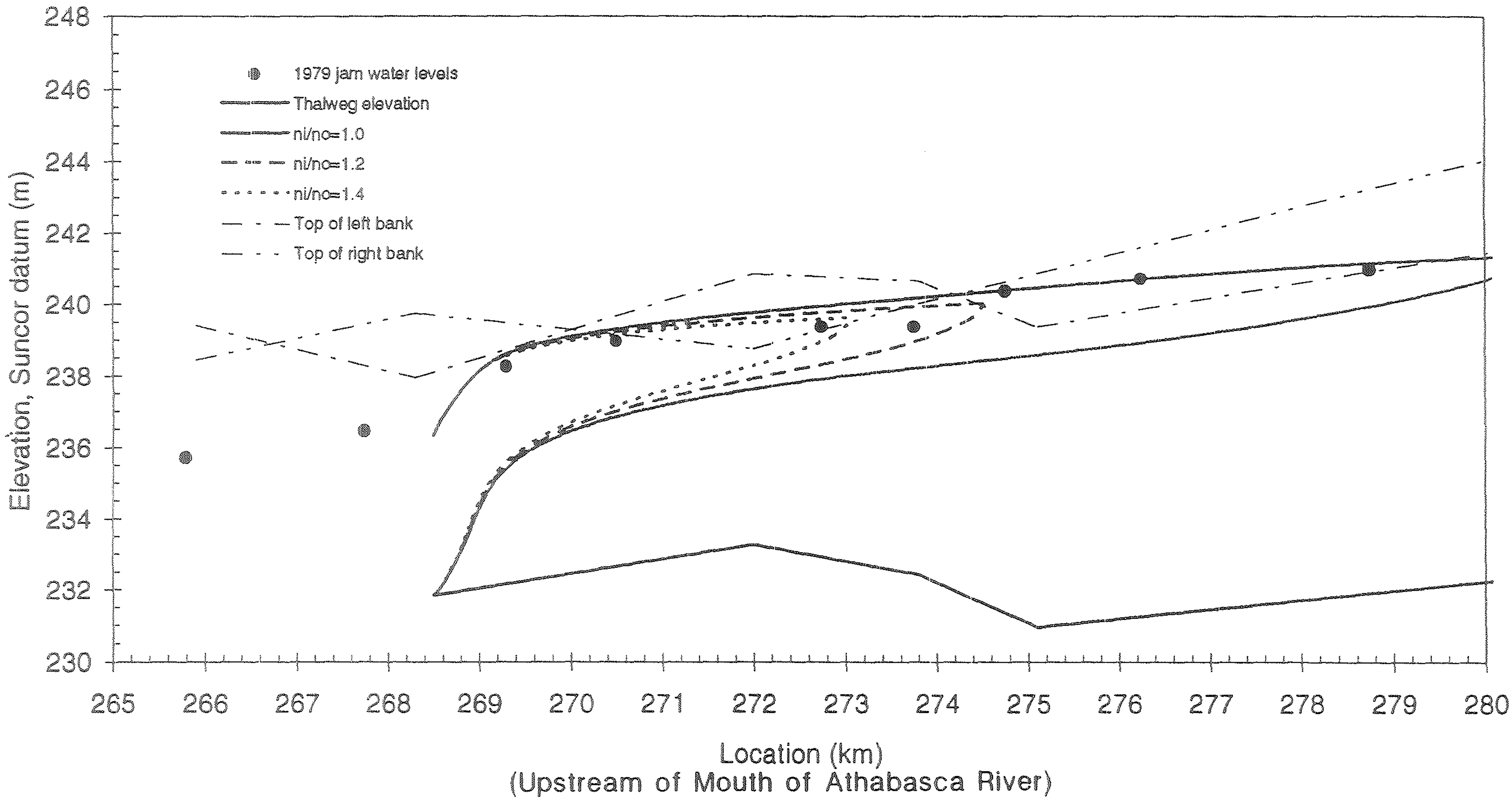


Figure 4.2.

TABLE 4.3
Characteristics of the Measured and Modelled 1979 Ice Jam Upstream of Suncor

Characteristic/Parameter	Value
Measured characteristics:	
Discharge (m^3/s)	1200
Typical channel width (m)	350 - 400
Bed slope (m/m)	0.00015
Typical total stage (m)	8.0
Prescribed parameters in RIVJAM:	
Porosity of ice jam	0.4
Dimensionless coefficient of internal friction (calibrated at Fort McMurray)	1.0
Seepage parameter	10.0
Ratio of vertical to horizontal ice pressures	12.0
Calibrated parameters:	
Toe conditions	grounded
Composite Manning roughness	0.042
Ratio of Manning ice roughness to Manning composite roughness[1]	1.0
Manning ice roughness	0.042

1 This may not be theoretically correct, but this ratio works the best for the assumptions in the model.

It should be noted that the grounding of the toe is an important process that determines the characteristics of the jams near the toe. Furthermore, it is apparent that the composite roughness and the ice roughness is somewhat lower than that measured at Fort McMurray. This is due to the more mobile bed (this effects the roughness differential between the bed and the ice) and the different flow depths under the ice, relative to the size of the roughness elements of the ice cover.

The equilibrium ice jam elevations for the various exceedence probabilities and their associated discharges were calculated for the proposed bridge site using the calibrated ice jam characteristics and the hydraulic characteristics developed from the surveyed cross sections. Table 4.4 summarizes the reach-average hydraulic characteristics and the calculated bed roughness for the open water measurements. Figure 4.3 illustrates the open water rating curve, the pre-breakup rating curve (ice thickness and roughness determined from the WSC

Ice Jam Rating Curve at Bridge Site

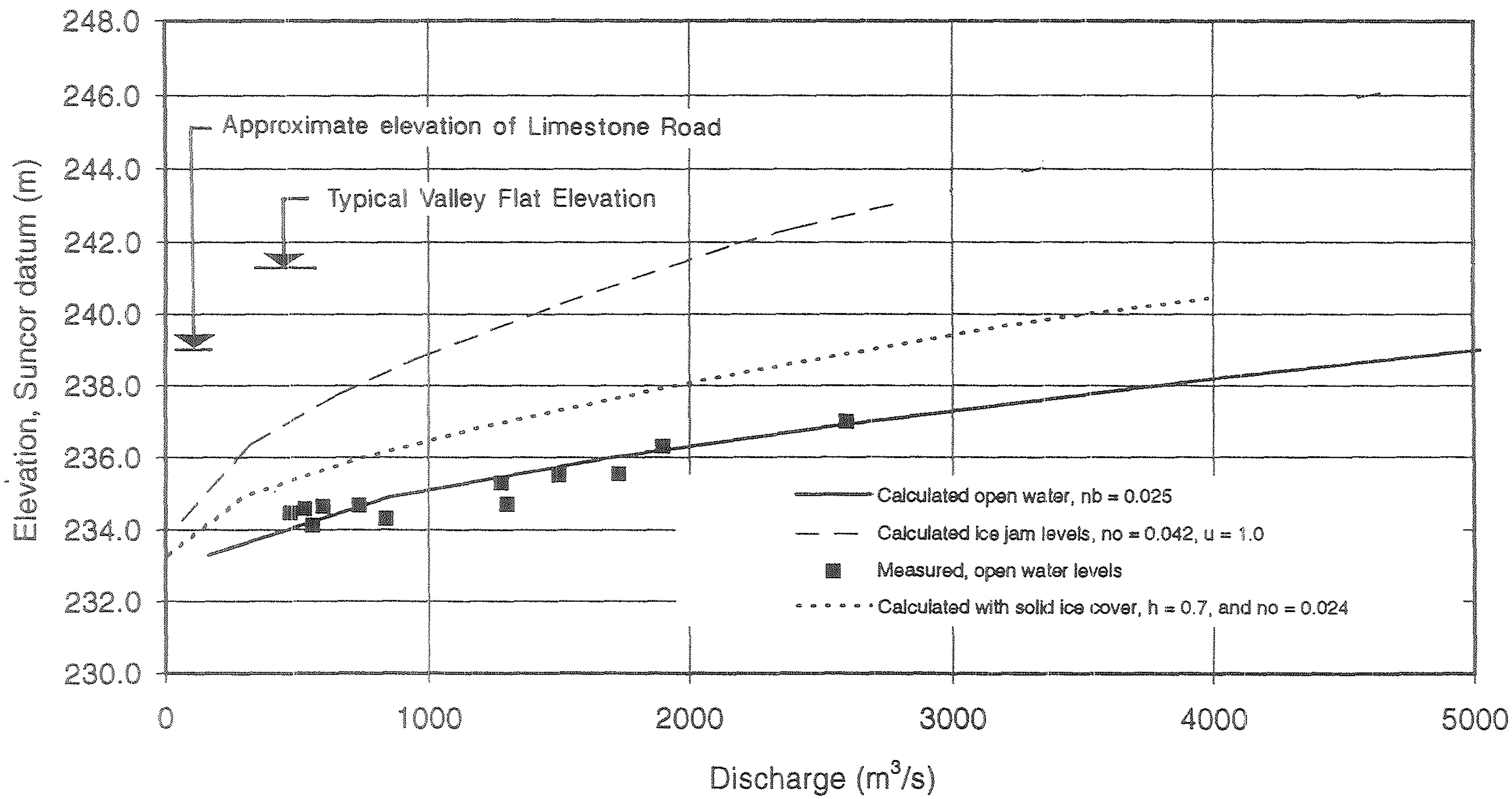


Figure 4.3.

gauge upstream), and the ice jam rating curve established for the site using the equilibrium jam approximation and the coefficients derived from the 1979 jam located just upstream of Suncor. It should be noted that the uniform flow stability analyses produces results that are identical to those from the **RIVJAM** analyses for the "No Constriction" case shown in Table 4.6.

TABLE 4.4
Reach-Average Hydraulic Characteristics at the Proposed Bridge Site

Reach[1]	261.34 km to 266.28 km		
Slope	0.00015		
Elevation[2] **(m)**	**Top Width (m)**	**Area (m^2)**	**Mean Depth (m)**
233.3	380	350	0.92
234.5[3]	470	850	1.8
235.0	480	1090	2.3
236.0	485	1580	3.3
238.0	510	2590	5.1
239.5	530	3370	6.4
240.0[4]	535	3630	6.8

1 The proposed bridge site is located at km 262.25, MacEwan Bridge is at km 294.9.
2 The elevation is referenced to Suncor Datum.
3 Average pre-breakup water level.
4 Top of valley flat at water intake.

<u>Ice Jam Frequencies</u>
As was mentioned earlier, there is some question about the appropriate frequencies of ice jams at the bridge site. The most conservative assumption is that frequencies are the same as at Fort McMurray. On the other hand, a case can be made to reduce the frequencies of the larger events (greater than about the 1 in 20 year jam) at the bridge by about two thirds on the basis of the ice jam observations. That is, only about 30 percent of the extremely severe ice jams that have been observed at Fort McMurray actually affect water levels at the bridge site. It would be appropriate, on the other hand, to maintain the same frequencies for the events with a return period of less than 20 years. Table 4.5 summarizes the ice jam frequencies derived by both approaches.

TABLE 4.5
Summary of Breakup Stage Frequencies and Representative Discharges at the Proposed Bridge Site

Exceedence Probability (%)	Return Period (years)	Representative Discharge (m^3/s)	Water Elevation (Suncor Datum - m) at the Proposed Bridge Site	
			Conservative Assumption	Observation Based Assumption
1	100	3160	243.6	242.0
2	50	2450	242.5	241.2
5	20	1530	240.2	240.2
10	10	1050	239.0	239.0
20	5	680	238.0	238.0
50	2	320	236.4	236.4

It is evident from Table 4.5 that bankfull stage of approximately 238.0 metres will be exceeded by an ice jam once every twenty years on the average. Furthermore, the 1 in 100 year event will exceed bankfull stage by about two to four meters, depending on the frequency distribution that is being used. However, it is possible to exceed the top of bank by about one third of the thickness of the jam without losing confinement. The presence of the bridge abutments may therefore increase the 1 in 100 year ice jam event by 0.5 to 2.0 metres by providing confinement beyond the existing bankfull stage up to, at least, the 100 year ice jam elevation. There should be no change in the ice jam elevations up to the 1:50 year event because of the bridge.

Observations of ice scars at the Fort Mackay bridge (located some 18 kilometres downstream of the proposed bridge site) suggest that the highest recorded ice level measured on October 23, 1974 is about 8 metres above the open water level. The Fort Mackay observation suggests a design elevation at the proposed bridge site of 242.0 metres. This is consistent with the observation-based reduction in the ice jam levels for the large return period events.

4.1.3 Cofferdams and Access Berms

Two ice-related issues that arise from the construction of the berm and the cofferdam to provide access to pour the footing and the shaft of the various piers are (1) the required height of the cofferdam to withstand overtopping due to an ice jam and (2) the possibility of a jam forming at the cofferdam site and scouring out the river bed around the base of the cofferdam.

Should the berm be in place during breakup, the magnitude of the constriction may increase the probability of jam formation because a thinner jam can be stable due to the decreased width of the section. However, the constricted section will be relatively short, and therefore the effect of the constriction may be minor and localized to the constricted section only. **RIVJAM** was run for a number of discharges and two different constriction ratios (reflective of the "Base Case" and the "Alternative Case") to determine the water levels which would result should an ice jam develop during construction. The results are shown in Table 4.6. Figure 4.4 shows the calculated ice jam profiles through the reach for the constriction scenarios for a 1 in 10 year discharge of 1050 m^3/s.

TABLE 4.6
Summary of Ice-jam Related Water Elevations with a Cofferdam in Place

Representative Discharge (m^3/s)	Calculated Water Elevation (Suncor Datum - m)			Return Period (years)
	No Constriction	Base Case	Alternative Case	
2250	241.40	241.41	241.44	100
1530	240.11	240.12	240.15	20
1050	239.03	239.04	239.06	10
320	236.37	236.38	236.39	2

It is evident from Table 4.6 that the constricted section does not have a large impact on the ice jam levels for the various carrier discharges. Depending on the overtopping risk one would be willing to assume, the top of the cofferdam could be placed at an elevation ranging from 236.4 to 241.4 metres (not including freeboard).

4.1.4 Scour at the Toe of the Ice Jams

The processes of scour at the toe of ice jams are not well understood, although they are of major concern when considering the design of structures in or under a river bed. The analyses conducted herein suggests that the toe of any jam that forms in this reach must ground to ensure stability of the toe. This grounding occurs under a solid sheet of ice and produces toe thicknesses in the order of 2.3 to 6.0 metres, depending on the discharge. Immediately after grounding, all the flow under the equilibrium portion of the jam is conveyed through the toe by seepage. However, given the loose sandy bed material, it is highly likely that some piping occurs and the seepage is augmented by flow under the toe, only a short time after grounding.

For design purposes, it is appropriate to assume that very little seepage occurs through the toe and that most of the flow occurs under the toe. Elementary analyses (similar to that carried out for flow through a submerged sluice gate) of both energy and momentum conservation around the toe suggest that a stable condition will occur with only about 0.2 to 0.5 metres of flow under the toe if it is assumed that no energy loss occurs upstream of the toe. At this flow depth, and with the typical discharges and bed material in the river, substantial scour would occur and the flow area would increase. In addition, the assumption

Calculated Ice Jam Profiles at Bridge Site - 1:10 Year Return Period

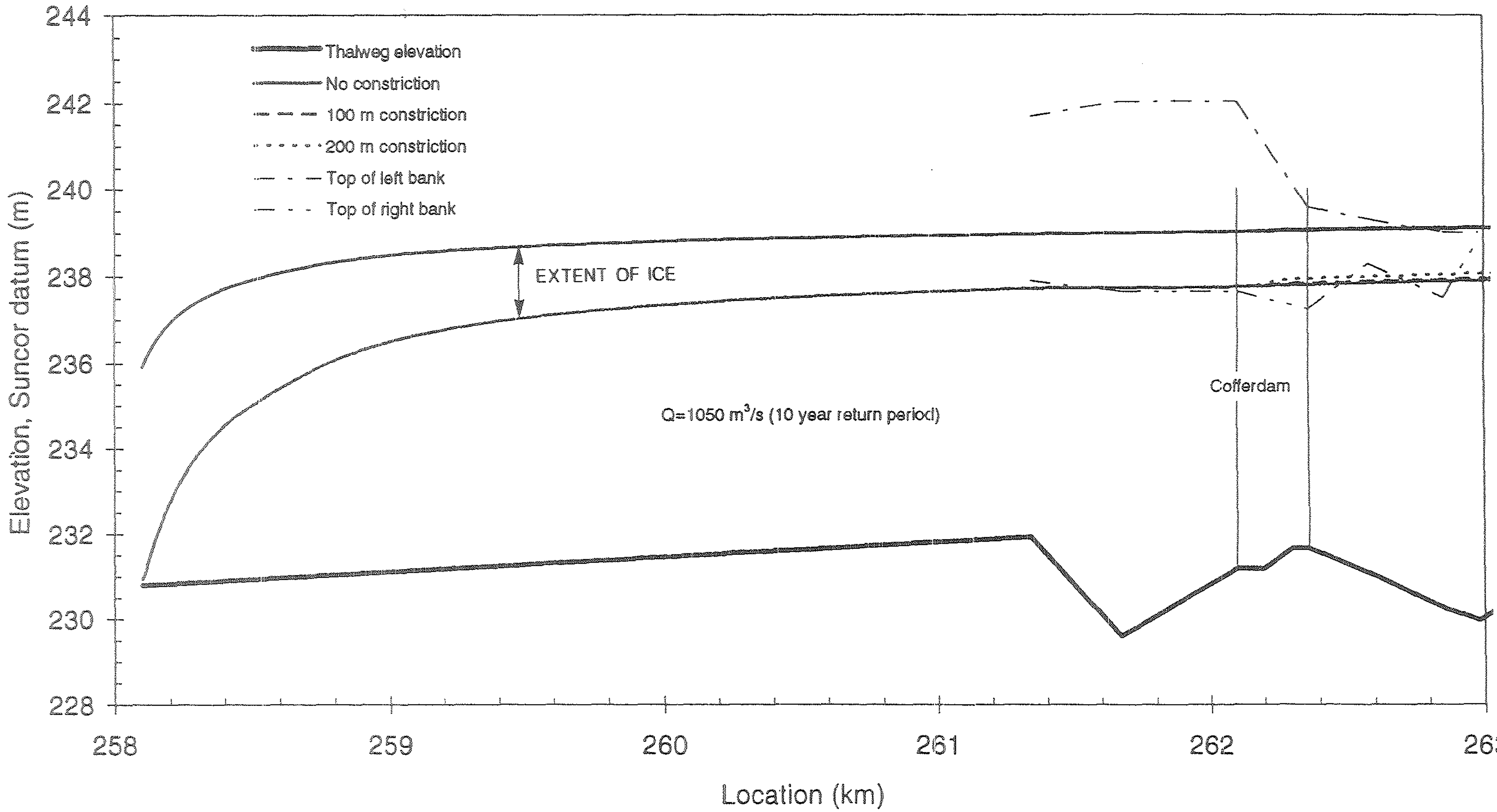


Figure 4.4.

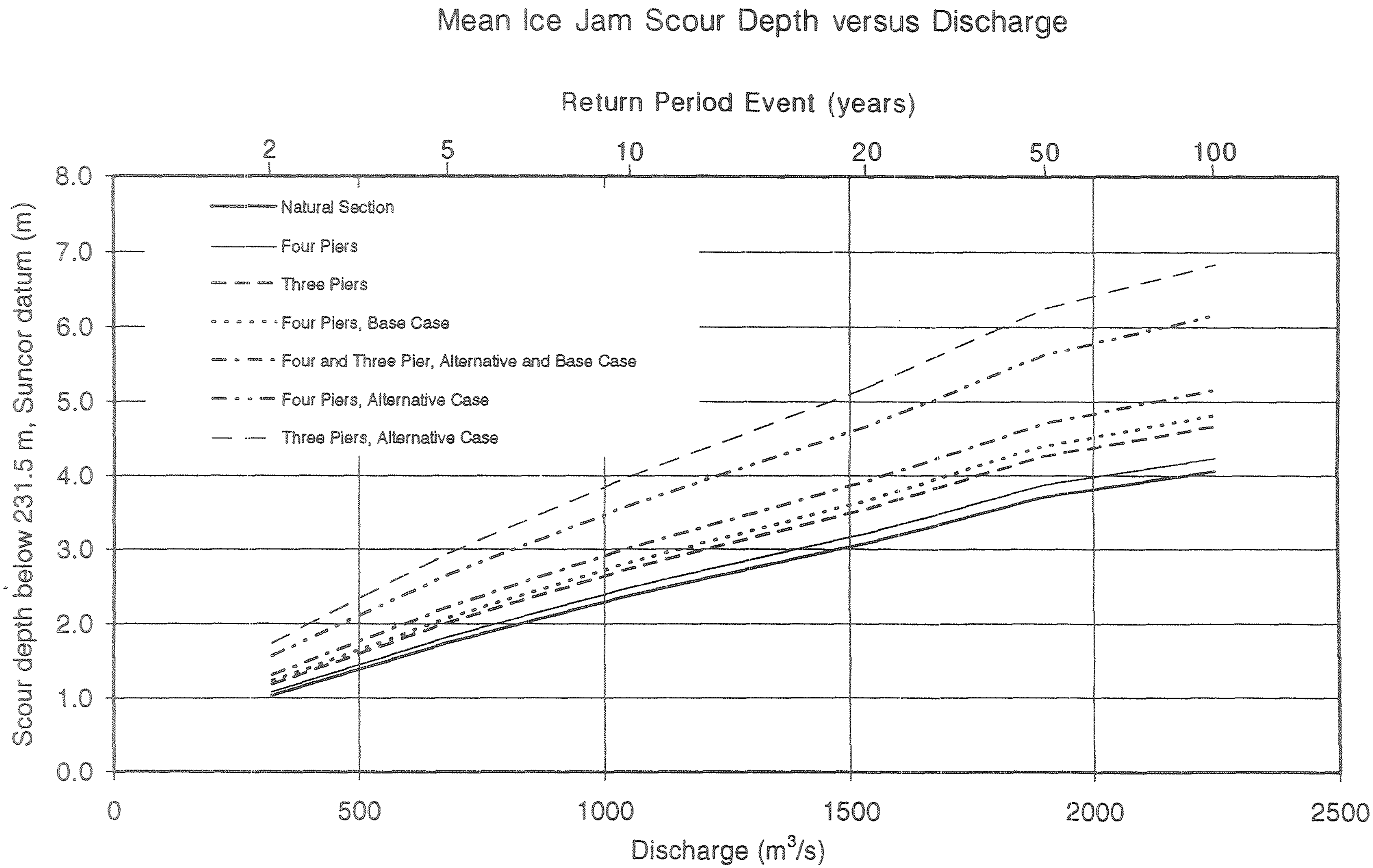
that there is no energy loss upstream of the grounded toe violates the notion of the nonuniform calculation of a stable ice jam profile in **RIVJAM**. Thus, it is most likely that the maximum scour depth under the toe of the jam would have to be great enough to ensure the passage of all the flow under the toe without being able to utilize the upstream head as defined by the equilibrium jam stage.

The upshot of this argument is that the mean scour depth under the toe of the jam would be equivalent to the flow depth under the ice cover downstream of the toe. The maximum scour depth would be some factor times the mean depth, depending on the shape and bottom width of the cross section at the toe. During this condition it would not be necessary to make allowances for additional scour around the piers. Figure 4.5 illustrates the mean scour depth in the bridge waterway for a variety of flows and constrictions that are representative of a number of possible options for constructing the bridge piers. Table 4.7 summarizes the expected design scour elevations for elevations below the toe of an ice jam at the bridge site.

The maximum scour depth shown in Table 4.7 was taken as 1.5 times the mean scour depth illustrated in Figure 4.5. Given the fine sand (D_{50} = 0.30 mm) and the existing rectangular-shaped channel, this value is appropriate, although other designers have traditionally chosen a higher value, say 2.0. Furthermore, by assuming that little flow was being diverted by seepage through the jam, the value of 1.5 in its own right is somewhat conservative. It is recommended that elevation 225 metres be adopted for scour consideration under the toe of the jam for the four pier bridge option. It should be noted that the scour elevation was determined with respect to a mean bed elevation of 231.5 metres at the proposed bridge crossing.

TABLE 4.7
Scour Depths Below the Toe of an Ice Jam at the Bridge Site for Various Bridge and Construction Scenarios

Return Period (years)	Scour Elevations in Bridge Waterway (Suncor Datum - m)					
	Four-Pier Bridge			Three-Pier Bridge		
	Bridge (No cofferdam)	Base Case (Cofferdam)	Alternative Case (Cofferdam)	Bridge (No cofferdam)	Base Case (Cofferdam)	Alternative Case (Cofferdam)
100	225.1	224.2	222.3	224.5	223.8	221.3
50	225.7	224.9	223.1	225.1	224.4	222.1
20	226.7	226.0	224.5	226.2	225.7	223.7
10	227.8	227.3	226.1	227.4	227.0	225.5
5	228.8	228.4	227.5	228.5	228.2	227.1
2	229.9	229.7	229.2	229.7	229.5	228.9



Note: Apply a scour factor of 1.5 times the mean scour depths above to estimate maximum local scour.

Figure 4.5.

For the condition with a cofferdam in place, the probability of a toe developing exactly at the cofferdam for only one year, is low. Therefore it is recommended that a lower return period, than that used for the bridge itself, be used to estimate the design scour.

There is some question about the ability of the scour hole below the toe to fully develop during the time that an ice jam may be in place. This becomes a question of the continuity of sediment transport. Given the high stage and low velocity upstream of the toe of the jam, one can assume that very little sediment will be transported into the developing scour hole. There is no doubt that the velocity under the toe of the jam is sufficient to scour the bed, therefore the critical issue is whether or not the sediment transport rate downstream of the jam is sufficient to transport the scoured material, in addition to that which it will transport locally from the bed. Table 4.8 summarizes the typical sediment transport rates that are required for the development of the scour below the toe and that would be for evident for the uniform flow condition downstream. From this table it is evident that the required transport rate to develop fully the expected scour is about an order of magnitude less than the transport rate downstream of the jam. For the entire range of flows that might be expected, there would be sufficient transport capability downstream of the jam toe to allow the scour hole to develop fully even over a time period as short as three days during which the jam might be in place.

TABLE 4.8
Comparison of Sediment Transport Rates for a Range of Flow Conditions for the Three-Pier Option

Discharge (m^3/s)	Characteristics of Scour Underneath the Toe of the Jam		Required Average Transport Rate (m^3/s) for Jams of Given Duration		Downstream Sediment Transport Rate[2] (m^3/s)
	Mean Depth (m)	Volume[1] (m^3)	3 days	7 days	
500	1.5	3100	0.012	0.0052	0.077
1000	2.6	9400	0.036	0.016	0.23
1500	3.5	17000	0.066	0.028	0.31
2000	4.4	27000	0.11	0.044	0.46

1 Assumes a bottom width of 370 metres and longitudinal scour hole slopes of 2.5:1 and 5:1 for the upstream and downstream ends of the scour hole, respectively.

2 Calculated from Colby's relationships.

4.1.5 Effect of Bridge and Cofferdams on the Initiation of Jams

It is expected that the bridge will not increase the likelihood of ice jamming in the reach. The proposed bridge is located downstream of a constricted section adjacent to the Tar Island Dyke. The typical channel width downstream of the bridge is 400 metres. The channel width along Tar Island Dike is about 350 metres. This is a constriction of about 12 percent, producing a channel width that is 88 percent of the channel downstream. The proposed four-pier bridge, which will be located immediately downstream of the constriction, will have an opening of about 370 metres if the width of the piers are not included. This is 92 percent of the channel width downstream and 106 percent of the channel width upstream. The three-pier bridge will have a width of 340 metres. This is 97 percent and 85 percent of the upstream width and downstream width respectively.

During the construction phase for the "Base Case", the cofferdam and access dike will further reduce the opening by some 100 metres, for both the four-pier and three-pier bridges, thereby producing a cross section width of about 86 percent and 68 percent of the width of the upstream channel for the two bridge options, respectively. These changes in width are not significant enough to enhance the initiation of jams. The "Alternate Case" will result in constrictions of 57 percent and 48 percent, respectively for the two bridge options. This may be enough to initiate an ice jam, therefore it is recommended that this amount of constriction not be in place during the breakup period.

4.1.6 Abutment Configuration for the Three-Pier Bridge

One of the options identified in the design of the three-pier bridge is vertical abutments. Vertical abutments would significantly increase the ice loads on the abutment. From the point of view of dynamic ice loads, a vertical edge that is protruding into the channel (as opposed to a sloped abutment) would be affected by moving ice floes of about the same size and velocity as would a pier in the centre of the channel. Thus the abutment would have to be designed for loads similar to those experienced by the pier, except over a larger width, say about 10 metres.

With respect to ice sheets shoving against the abutment, the magnitude of the load on the abutment would be about the same as the load that a surge of ice exerts on the solid sheet downstream during an ice run. This is the only mechanism that could generate loads of any significant magnitude given the width of the channel at this location. Studies have suggested that the ice sheet at the moving ice front fails by buckling and the load is a function of the ice thickness to the power of 1.5 and the square root of the modulus of elasticity. For ice with a nominal ice thicknesses of about 1.0 metres and a typical modulus of elasticity of 175 MPa, the load on the abutment would be about 400 kN/m.

If the abutment was sloped, the force on the face would be reduced because the ice would ride up the slope and fail in bending. In this case the load would be a function of the angle of the abutment, the coefficient of friction, the strength and modulus of elasticity of the ice, and the ice thickness to the power 1.25. For abutment angles of 40 and 60 degrees, the horizontal force would be 60 and 180 kN/m, respectively. This is substantially lower than what a vertical abutment would experience. To minimize the exposure of the abutment to high ice loads, a sloped abutment is recommended.

The other issue related to the three-pier alternative is the location of the left abutment. There is no technology available to quantify the impact that the position of the left abutment would have on the ability to pass ice. However, from an examination of the upstream channel planform, and the natural position of the thalweg, it is recommended that the reduced length of the bridge be achieved by moving the right abutment out and maintaining the left abutment as close as possible to the left bank. This is discussed further in Section 5.3 and 7.0.

4.2 ICE FORCES

4.2.1 Design Ice Thicknesses

Measurements of late winter ice thicknesses are available from a number of sources, including at the WSC gauge in the study reach, miscellaneous measurements undertaken by the City of Fort McMurray in the vicinity of the mouth of the Clearwater River, observations of ice characteristics within the jams by Andres and Doyle (1984), and from ice thickness measurements made during dye tests in the area (Van Der Vinne and Andres, 1992). The WSC measurements are difficult to interpret because they measure the total ice thickness, including frazil. This biases the ice thicknesses towards the high side. On the other hand, the WSC data probably underestimates the thickness of the ice that is produced in the rapid areas upstream of Fort McMurray. Table 4.9 summarizes the ice thicknesses measured in the area.

TABLE 4.9
Measured Late-Winter Ice Thicknesses on the Athabasca River Upstream of the Proposed Bridge Site

Location	Source	Years	Average Ice Thickness (m)			Comments
			Minimum	Mean	Maximum	
WSC Gauge at Fort McMurray	ARC	1962 to 1987	0.70	0.91[1]	1.4[2]	Ice formed in a mild-sloped channel
WSC Gauge at Athabasca	WSC	1954 to 1972	0.66	0.82	1.1	Upstream of study area, provides estimate of lower limit of ice thickness
Fort McMurray	ARC	1977 to 1979	-	-	1.5	Typical of ice from rapids upstream of Fort McMurray
Fort McMurray	AEP	1984 to 1986	0.6	1.0	1.5	Spatial average from area at confluence of the Clearwater River
Boiler Rapids	ARC	1992	0.41	0.5	0.67	Based on data from one section
Fort McMurray	ARC	1992	0.82	1.0	1.24	Based on data from one section

[1] For 1984 to 1987 only.
[2] Includes frazil deposits.

An analysis of the winter severity at Fort McMurray, based on the meteorological records for 52 years with complete records between 1920 and 1978 indicates that the average winter exhibits about 2500°C-days of freezing (Andres, 1991). The measured maximum °C-days of freezing is 3150 and the minimum measured °C-days of freezing is 1600.

Given a range of winter temperatures and winter snowfalls, a range of thicknesses can be calculated from simple energy considerations and ice growth models. Table 4.10 summarizes the computed ice thicknesses for a variety of conditions.

TABLE 4.10
Calculated Ice Thicknesses at Proposed Bridge Site

Winter Severity	°C-days of Freezing	Exceedance Probability (%)	Snowfall (%) of Normal[1]	Calculated Ice Thickness at End of March (m)
cold	2900	5	100	1.17
normal	2490	50	50	1.29
			100	1.11
			150	0.98
warm	1700	95	100	0.93

1 The normal snowfall is 112.4 cm from November to March, inclusive. The maximum monthly snowfall is in November and the minimum monthly snowfall is in March.

It is evident that the maximum calculated ice thicknesses are in the range of 1.2 to 1.3 metres. This is consistent with the observed ice thicknesses. It is recommended that a minimum ice thickness of 1.3 metres be adopted for design purposes. This is somewhat greater than the 1.1 metres ice thickness suggested by Northwest Hydraulic Consultants Ltd (NHCL) for the Fort Mackay bridge. However, given the proximity of the proposed bridge site to the thick ice that can be produced in the steep reaches upstream of Fort McMurray, the increased design ice thickness would be a prudent strategy.

The above recommendation is derived from estimates of maximum winter ice thicknesses. In most years, however, the ice thins before breakup so the actual ice thickness probabilities are somewhat less than this.

4.2.2 Design Ice Strengths

Breakup on the Athabasca River at the bridge site occurs at melting temperatures due to the breakup initiation processes upstream of Fort McMurray as described by Andres (1986). The river is unregulated and is not subject to rapid mid-winter snowmelt runoff so mid-winter breakup at below freezing ice temperatures are not expected. At breakup, the condition of the ice at the bridge site is variable, ranging from substantially disintegrated to internally sound ice. Ice from upstream of Ft. McMurray is also transported through the reach during breakup. This ice is also internally sound but the floes are typically smaller in size than those generated locally.

The CSA-S6-88 Bridge Code recommends that an ice strength of 1100 kPa be used for design if breakup occurs at melting temperatures but the ice moves in large pieces and is internally sound. This is similar to the maximum ice strength of 1200 kPa measured on the Athabasca River upstream at the Hwy 2 bridge crossing at Hondo but somewhat less than the 1 in 100 year return period value of 1500 kPa based on 19 years of record (Van Der Vinne, 1988). The frequency distribution of ice strengths at Hondo are given in Table 4.11. An ice strength of 1200 kPa is recommended for design. A reduction in ice strength for high ice levels is not recommended because the ice transported from upstream is still internally sound.

TABLE 4.11
Ice Strength Probabilities Derived from Hondo Measurements (Van Der Vinne, 1988)

Exceedance Probability (%)	Ice Strength at Hondo (kPa)
1%	1500
5%	1200
10%	1050
20%	900
50%	600

The effective ice pressure on the pier is obtained by multiplying the ice strength by an aspect ratio coefficient as defined in CSA-S6-88. The value of this coefficient is 2.1 for a 2.0 metres wide pier and an ice thickness of 1.3 metres. Thus, the effective pressure on the pier is 2500 kPa. This is slightly higher than the effective pressure of 2100 kPa used to design the Fort Mackay bridge (NHCL, 1978) using the CSA-74 Code.

4.2.3 Design Elevation of Ice Load

The elevation at which ice loads are applied to the piers varies from year to year as well as over the duration of each breakup. Breakup typically begins at water surface elevations of between 234 and 236 metres. The water level then rises rapidly if and when an ice run occurs. Maximum water levels are those associated with the development of ice jams. The ice is not moving in the jam but is moving immediately before the jam elevation is attained.

The 1 in 100 year return ice jam elevation was determined to be 242.0 metres. The ice load would be applied at an elevation of one-half of the ice thickness below the ice jam elevation or at 241.5 metres. This elevation is recommended as the high ice design elevation.

The ice load may be applied at elevations ranging from 234 metres to 241.5 metres. Thus if pier nose protection is required it should extend from about 233 metres to 242 metres. Protection may be required at a higher elevation if an inclined nose is selected because the ice will ride up the nose.

4.2.4 Design Loads

Dynamic Loads

Typically, the largest ice loads on bridge piers are generated by moving ice floes crushing against the pier nose. This load is a product of the ice thickness, the contact pressure, and the pier width. The longitudinal design ice load on a 2.0 metres wide bridge pier is 6500 kN as calculated using the CSA-S6-88 design code assuming that the piers are aligned with the direction of flow. This load would increase with increasing pier width but the increase would be offset somewhat by a reduced aspect ratio coefficient.

The longitudinal load increases significantly with increased skewness of the pier relative to the flow direction. The CSA-S6-88 design code recommends that the projected area of the pier perpendicular to the direction of flow be used to determine the design load and that this load then be resolved into its longitudinal and transverse components. The increase in longitudinal load with skew angle is given in Table 4.12. These values are based on a 10 metres long pier with round ends of 1 metre radius. A skew of as little as 10 degrees produces an increase in longitudinal load of about 50 percent thus it is beneficial that the piers be aligned parallel to the direction of flow.

TABLE 4.12
Variation in Dynamic Ice Loads with Skew Angle of Piers

Angle from Pier Axis (degrees)	Projected Width (m)	Ratio of Skewed Load to Straight Load	Ratio of Longitudinal Load to Straight Load	Ratio of Transverse Load to Straight Load[1]
0	2.00	1.00	1.00	0.15
2	2.35	1.11	1.11	0.22
4	2.70	1.21	1.21	0.24
6	3.05	1.31	1.30	0.26
8	3.39	1.40	1.39	0.28
10	3.74	1.50	1.48	0.30
12	4.08	1.59	1.56	0.33
14	4.42	1.68	1.63	0.41
16	4.76	1.77	1.71	0.49
18	5.09	1.86	1.77	0.58

[1] Minimum transverse load is 20 percent of total load for skewed piers.

The above design load is based on a 1 in 20 year ice strength, a 1 in 20 year ice thickness and is applied at a 1 in 100 year elevation. In comparison, the bridge at Fort Mackay was designed using the 1 in 2 year ice thickness, the 1 in 10 year ice strength and the 1 in 100 year elevation.

It is suggested that the design ice characteristics producing the lower combined risk factor be used for the proposed bridge since the costs associated with the loss of this bridge are much greater than those at the Fort Mackay bridge.

Impact Load Reduction Factor

In some cases, an impact load reduction factor can be applied to the design load especially for the smaller floes associated with the highest ice levels. The CSA-S6-88 design code allows for a reduction in design load if the kinetic energy of the flows is limited. The impact load reduction factor was found to be a function of floe area and velocity, ice strength and pier width (Van Der Vinne, 1989).

Typical floe velocities observed at Ft. McMurray are about 2.0 m/s but floe velocities greater than 5 m/s have been observed. Assuming a round pier nose, any floes greater than 15 metres in diameter will crush over the full width of the pier at a velocity of 5 m/s, while floes 35 metres in diameter will produce crushing loads at a velocity of 2 m/s. The range of large floe diameters measured at the Hondo and Pembridge test sites given in Table 4.13 indicate that diameters of 10 percent of the river width are not uncommon in an ice run. Floes larger than 35 metres in diameter are likely to occur at the proposed bridge site, therefore, it is recommended that no impact load reduction be applied.

TABLE 4.13
Distribution of the Diameters of Large Ice Floes in an Ice Run

Ratio of Floe Diameter to River Width	Percent Exceedance (Hondo and Pembridge data) (%)	Equivalent Floe Diameter at Suncor (m)
0.01	100	3.5
0.05	97	17.5
0.10	64	35
0.50	7	175
1.00	0	350

Bending Load Reduction Factor

Longitudinal loads may also be reduced due to bending failure on an inclined pier nose. However, for the present geometry, a nose inclination of 65 degrees is required before any reduction is allowed based on the provisions in the CSA-S6-88 design code. However, substantial reductions can be realized at nose inclinations of 60 degrees or 55 degrees as indicated in Table 4.14.

TABLE 4.14
Variation of Bending Failure Loads with Angle of Nose Inclination

Angle of Nose Inclination from Horizontal (degrees)	Bending Failure Load (kN)
70	6500[1]
65	5800
60	3800
55	2800

[1] Upper limit based on crushing load on pier width of 2.0 metres.

Transverse Loads
Transverse loads may be generated by three different mechanisms: floes contacting only one side of the pier nose; piers skewed to the direction of flow; and thermal expansion.

Even when the piers are aligned with the direction of flow, some allowance must still be made for ice loads which are not applied along the long axis of the pier. The CSA-S6-88 Bridge Code recommends that a full longitudinal load and a transverse load of 15 percent of the longitudinal load be applied to the pier nose. Alternatively, a transverse load which is a function of horizontal nose angle as given in Table 4.15 should be applied to the pier nose along with one-half of the longitudinal load. The load configuration which gives the worst effect should be used for design.

If the piers are significantly skewed to the flow direction, the transverse loads can be much larger than those discussed in the previous paragraph. The transverse loads estimated from the projected areas of various pier angles are listed in Table 4.12. The minimum transverse load in this case should not be less than 20 percent of the total load. These loads increase significantly with skew angle.

It is also possible that a pier may have a solid ice cover on one side but no ice on the other. In this case, thermal expansion pressure will produce a transverse load on the pier at a elevation of about 235 metres. This elevation is obtained from the solid ice cover rating curve for a 1 percent winter flow exceedance of about 300 m^3/s (Van Der Vinne and Andres, 1992).

TABLE 4.15
Variation of Transverse Ice Load[1] with Nose Angle in Horizontal Plane as a Fraction of the Longitudinal Load

Nose Angle in Horizontal Plane (degrees)	Transverse Load as Percent of Longitudinal Load (%)
140	9
130	14
120	19
110	24
100[2]	29
90	36
80	43

1 To be applied with one-half the longitudinal load if it produces a greater load than the full longitudinal load with a transverse load of 15 percent of the longitudinal load.
2 Round nose is similar to 100 degree nose angle.

Both data on ice pressures and theoretical analysis of ice pressure processes are limited. Monfore (1949) measured ice pressures of 220 kN/m to 350 kN/m in a reservoir with steep sides. Ashton (1986) estimates typical thermal pressures of about 300 kPa for a rapid rise of temperature of 2.8°C/h from -40°C to 0°C in reservoirs. At a temperature increase of 1°C/h, Michel (1978) suggests that the ice strain rate will be about 1.4×10^{-8}/s and the ductile strength would be about 250 kPa at -10°C. The strain associated with this stress is about 3×10^{-5} or 3 mm in 100 metres. The elastic response of the pier foundations may release this stress initially but temperature increases of more than about 2°C will still produce a failure stress in the ice if the piers have the same order of stiffness as the instrumented Hondo pier. Thus, the piers should be designed to withstand thermal pressures of about 300 kPa.

The thermal forces generated by the ice, act on the piers which are in the main portion of the channel. This portion of the channel typically has the minimum ice thickness thus the design value for ice thickness associated with the thermal pressure can be taken as 0.9 metres. This is similar to the minimum late winter ice thickness. Assuming a 10 metres long pier, the maximum thermal force generated is expected to be about 2700 kN.

<u>Vertical Loads</u>
A vertical ice load on the pier may be generated due to ice adhesion to the pier combined with rapid water level fluctuations. The rapid rise in water level associated with the breakup ice run may produce this vertical load before the ice is broken away from the piers. An estimate of this uplift force obtained from the CSA-S6-88 Bridge Code is 13.2 kN/m of perimeter for 0.9 metres thick ice, or 300 kN for a pier perimeter of 25 metres.

The actual vertical ice load is proportional to the volume of ice attached to the pier. The ice sheet may crack as it bends, due to the uplift force of the rising water. However, if the ice strength is greater than about 400 kPa, a 0.9 metre thick sheet will likely not crack due to bending. In this case, the width of ice affecting a pier is one-half of the distance to the next pier or about 50 metres. The uplift force on the pier generated from a submerged ice sheet 0.9 metres thick by 50 metres wide over a 25 metres perimeter is about 900 kN.

<u>Ice Jam Loads</u>

The CSA-S6-88 Bridge Code recommends that an ice jam pressure of up to 10 kPa be used for pier design. From the RIVJAM analysis it was determined that the adjusted 1 in 100 year ice jam discharge of 2250 m^3/s would produce a maximum ice thickness of 6.0 metres if the toe of the jam was grounded at the bridge site. This would produce an ice jam load of 120 kN on a 2 metre wide pier which is insignificant relative to the longitudinal loads generated by moving ice.

<u>Summary</u>

Table 4.16 summarizes the various ice loads for the combination of ice characteristics recommended for the design of the proposed bridge piers.

TABLE 4.16
Summary of Ice Loads on the Bridge Piers

Load Type	Nose angle (°)		Magnitude (kN)	Elevation (m)
	Vertical	Horizontal		
Longitudinal[1]	90	-	6500	241.5
	60	-	3800	241.5
Transverse	-	100[2]	1100[3]	235.0
Thermal[4]	-	-	2700	241.5
Ice jam	-	-	120	237.0
Vertical[5]	-	-	900	235.0

[1] Refers to loads parallel to the direction of flow in the river.
[2] Equivalent to round pier nose.
[3] Assumes pier nose inclined at 60 degrees.
[4] Assumes a pier length of 10 metres.
[5] Assumes a pier circumference of 25 metres.

5.0 SCOUR PREDICTIONS

5.1 APPROACH

The recommended design criteria for scour protection of piers and bridge abutments (and related pier foundation designs and abutment protection) is the scour depth of the 1:100 year ice jam event and the 100 year open water flood event with safety factor, whichever is greater. Predicted ice jam scour is discussed in Section 4.1. Open water flood scour is discussed below. In view of the severe consequences of failure and the risk of a 1:100 year scour event being exceeded during the design life of the bridge, a check should also be conducted to ensure that no failure would occur in the event of a 1:500 year return period flood event without safety factor.

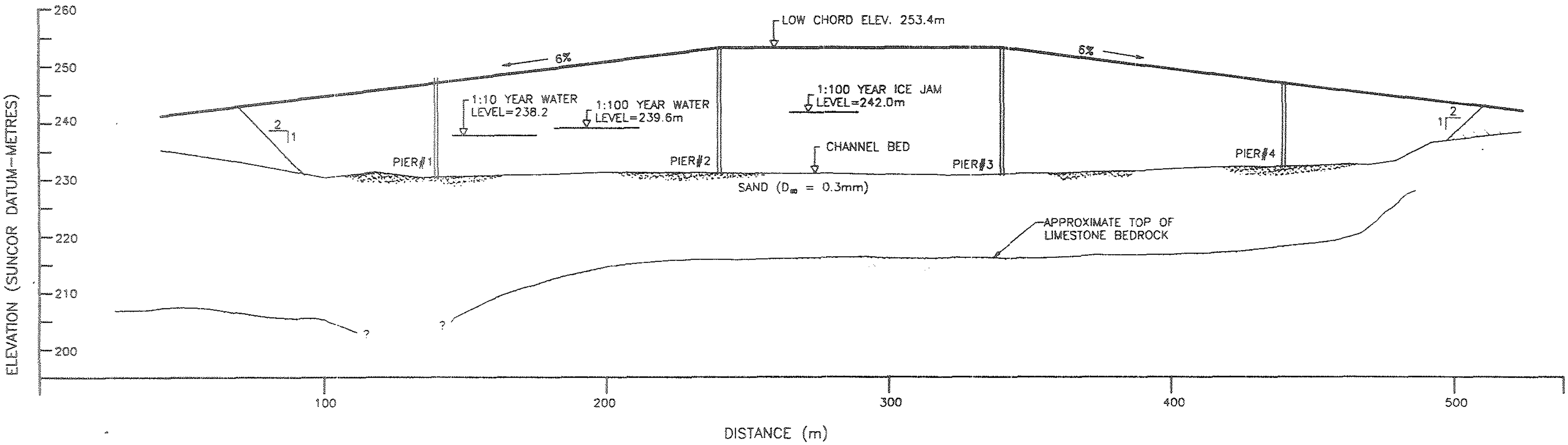
Cofferdam scour predictions, in Section 4.1 are based on ice conditions, and apply to the right side cofferdam. Cofferdam scour predictions for open water conditions, are given below. They apply to the left side cofferdam alternatives and to circular sheet pile cofferdams proposed for the middle piers.

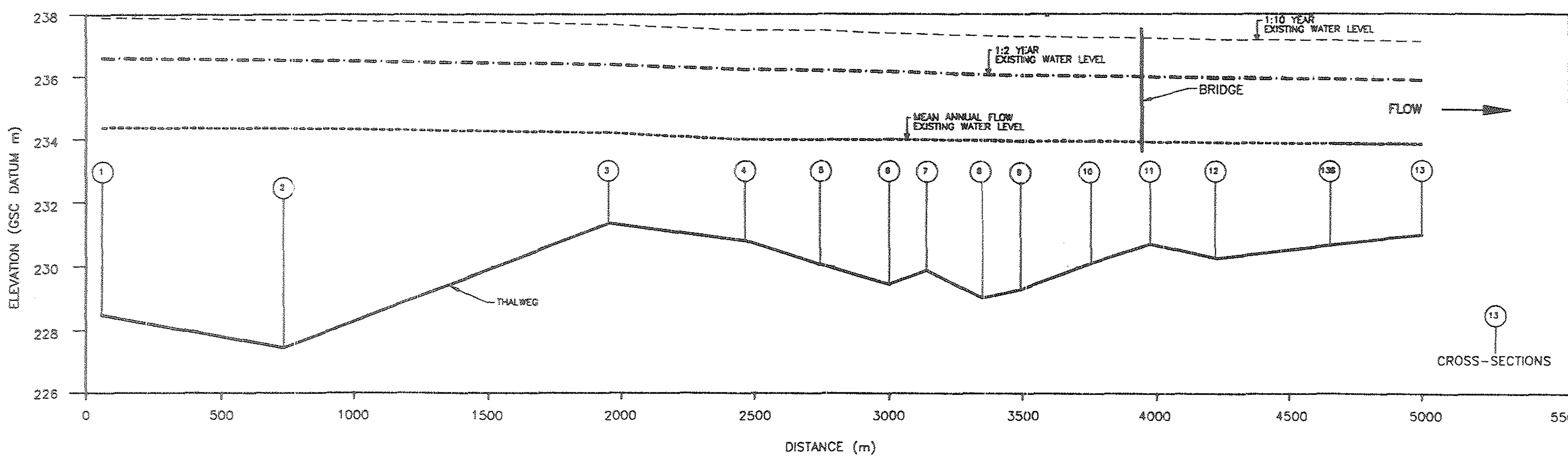
Open water scour is defined as the sum of general scour (which develops due to higher velocities of floods and constriction of flow), bedforms such as dunes, and local scour at piers and at the nose of cofferdams.

The magnitude of riverbed scour is based upon local hydraulic characteristics and the type of riverbed material. The riverbed material consists of uniformly graded fine sand based upon the available drillhole logs. Average bed material size (D_{50}) is 0.3 mm based upon sieve analysis data from the drill hole samples (Klohn Crippen, 1995). The logs suggest a poorly defined change in density and gradation at approximately 7 metres depth (elevation approximately 226 ± metres). However, the available sample data is insufficient to confirm any change in soil type with depth. Therefore, bed armouring at increasing scour depths cannot be assumed for scour computations.

The river cross section at the bridge and bed profile in the vicinity of the bridge is shown in Figure 5.1.

General scour is computed based on Blench's regime theory (Blench 1969) which relates scour to the constriction in flow created by the bridge abutments/piers and cofferdams. An allowance of one half the maximum expected height of the bedforms is added to the general scour to define a general scour envelope. Dune bedforms are expected to develop during flood conditions on the Athabasca River. Dune height has been estimated to be 1.2 metres at the 1:100 year return period flood (Allen, 1963). Because of the wide range of predictions and variety of conditions tested, local pier scour is predicted based upon review and application of a variety of empirical formulas given in the literature. The various relationships were mainly developed from laboratory tests supported with some field observations (Neill 1964, Larras 1963, Laursen and Toch 1956, Shen 1969 and 1971, Hanchu 1971, Breusers *et al* 1977, Jain and Fischer 1979, Melville and Sutherland 1988, U.S. Dept. of Transportation, 1993).





AGRA Earth & Environmental Limited	CLIENT: H.A. SIMONS LTD.	DATE: JAN 1996
	PROJECT: SUNCOR–ATHABASCA RIVER BRIDGE	JOB No. CW1466.00
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5.2 OPEN WATER COFFERDAM SCOUR

Predicted general open water scour for both the Base Case and Alternative cofferdam configurations are shown on Figure 5.2. Total scour is computed based upon the constriction in flow due to the cofferdams using the following formula:

$$y_s = Z \bullet y_1 \bullet \left(\frac{W_1}{W_2}\right)^{\chi} \qquad (1)$$

where:

y_s	=	scour depth at contraction
y_1	=	average depth at section upstream of contraction
W_1	=	width of upstream flow section
W_2	=	width of corresponding flow section at the constriction
Z	=	local scour multiplication factor
χ	=	exponent which may vary from 0.56 to 0.86 but is taken as 0.67 for a sand bed river in this situation

Due to the significant size of the cofferdams, the flow splits into separate channels. Therefore, the proportion of flow passing between each cofferdam was computed and potential scour was analyzed separately. As a result, a local scour multiplication factor of only 1.5 was applied which is more typical of local scour along a gradual transition as opposed to at the nose of a cofferdam. One-half the expected dune height is incorporated into the maximum potential scour predictions in Figure 5.2.

The results using the above approach are comparable to applying Blench's regime method with an equal scour multiplication factor (Z) of 1.5. By comparison, recommended scour multiplication factors for the nose of a spur (similar to that created by the earthfill cofferdams) typically range from 2.0 to 2.75 for the regime approach. Such high factors are not applied in this case because of the high sediment transport of the river and mobile bed conditions. This will cause the river to quickly adjust to the constriction by scour and deposition to allow gradual transitions to develop around the cofferdam. The resulting general scour, without armouring, is over 7 metres during a 1:20 year flood.

Due to the deep scour associated with flooding during bridge construction, the use of sheet piling around the nose of the earthfill cofferdam may be preferable to an armoured earthfill cofferdam as discussed in Section 6.1.

The Alternative cofferdam configuration results in slightly greater predicted scour caused by the flow constriction of the cofferdam. This cofferdam configuration forces the channel thalweg to move far over to the right side of the channel. Consequently, greater scour protection is needed for this alternative, as discussed later in Section 6.1.

PREDICTED TOTAL OPEN WATER SCOUR AT COFFERDAMS

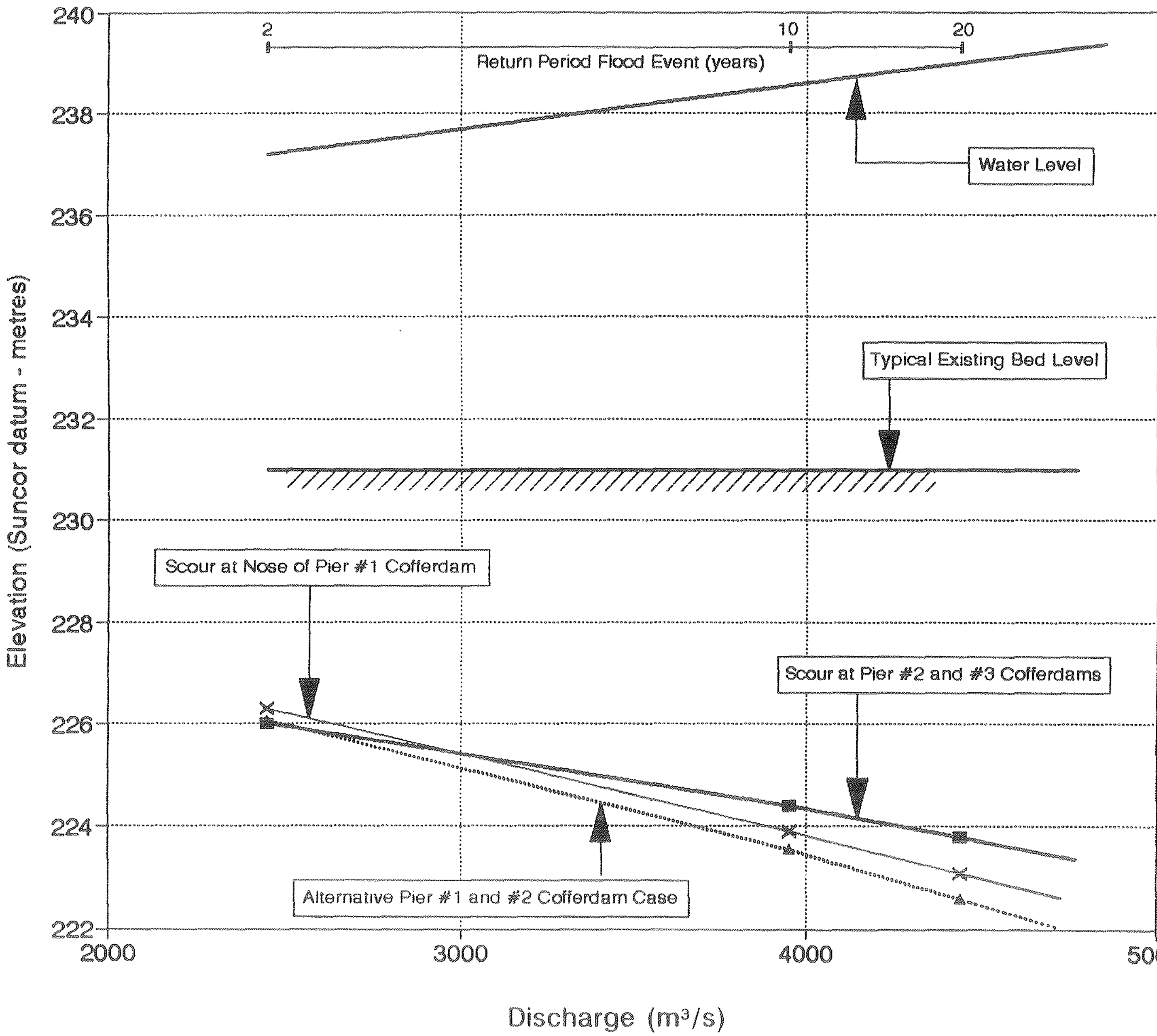


Figure 5.2.

5.3 OPEN WATER BRIDGE SCOUR

5.3.1 Four Pier Bridge Option

General Scour

General scour at the bridge section using the regime method with a scour multiplication factor (Z) of 1.5 is computed to reach elevation 226.6 during the 1:100 year flood. Subtracting 0.6 metres which represents one half the estimated dune bedform height, results in a general scour elevation of 226.0 metres. This elevation also coincides with the possible increase in soil density which is inferred by the drill hole logs.

Potential scour is expected to be less on the right (east) channel side where deposition and bar formation is presently occurring immediately downstream of the proposed bridge. The main channel thalweg can be expected to shift anywhere between the extreme left (west) bank and Pier 3. A general scour envelope is therefore a horizontal line at an elevation of 226.0 metres, from the toe of the left bank to Pier 3. The general scour envelope is then raised between Pier 3 and the toe of the right bank by following the trend of the existing bed slope at the bridge section. This results in an estimated general scour envelope elevation of 227.4 metres at Pier 4.

The computed general scour estimate is approximately 5 metres. The general scour is two metres deeper just upstream in the narrow confined reach of river between Sections 6 and 9. The scour Z factor of 1.5 is appropriate considering the slightly meandering flow conditions.

Pier Scour

Potential local pier scour estimates were based on a 2.5 metre wide (at the bottom) wedge-nosed pier. Local scour was estimated based on a variety of empirical methods recommended in the literature. Results vary widely from under 2 metres to 9 metres, however, most predictions range from 3.5 to 5.6 metres. The average local scour estimate is 4.6 metres assuming that there is no skew to the direction of the flow.

The proposed bridge alignment suggests the bridge piers, if aligned perpendicular to the axis of the bridge, would be about 8 degrees skewed to the direction of flow during flood conditions. The direction of flow through the bridge during flooding is estimated by drawing a line tangent from the upstream projecting east bank to the west side of the island just downstream of the bridge. Because of the shifting bed conditions observed on the Athabasca River in this reach, the actual local direction of flow at any pier may easily vary by ±5 degrees from this estimated flow direction. The impact of skew on local pier scour is illustrated below.

Pier Skew to Flow	Local Pier Scour (m)	Comments
0°	4.6	-
8°	6.7	Estimated present skew during flood with pier aligned perpendicular to bridge axis.
13°	7.9	Possible maximum skew.

Design Scour

According to Section 4.1.4, the 1:100 year ice jam could result in general scour to elevation 225.0 metres. This is slightly lower than the 1:100 year open water scour depth and is therefore the critical design condition. Local scour at piers and abutments is normally added to the general scour depth, unless local scour is controlled by structural erosion control.

The concrete pile cap can provide an effective means of limiting local pier scour if local scour does into extend beyond the edges of the pile cap. The present foundation plan incorporates a 14 metre wide by 22 metre long by 1.5 metre thick pile cap. Therefore, the pile cap would project horizontally 5.75 metres from the sides of the piers. The primary mechanism of local pier scour is downward horseshoe vortex currents. These effects typically do not extend out from the pier further than two times the pier width or 5 metres in this case (Jones *et al*, 1992 and Parola *et al*, 1996). The pile cap will therefore effectively control the depth of local pier scour on the sides provided there is no significant skew (ie., skew is less than 10 degrees).

The zone of greatest potential scour typically extends directly upstream of the pier from the direction of flow with deposition on the downstream or leeward side. Weaker wake vortex currents can also result in local scour on the downstream side of the pier. Sufficient cover for the pile cap should be provided to ensure it is not exposed under design scour conditions because any scour which develops to expose the edges of the pile cap will rapidly promote much deeper local scour. Therefore, the top of pile cap elevations could be set at 225.0 metres at all four piers and riprap protection should be provided around the front of the piers at the pile cap level. Details of this protection are discussed further in Section 6.2.

In the event piers 1, 2 or 3 are expected to be skewed more than 10 degrees to the direction of flow during flood, consideration should be given to lowering the top of pile cap elevation to 223.5 metres.

Extreme Flood Design Check

The general scour level during the 1:500 year open water flood of 7700 m^3/s would be at elevation 224.6 metres at piers 1, 2 and 3. With the top of pile cap set at elevation 225.0 metres, local pier scour would occur at the edges of the pile cap. Local pier scour theory predicts that the maximum potential scour may extend to bedrock (217 metres ±), because the top of pile cap becomes exposed by general scour. However, considering a possible increase in bed material density below elevation 226 metres and the potential for some form of natural bed armouring, the additional depth of local scour should not exceed

5 metres. Structural designers should ensure that the piers are stable in the event of general scour to an elevation of 220 metres, though the structural safety factor for this condition may be reduced. An alternative is to provide additional riprap protection around the entire pile cap at the pile cap level to ensure scour does not extend below the pile cap level in the vicinity of the pile cap. Details of this protection are discussed in Section 6.2.

Abutment Scour
If smooth armoured transition sections are provided at both abutments, the maximum potential scour should be as defined by the ice jam scour elevation of 225.0 metres.

5.3.2 Three Pier Bridge Option

General open water scour for the three pier bridge option was computed in the same manner as the four pier option. Because of the greater constriction, the resulting general scour level accounting for bedforms, is elevation 223.9 metres. This elevation is lower than the predicted ice jam scour and would therefore be the critical design scour level, excluding effects of local pier scour.

It is assumed that the Piers 1, 2 and 3 and the left abutment of the three pier scheme would be at the same location as Piers 1, 2 and 3 and the left abutment of the four pier scheme. However, the right abutment of the three pier option would be located at about the location of Pier 4 of the four pier option. The general scour level is therefore recommended to extend horizontally across the entire bridge section and should apply to all three pier locations as well as the abutments. If the locations of the bridge abutments change, the general scour elevation would change.

Local pier scour estimates would be the same as discussed for the four pier option. Riprap protection around the front of the pile cap similar to the four pier option would be required to protect against local pier scour.

Extreme Flood Design Check
The predicted general scour level during the estimated 1:500 year return period flood is elevation 222.2 metres for the three pier bridge option. If the top of pile cap elevation is constructed above this elevation, local scour could extend well below the pile cap similar to the four pier option. To prevent failure, the piers could be designed to be stable even with scour to bedrock, albeit with lesser safety factors. Alternatively, riprap protection could be provided around the pile cap as discussed in Section 6.2.

6.0 EROSION AND SCOUR PROTECTION

6.1 COFFERDAM PROTECTION

6.1.1 Base Case Cofferdam for Pier 1

The average channel velocity is 3.0 m/s and local velocities at the nose of the cofferdam can exceed 3.6 m/s during the 1:20 year flood event. The corresponding 1:20 year depth of flow, without scour, at the end of the cofferdam is approximately 8 metres.

On this basis, the minimum recommended armour protection of the earthfill cofferdam should consist of a 0.4 metre thick layer of cobbles (ie., 2 x D_{50}, where D_{50} = 200 mm) at a 3:1 slope. Rounded stone is assumed for this design, although angular rock is preferred. A riprap apron 0.8 metes thick should extend 13.4 metres from the toe of the cofferdam to allow for anticipated local scour to elevation 223.0 metres. This apron assumes that a 2:1 scoured slope develops with an allowance for 50 percent of the armour to be ineffective. A long, thin apron is assumed in order to promote scour and flow further away from the cofferdam. However, a shorter thicker apron with the same amount of rock could be used. In the event a reduction in scour protection is deemed appropriate, various apron lengths for the corresponding design flood events are indicated along with a typical layout illustration in Figure 6.1.

In view of the extensive armour protection required for this type of cofferdam, sheet piling around the nose might be considered as an alternative to control scour and seepage.

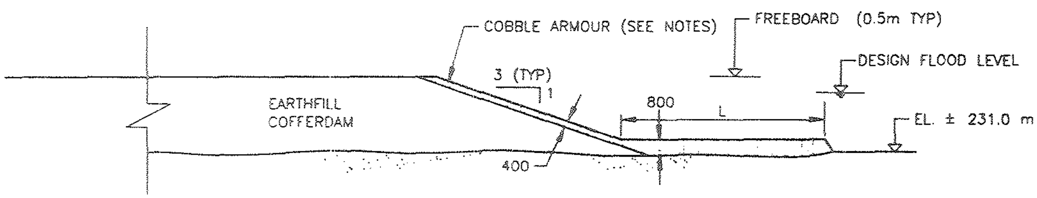
6.1.2 Alterative Cofferdam for Pier 1 and 2

This option is not recommended due to the 40 percent constriction of the channel (to a width of approximately 245 metres) with the resulting high velocities (> 4 m/s at the 1:20 year flood) and deep potential scour. Erosion protection requirements for this cofferdam are also illustrated in Figure 6.1. The same size of rock armour is shown as that proposed for the base case cofferdam. However, the factor of safety reduces from approximately 1.27 for the Base Case to 1.20 for the Alternative cofferdam.

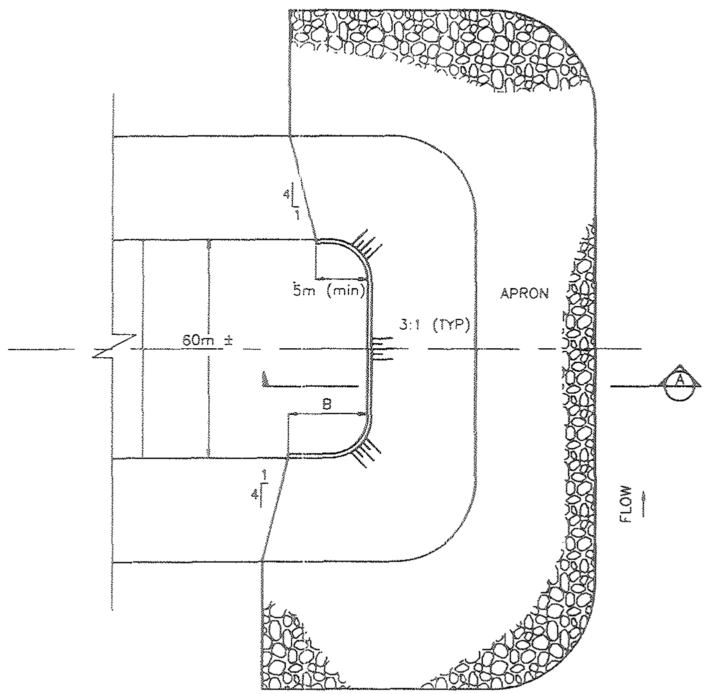
Because of the extensive constriction, flow might be expected to develop parallel along the upstream side of the cofferdam. Extending the armour protection for at least 30 metres around the upstream side of the Alternative cofferdam is therefore recommended, as indicated in Figure 6.1.

6.1.3 Winter Cofferdams

Winter cofferdam scour and erosion protection requirements, if exposed to breakup would be similar to those described above for the open water cofferdams. Predicted scour levels are specified in Section 4.1. Because of the high velocities during ice jam scour, use of larger Class I size riprap (D_{50} = 300 mm) would be required.



SECTION A–A



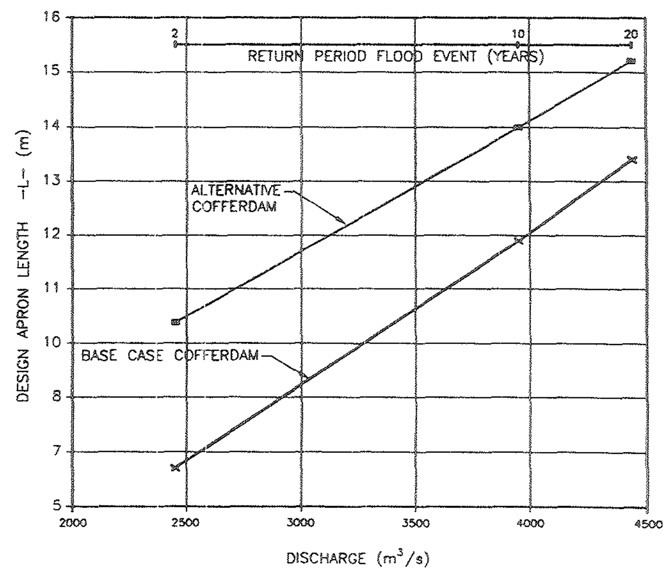
PLAN

NOTE:

1. **ROCK ARMOUR GRADATION**

PERCENT PASSING BY MASS	SIZE (mm)
100	300
30–80	250
20–50	200
5–20	100

SIZES QUOTED ARE EQUIVALENT SPHERICAL DIAMETERS.
SPECIFIC GRAVITY IS IN THE RANGE OF 2.4 – 2.9.



B = Minimium 15 metres for Base Case Cofferdam (See Figure 3.3)
B = Minimium 30 metres for Alternative Cofferdam (See Figure 3.4)

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	PROJECT: SUNCOR–ATHABASCA RIVER BRIDGE	JOB No. CW1466.00
	TYPICAL COFFERDAM ARMOUR PROTECTION WA[illegible] EA[illegible]	CAD FILE: CW\1466\1466–503
		61[illegible]

Flow velocities and local scour during ice conditions before breakup are expected to be minor. During this time (January to March) the median discharge is less than 200 m^3/s and the flow may exceed 300 m^3/s only about one percent of the time. Assuming the worst case flow condition of 300 m^3/s, the velocity in the constricted section will be 0.65 m/s for the Base Case and 0.82 m/s for the Alternative Case for the four pier bridge. For the three pier bridge, the velocity in the constricted section will be 0.72 m/s for the Base Case and 1.2 m/s for the Alternative Case.

For velocities of the magnitude indicated above, additional armour protection is not deemed to be necessary provided the fill material is of reasonable quality and properly compacted during construction. If this cannot be achieved, a cobble layer could be placed at the end of the berm to provide some degree of protection.

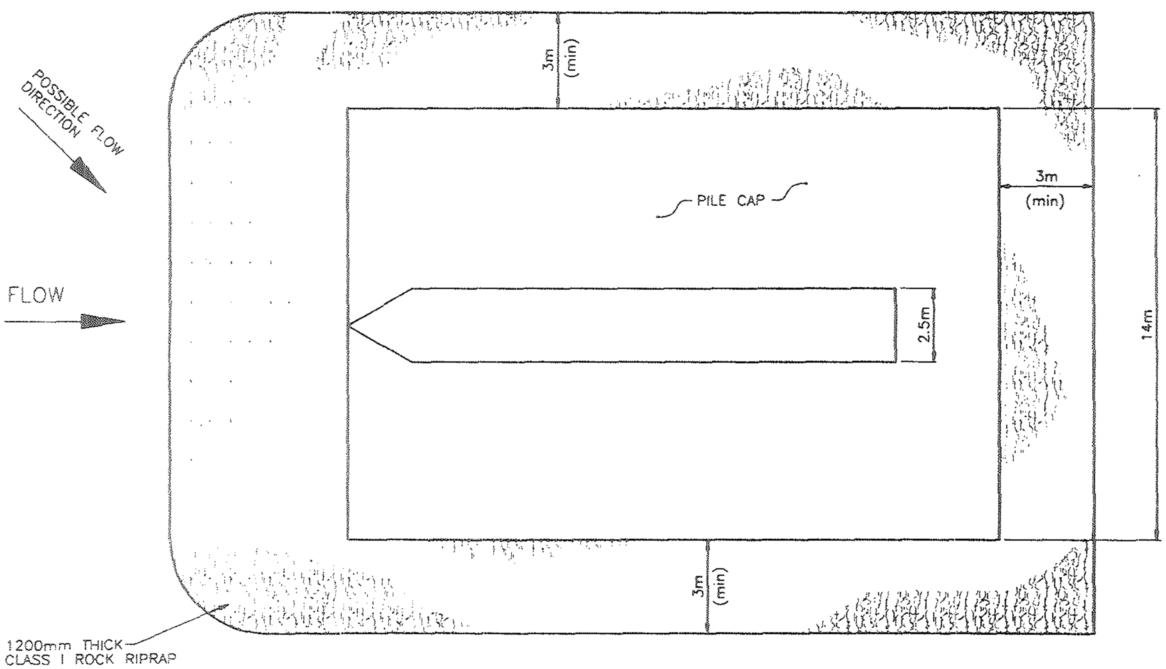
6.2 BRIDGE EROSION PROTECTION

6.2.1 Pier Protection

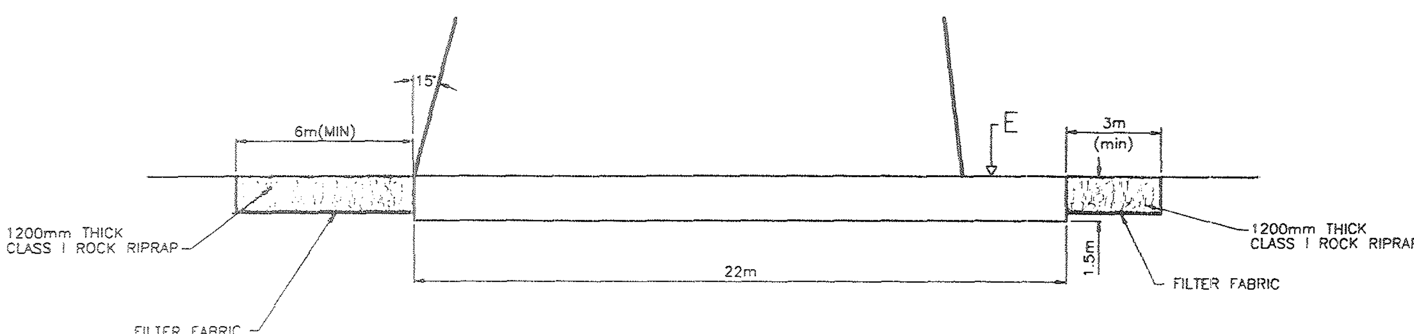
Pier scour protection is required for both the three and four pier bridge options. The top of pile cap elevations are 225.0 metres for the four pier bridge and 223.9 metres for the three pier bridge. These are the 1:100 year general scour levels.

For local pier scour a minimum of 6.0 metres of armour protection is deemed necessary at the front of the pile caps at the pile cap level provided the pile caps are set at or below the design general scour elevations. Riprap protection is also required as specified on Figure 6.2 to minimize local scour around the pile cap during flood events exceeding the 1:100 year flood. Structural designers should ensure that the piers and pile foundations are stable during a 1:500 year flood when general scour and local scour could reduce the lateral support around the pier by a further 5.0 metres below the top of the pile cap. These recommendations comply with or exceed the recommendations of various investigators (Dey *et al*, 1995; Breusers and Raudkivi, 1991; Bonasoundas (1973); and Neill (1973).

The minimum average (D_{50}) size of riprap should be 300 mm to provide adequate protection for the expected flow velocities. This size is based on the methodology by Chiew (1995) and is on the conservative side of minimum rock sizes calculated by other investigators. A minimum thickness of 1200 mm or four times the average rock size is recommended on all sides of the pier. The extent of riprap at the front of the pier provides excess material for launching. A granular filter layer beneath the riprap is not required with this thickness of riprap, however, the riprap should be placed on geotextile.



PLAN
N.T.S.



SECTION
N.T.S.

NOTE:

1. CLASS I ROCK RIPRAP GRADATION

PERCENT PASSING BY MASS	SIZE (mm)
100	450
30–80	350
20–50	300
5–20	200

SIZES QUOTED ARE EQUIVALENT SPHERICAL DIAMETERS. SPECIFIC GRAVITY IS IN THE RANGE OF 2.4 – 2.9.

2. ELEVATION E (m) :FOUR PIER BRIDGE =225.0
:THREE PIER BRIDGE =223.9

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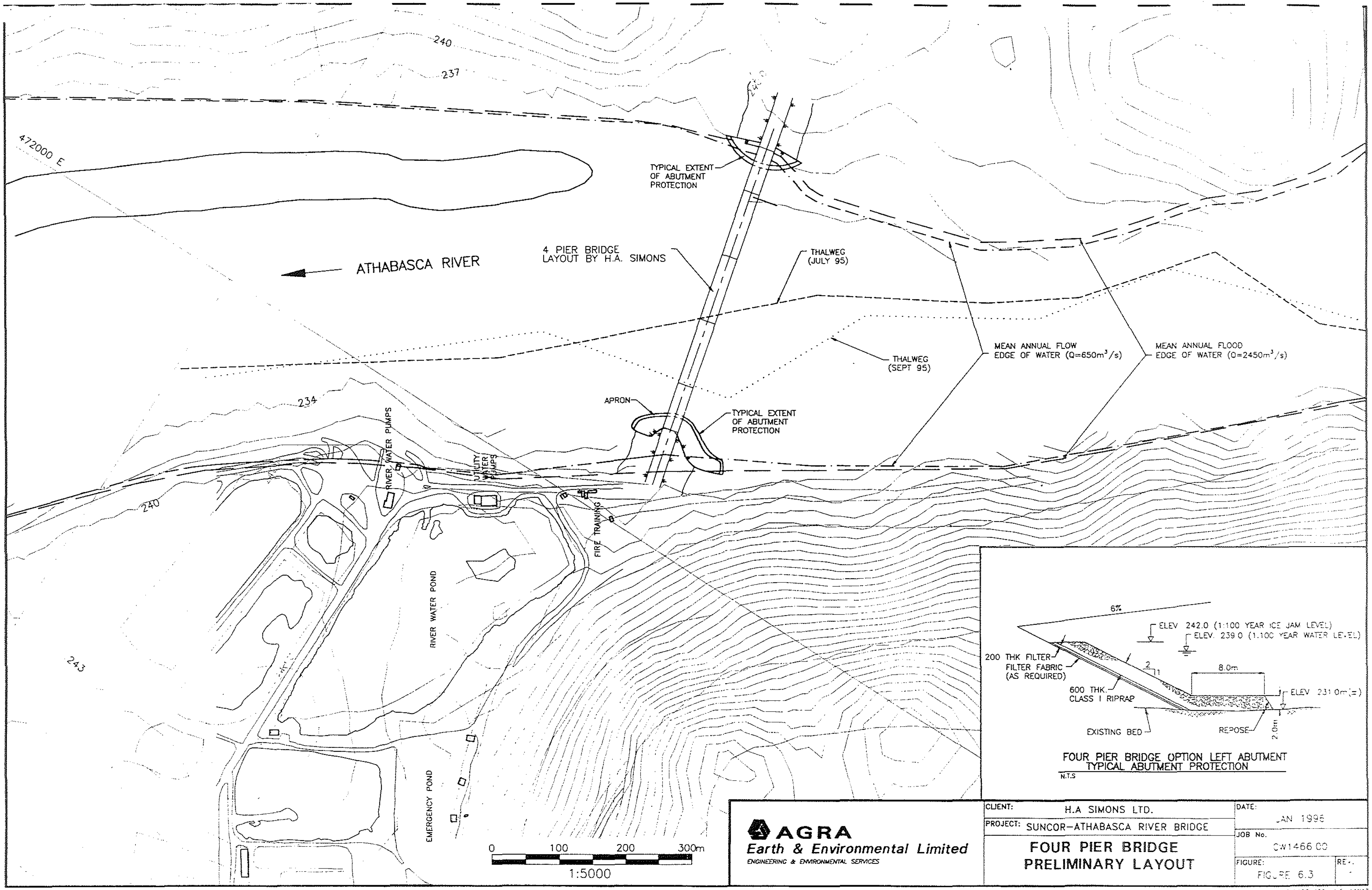
	PIER SCOUR RIPRAP PROTECTION
DATE:	JAN 1996
JOB No.	CW1466.00
CAD FILE:	CW\1466\1466-501

5.2



240
237
472000 E
ATHABASCA RIVER
TYPICAL EXTENT OF ABUTMENT PROTECTION
4 PIER BRIDGE LAYOUT BY H.A. SIMONS
THALWEG (JULY 95)
THALWEG (SEPT 95)
MEAN ANNUAL FLOW EDGE OF WATER (Q=650m³/s)
MEAN ANNUAL FLOOD EDGE OF WATER (Q=2450m³/s)
APRON
TYPICAL EXTENT OF ABUTMENT PROTECTION
234
RIVER WATER PUMPS
UTILITY WATER PUMPS
FIRE TRAINING
240
RIVER WATER POND
243
EMERGENCY POND
0 100 200 300m
1:5000
6%
ELEV 242.0 (1:100 YEAR ICE JAM LEVEL)
ELEV. 239.0 (1:100 YEAR WATER LEVEL)
200 THK FILTER
FILTER FABRIC (AS REQUIRED)
600 THK. CLASS I RIPRAP
2
1
8.0m
ELEV 231.0m(±)
EXISTING BED
REPOSE
2.0m
FOUR PIER BRIDGE OPTION LEFT ABUTMENT
TYPICAL ABUTMENT PROTECTION
N.T.S
AGRA
Earth & Environmental Limited
ENGINEERING & ENVIRONMENTAL SERVICES
CLIENT: H.A SIMONS LTD.
PROJECT: SUNCOR–ATHABASCA RIVER BRIDGE
FOUR PIER BRIDGE
PRELIMINARY LAYOUT
DATE: JAN 1996
JOB No. CW1466.00
FIGURE: FIGURE 6.3
REV.

6.2.2 Abutment Protection

Four Pier Bridge

Abutment protection during flood conditions is mainly required at the left bank because it projects into the deep section of the channel. The right bank is on the inside of a gradual bend and is in a zone of expected deposition. Abutment protection is therefore less critical on the right but is necessary to ensure bank stability following construction. A cobble armour, similar to that suggested for the cofferdams, would provide adequate protection during 1:100 year flood conditions. Mean channel velocities are expected to be only 2.1 m/s and maximum local velocities are expected to be about 2.7 m/s.

Larger Class I (D_{50} = 300 mm) riprap protection is required to provide protection against ice shove and potential high velocities which develop during ice jam scouring. Bank protection to the 1:100 year maximum ice level of 242.0 metres is proposed for both banks.

A rock apron is required to protect against scour to elevation 225 metres at both abutments because maximum ice jam scour could develop equally anywhere across the channel section. Bank protection should be composed of bank armouring above the existing river bed level and a riprap apron to provide bank erosion protection below the existing bed level. This configuration is shown on Figure 6.3. The riprap apron will launch during flood events to form a sloped bank armour below the existing bed level. To account for possible movement and loss of some rock during ice scouring, an efficiency factor of 2 is recommended for the rock apron. Standard practice where ice is not a major concern is to apply an efficiency factor of 1.5 (i.e. 50 percent of the riprap material is assumed to be ineffective on a fully developed scoured slope). On the left abutment, where the existing bed is at approximately 231.0 metres, a 2.0 metre thick by 8.0 metre wide apron is required. It should extend around the abutment and be keyed back into the upstream bank, as illustrated on Figure 6.3. A similar apron is required on the right abutment where the river bed is at elevation 233.0 metres. However, it should be excavated into the bed by 2.0 metres to elevation 231.0 metres to provide a similar level of scour protection as on the left side.

Provided the left bank abutment blends into the existing bank upstream of the bridge as suggested on Figure 6.3, further erosion protection measures along TID are not required as a result of the bridge. The left abutment may, in fact, cause local deposition on its immediate upstream side. Bank erosion protection is required along the currently eroding reach of bank adjacent to TID, however, this protection is required regardless of the bridge. A reduced level of armour protection is expected to be adequate for this eroding bank section, for current operations. A merging of the two levels of bank protection is recommended to occur approximately 150 metres upstream of the bridge.

Three Pier Bridge

Class I riprap protection similar to the four pier bridge option is recommended, however scour protection is required to elevation 223.9 metres. The apron length should be extended to 8.7 metres but a 2.0 metre apron thickness is adequate. A vertical abutment, would have to be designed for dynamic ice loads similar to those experienced by the piers, as discussed in Section 4.1.6. Gradual riprap transitions to the existing natural banks would also be required.

7.0 IMPACT OF BRIDGE ON RIVER FLOW CHARACTERISTICS

7.1 NAVIGATION

The navigation channel for the four pier bridge option would be between Pier 2 and Pier 3, as identified by the bridge profile shown in Figure 5.1. The navigation channel for the three pier bridge option would be between Piers 1 and 3. It is assumed that the three pier bridge profile would be horizontal from Pier 1 to Pier 3, providing two sections for navigation.

The results of the river surveys shown on Figure 5.1 indicate the river bed is moderately uniform from the left abutment to Pier 3 with less than 1.0 metre variation in bed level. The two surveys conducted in July and September of 1995 indicate that the channel thalweg, shown in plan in Figure 6.3, has shifted from near Pier 2 to near Pier 1 between July and September 1995. This amount of channel bed shifting is common on this reach of the Athabasca River. The annual peak flow of 3090 m^3/s, which occurred between the two surveys on August 18, 1995, was about a 1:5 year return period flood. The channel thalweg profile in September after this years peak flow was approximately 0.6 metres lower than the thalweg surveyed in July 1995. Scour areas were up to 1.5 metres lower than the July survey. Previous annual flood peaks from 1991 to 1994 have all been below average. The most recent significant flood was 4660 m^3/s in 1986. The absence of recent large floods may have caused infilling of the scour holes and stabilizing or entrenching of the thalweg.

The channel thalweg is expected to remain on the left side of the channel where it is presently located. A local deep thalweg section is aligned adjacent to the left abutment and is expected to remain. Local thalwegs adjacent to banks are typical for large sand bed rivers. The principal thalweg is expected to remain in the vicinity of Piers 1 and 2 as indicated by the recent surveys. The current horizontal bed between Pier 1 and Pier 3 is expected to remain, therefore the navigation channel section could be located either between Piers 1 and 2 or between Piers 2 and 3.

The preferred navigation channel would be between Pier 1 and Pier 2, because it is located on the left where the main channel flow is expected to remain. Attempting to train the thalweg to remain in the centre section, between Pier 2 and Pier 3, is not considered feasible or desirable. This would require a series of long low level spurs or instream flow deflectors projecting from the left bank. The upstream spur would need to be located over 400 metres upstream of the bridge in order to develop a uniform meander pattern. The main flow section would be restricted to approximately 350 metres in width similar to the upstream narrow section. In addition to the cost, concerns with such extensive river training would include the following:

(a) promote greater attack along the left bank at TID in the narrow section;

(b) increase the risk of ice jams and damage due to ice;

(c) result in extensive deposition in the vicinity of the Fresh Water Inlet possibly ending its service life;

(d) promote downstream channel bed changes and related erosion; and

(e) present an instream obstruction to boaters.

7.2 FRESH WATER INLET

The left abutment, as presently located, is expected to result in deposition both upstream and downstream of the bridge. A point bar would probably form at the armoured abutment. Based upon the natural meander wavelength pattern of the river, it can be expected that the zone of deposition may extend down as far as the Fresh Water Inlet area. However, the existing spur at the Fresh Water Inlet may still be effective in maintaining sufficient flow depth at the inlet.

The extent of sediment deposition in the vicinity of the inlet over time will depend upon the combination of a number of unpredictable but inter-related factors. These include the flow regime (sequence of flood events) shifting sand bars, any local debris impacts, and ice jam and breakup conditions. It is therefore reasonable to assume sedimentation will become more significant at the intake in the future, whether or not the bridge is in place. It is possible that the Fresh Water Inlet will not be significantly impacted by the proposed bridge. In the event deposition does have an excessive impact on the inlet, minor modifications to the existing inlet and spur could probably be implemented to reduce local sedimentation at the Fresh Water Inlet and improve conditions. Moving the west abutment of the bridge further west, as discussed below, would reduce the risk of sedimentation at the inlet.

The bridge will likely have an impact on sedimentation at the right bank. The orientation of Pier 4, if perpendicular to the present design axis of the bridge, and the right abutment will increase the rate of deposition along the right bank. It might be anticipated that the right side sub-channel just downstream of the bridge may eventually close off due to this deposition. This may well occur naturally but will be accelerated by the bridge. Deposition on the right bank may improve the stability of a deep channel in the vicinity of the Fresh Water Inlet.

7.3 BRIDGE ALIGNMENT AND LOCATION OF ABUTMENTS

The location of the left abutment in the deep water area of the Athabasca River increases the risk of scour and damage by ice floes. The risk could increase in the future because the river thalweg may entrench at the left bank. However, the right bank is located in a depositional area where the hydraulic and ice forces are less significant. Moving the bridge abutments and piers 50 to 75 metres westward would reduce the risks of excessive scour and ice damage.

Realignment of the bridge piers to minimize the skew angle would also improve the river flow characteristics and reduce the risk of excessive scour and ice forces. If possible, the bridge piers should be realigned by up to 8 degrees counterclockwise.

8.0 CONCLUSIONS AND RECOMMENDATIONS

1. Recommended bridge design parameters for the three and four pier bridge options are summarized in Table ES-6 below.

TABLE 8.1
Summary of Bridge Design Parameters

Item	Design Parameter	Unit	Four Pier Bridge	Three Pier Bridge
1	Mean Annual Water Level (655 m^3/s)	m	234.9	234.9
2	Mean Annual Flood Level (2450 m^3/s)	m	237.0	237.0
3	Mean Annual Ice Jam Level	m	236.4	236.4
4	1:10 Year Flood/Navigable Flood Level (3900 m^3/s)	m	238.2	238.2
5	Bridge Low Chord Elevation (15.2 m + Item 4) at highest span	m	253.4	253.4
6	1:100 Year Flood Level (5950 m^3/s)	m	239.6	239.7
7	1:100 Year Ice Jam Level/Top of Abutment Riprap Protection	m	242.0	242.0
8	Design Scour Level = Minimum top of foundation and abutment scour protection level (1)	m	225.0 (I)	223.9 (F)
9	Design Maximum Ice Thickness	m	1.3	1.3
10	Ice Strength	kPa	1200	1200

(1) Riprap protection around the pile cap foundation is also required to protect against local scour and extreme flood conditions
I = 1:100 year ice jam scour
F = 1:100 year flood scour

Dynamic ice loads for 1:20 year ice strength with a 1:20 year ice thickness applied at a 1:100 year elevation are summarized in Table ES-7.

TABLE 8.2
Ice Loads on the Bridge Piers for 20 Year Ice Strength, 20 Year Ice Thicknesses and 100 Year Ice Elevation

Load Type	Nose angle (°)		Magnitude	Elevation
	Vertical	Horizontal	(kN)	(m)
Longitudinal[1]	90	-	6500	241.5
	60	-	3800	241.5
Transverse	-	100[2]	1100[3]	241.5
Thermal[4]	-	-	2700	235.0
Ice jam	-	-	120	237.0
Vertical[5]	-	-	900	235.0

[1] Refers to load parallel to direction of flow in the river.
[2] Equivalent to round pier nose.
[3] Assumes pier nose inclined at 60 degrees, refer to Table 4.12 for effect of pier skew.
[4] Assumes a pier length of 10 metres.
[5] Assumes a pier circumference of 25 metres.

2. The bridge alignment and abutment locations for either the three or four pier bridge options do not result in excessive constrictions to flow in the Athabasca River. The maximum backwater affect is 17 cm due to the three pier bridge option during a 1:100 year flood. Similarly, either bridge option is not expected to increase the likelihood of ice jamming or bank erosion in this reach. Rapid drawdown rates of 0.5 m/h are usually associated with an ice run or a jam of very short duration. Rapid drawdown is therefore not a major concern affecting bank stability. Additional bank erosion protection along TID is therefore not required as a result of the proposed bridge.

3. The Alternate cofferdam configuration results in constrictions of over 40 percent of the channel width. The left side cofferdam would cause a large flow shift. The constriction caused by the right side cofferdam may be enough to initiate an ice jam. Therefore, this cofferdam should be removed before the breakup period.

4. The cofferdams are subject to extensive river bed scour because of the highly mobile sand bed river conditions in this reach of the Athabasca River. Extensive riprap aprons are required to accommodate this scour. Alternatively, the cofferdams could be protected by deep sheet piling.

5. An ice bridge is expected to be used during construction. The ice bridge should be broken up into ten or more units to reduce the risk of creating an ice jam during breakup.

6. The bridge alignment is estimated to be skewed 8 degrees from perpendicular to the direction of flow during floods. This is an upper limit of the tolerable skew for the design conditions discussed in this report because actual local skew on some piers may be even greater during a flood. If possible, the bridge pier alignment should be turned counterclockwise by up to 8 degrees to minimize the risk of scour and to reduce ice loading.

7. The present location of the left abutment projects into the river from the existing bank by about 75 metres. Moving the abutment further into the river is not recommended because the main flow of the river is presently aligned along the left bank side of the channel. Moving the four pier bridge configuration 50 to 75 metres westward is recommended because this would reduce the risk of excessive scour, reduce the risk of ice damage and reduce the risk of sedimentation at the Fresh Water Intake.

   If the three pier bridge option is selected, the left abutment and Piers 1, 2 and 3 should be in the same locations as piers 1, 2 and 3 of the four pier bridge option. The right abutment of the three pier option should be located at about the location of Pier 4 of the four pier bridge option.

8. The hydraulic investigations of the three pier bridge option are based on vertical abutments. These abutments would have to be designed for dynamic ice loads that are similar to those experienced by the piers, except applied to a larger width of about 10 metres. To minimize the exposure of the abutment to high ice loads, a sloped abutment could be constructed. A slope of 45 degrees or steeper would have a minor impact on the hydraulic results presented in this report.

9. Historical ice jams observed in the reach upstream of the bridge site can be expected to occur in the vicinity of the bridge. As a result, ice jam levels are higher than open water flood levels for similar return periods.

10. Protection against local pier scour at the piers is provided by the foundation pile cap which extends nearly 6 metres on both sides of the piers. This protection for local scour should be supplemented by riprap protection around the pile cap at a level equal to the top of the pile cap. The riprap protection should be sufficient to provide protection in the event of an extreme flood, such as the 1:500 year flood.

11. Bed levels across the channel between piers 1 and 3 are expected to remain relatively uniform. Either of these sections could serve as the navigation channel section. River training works should not be used to control the location of the thalweg in the centre bridge section.

12. The proposed bridge will cause some deposition along the left bank. This may extend downstream as far as Suncor's Fresh Water Inlet. The potential impact of such sedimentation is expected to be minor. If deposition does become a major problem, it may be possible to modify the existing inlet to remedy the situation.

13. All recommendations in this report are based on the bridge alignment and configuration provided by H. A. Simons. If any changes occur to the bridge alignment or configuration, including piers and abutments, AEE and Trillium Engineering and Hydrographics Inc. should be advised of these changes and given an opportunity to revise these recommendations.

Respectfully submitted,

AGRA Earth & Environmental Limited

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Reviewed by:

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Ice Specialist
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Gary R. E. Beckstead, M.Sc., P.Eng.
Reviewer

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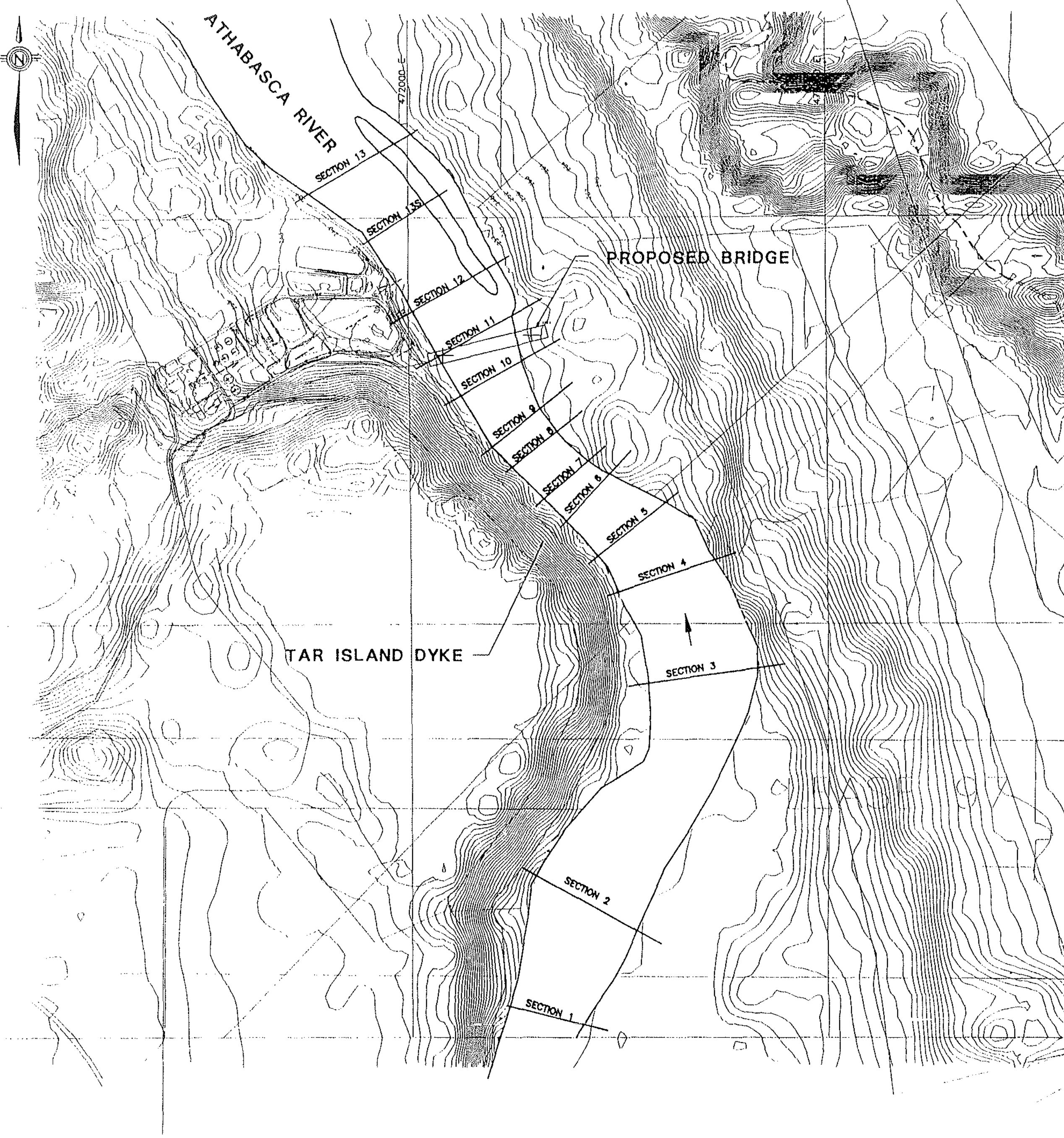
APPENDIX A

River Cross-Sections



NOTE:

1. CROSS-SECTIONS WERE SURVEYED BY AEE (JULY, 1995) USING AN ECHO SOUNDER FOR CONTINUOUS CHANNEL CROSS-SECTION MEASUREMENT, AND A THEODOLITE FOR LEFT AND RIGHT BANK SURVEYS.
2. CROSS-SECTIONS 13S AND 1 ARE COMBINED SECTIONS FROM AEE AND SUNCOR (SEPT, 1995) SURVEYS.

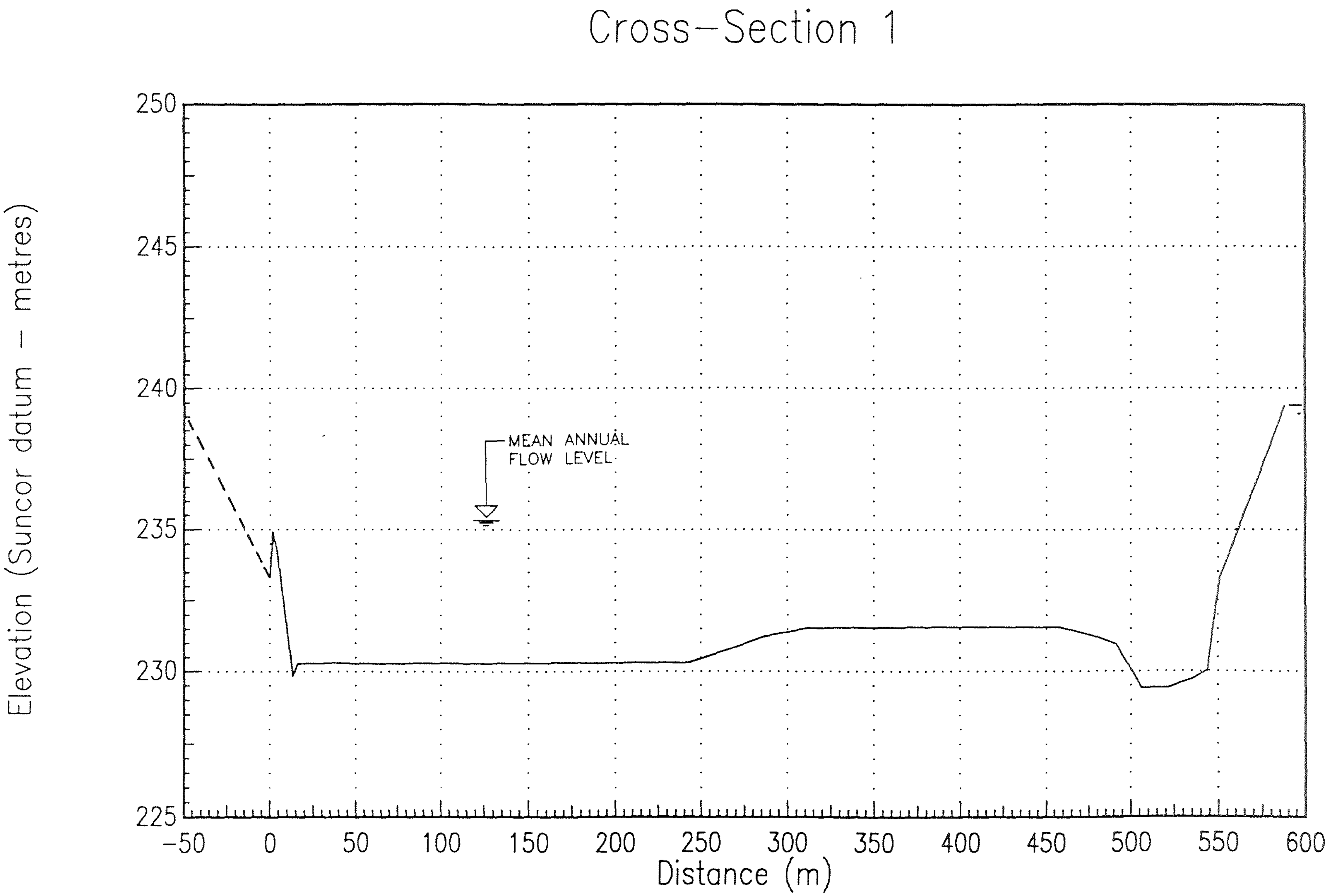
0 200 400 600 800 1000m

1 20000

AGRA
Earth & Environmental Limited
ENGINEERING & ENVIRONMENTAL SERVICES

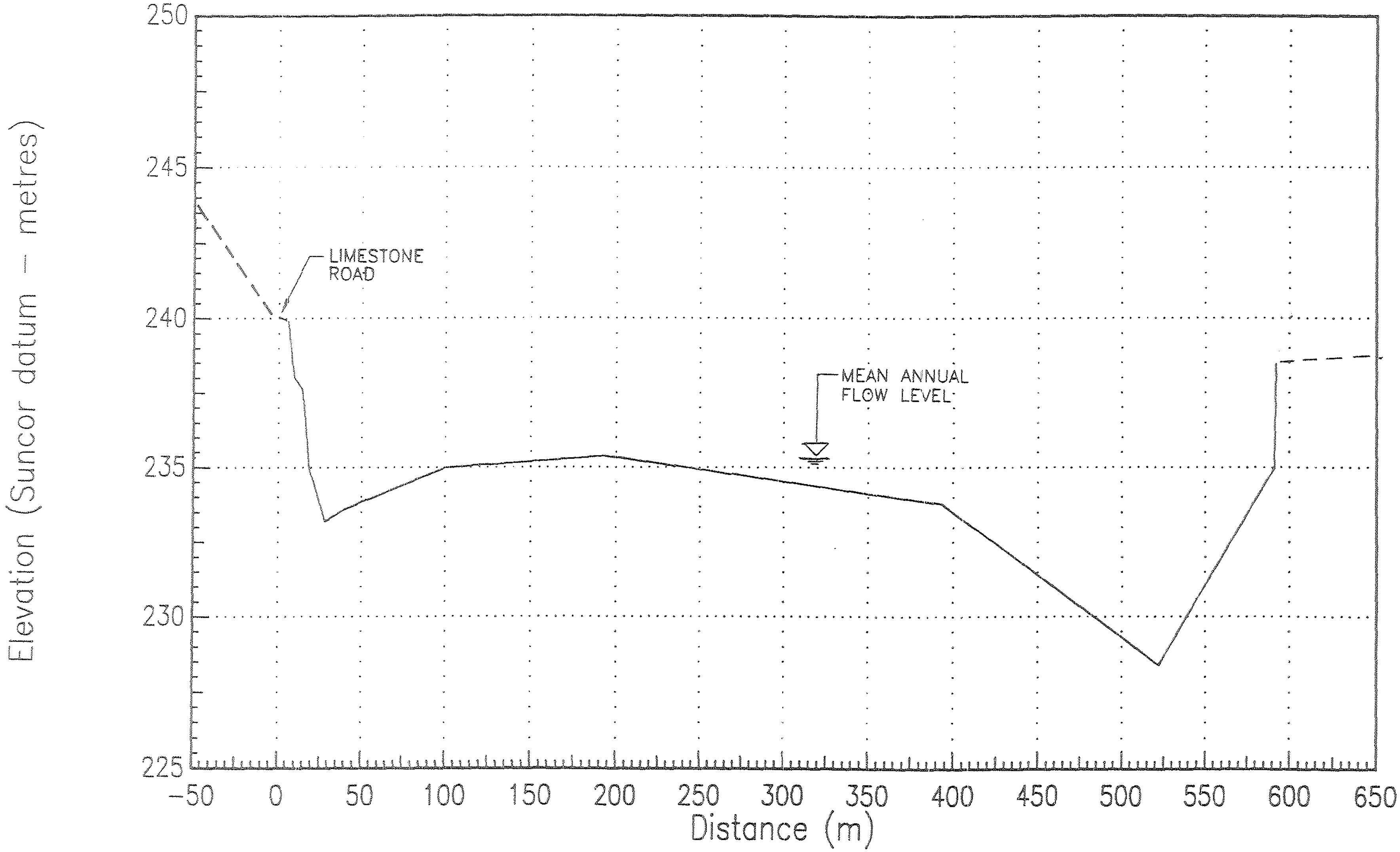
CLIENT: H.A. SIMONS LTD.	DATE: DECEMBER 1995	
PROJECT: SUNCOR-ATHABASCA RIVER BRIDGE	JOB No. CW1466.00	
CROSS-SECTION LOCATIONS	FIGURE: FIGURE A-1	REV. 1

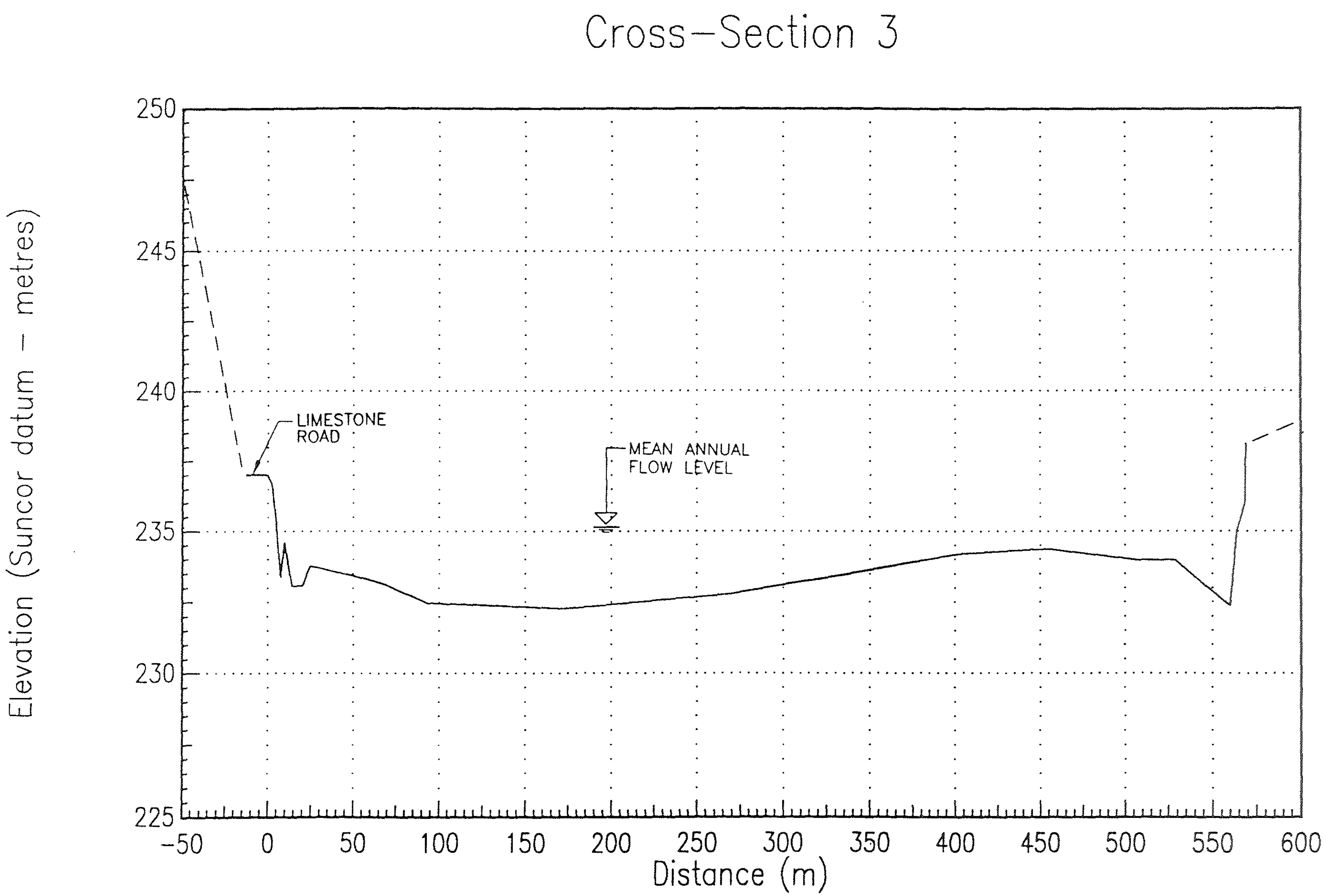
I:\CW\1466\1466-101.DWG 1:20



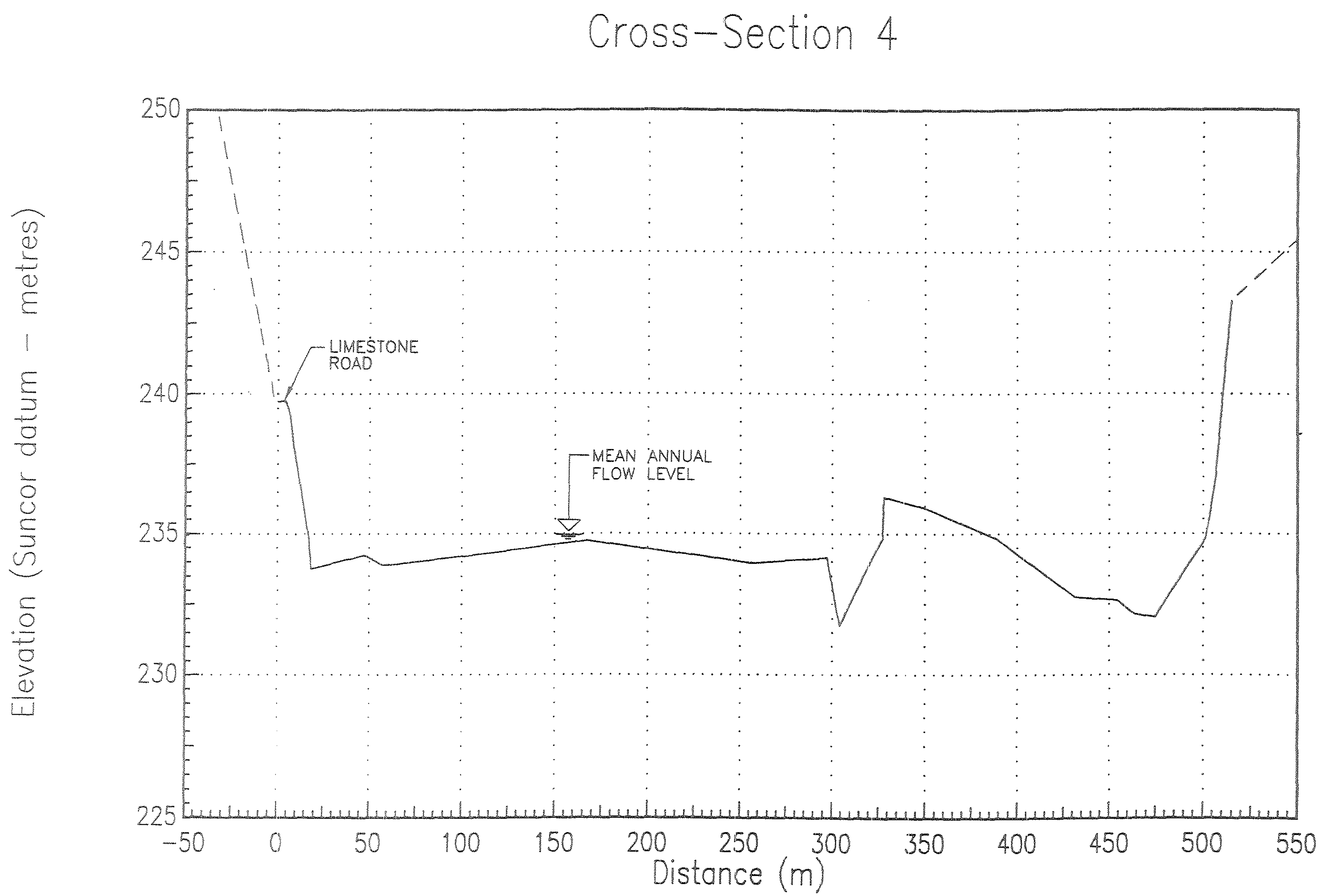
Cross-Section 1
Elevation (Suncor datum - metres)
250
245
240
235
230
225
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
Distance (m)
MEAN ANNUAL
FLOW LEVEL

Cross-Section 2

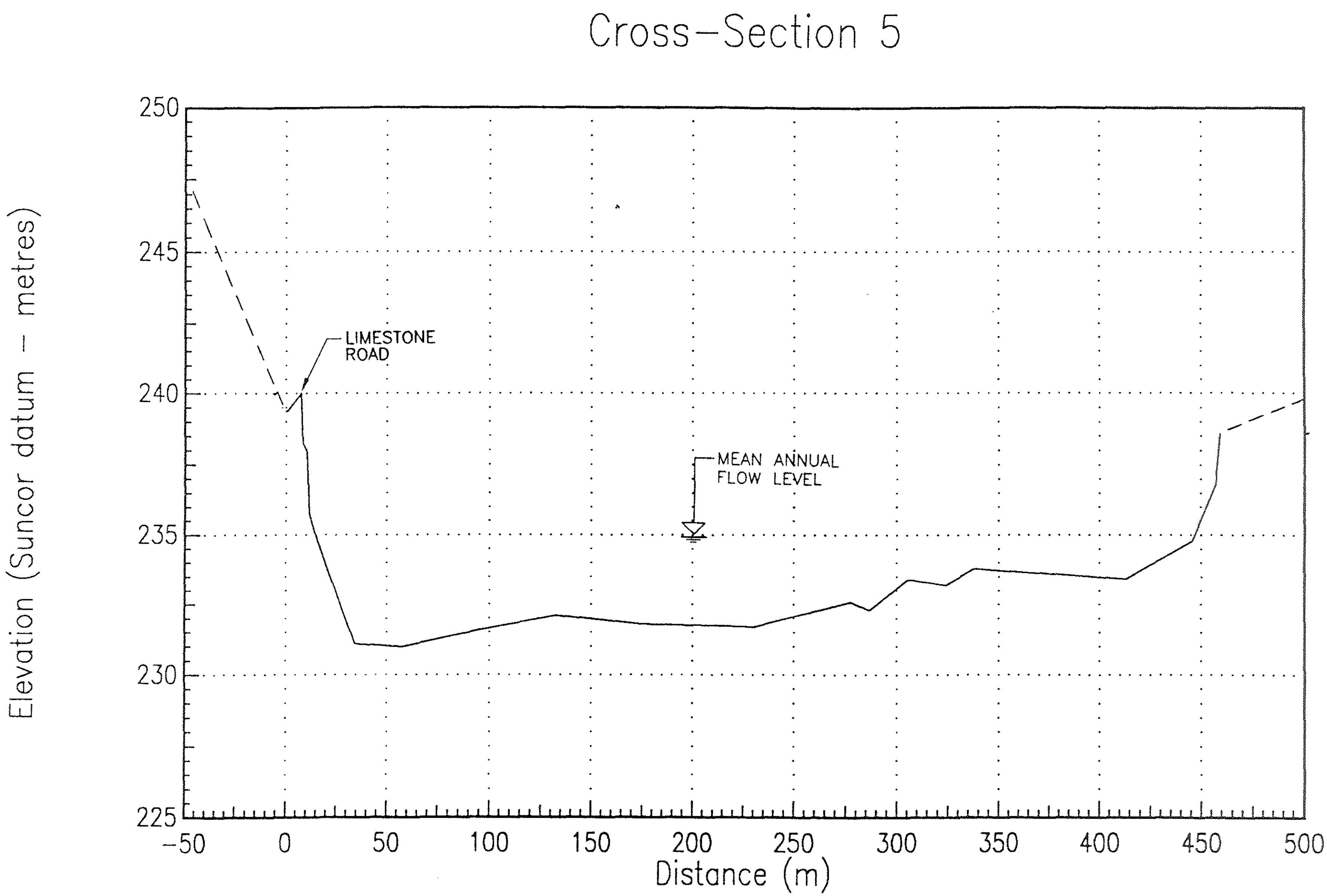


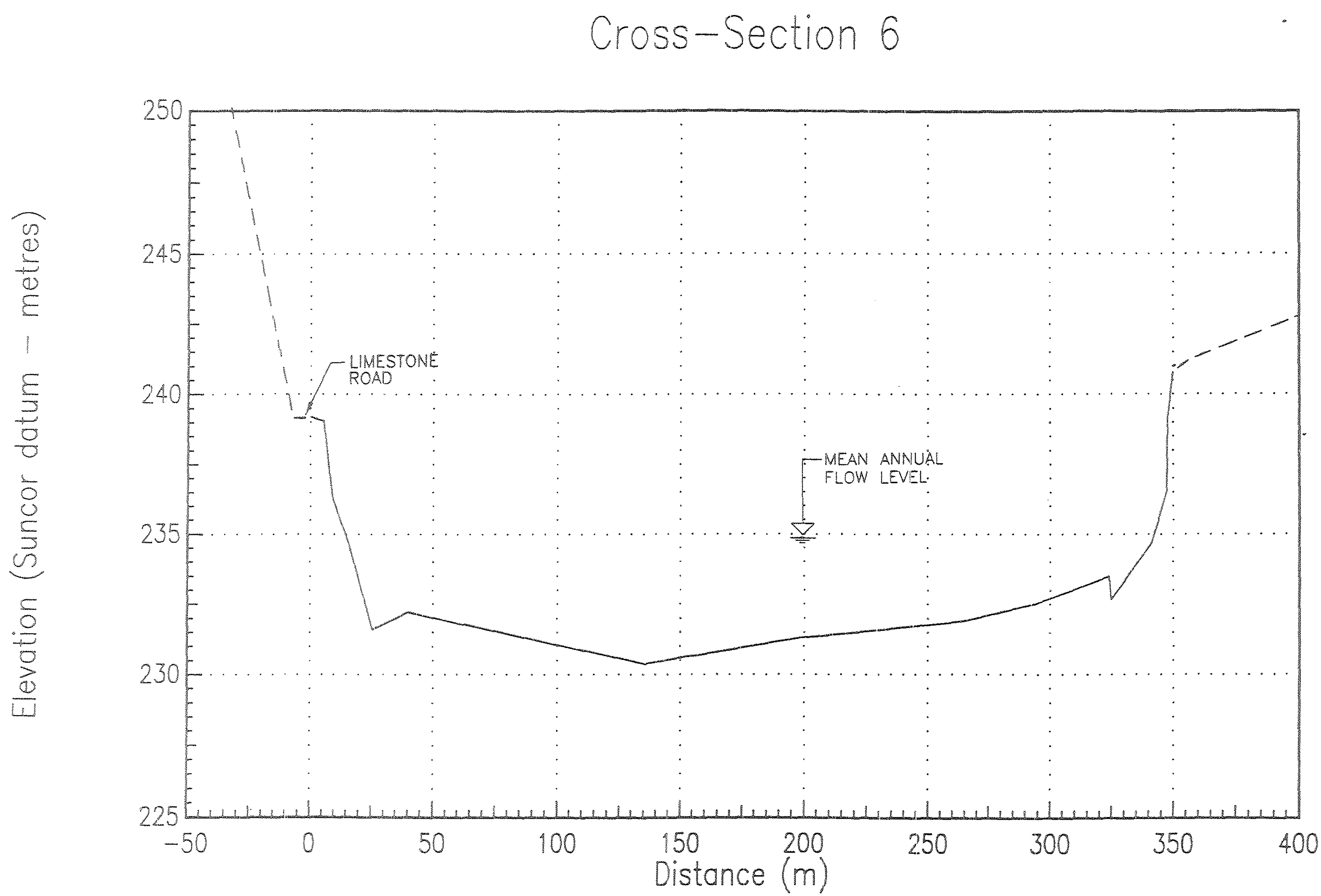


Cross-Section 3
Elevation (Suncor datum - metres)
250
245
240
235
230
225
LIMESTONE ROAD
MEAN ANNUAL FLOW LEVEL
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
Distance (m)

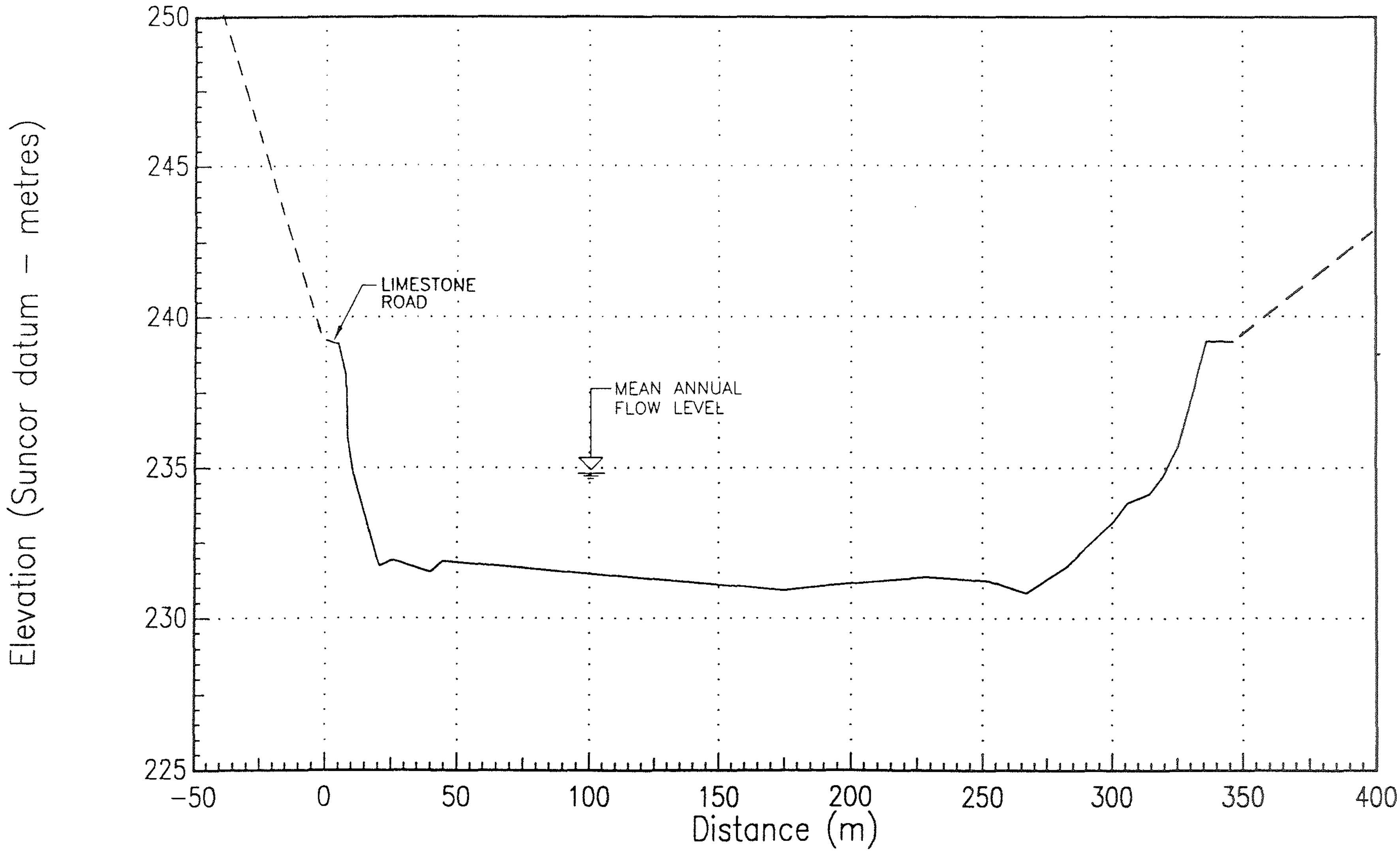


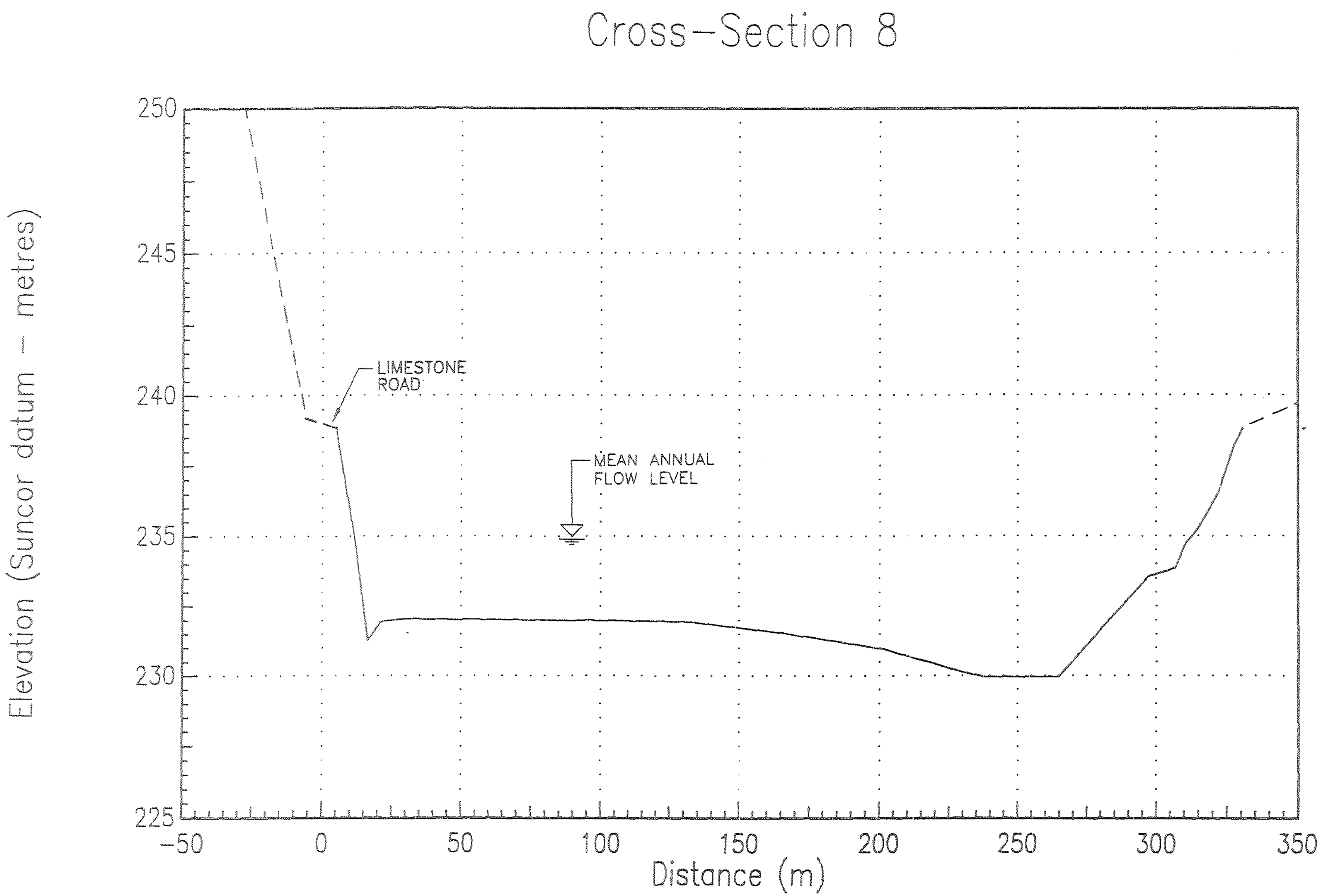
Cross-Section 4
Elevation (Suncor datum - metres)
250
245
240
235
230
225
-50
0
50
100
150
200
250
300
350
400
450
500
550
Distance (m)
LIMESTONE ROAD
MEAN ANNUAL FLOW LEVEL


Cross-Section 5
Elevation (Suncor datum - metres)
250
245
240
235
230
225
-50
0
50
100
150
200
250
300
350
400
450
500
Distance (m)
LIMESTONE ROAD
MEAN ANNUAL FLOW LEVEL


Cross-Section 6
Elevation (Suncor datum - metres)
250
245
240
235
230
225
-50
0
50
100
150
200
250
300
350
400
Distance (m)
LIMESTONE ROAD
MEAN ANNUAL FLOW LEVEL

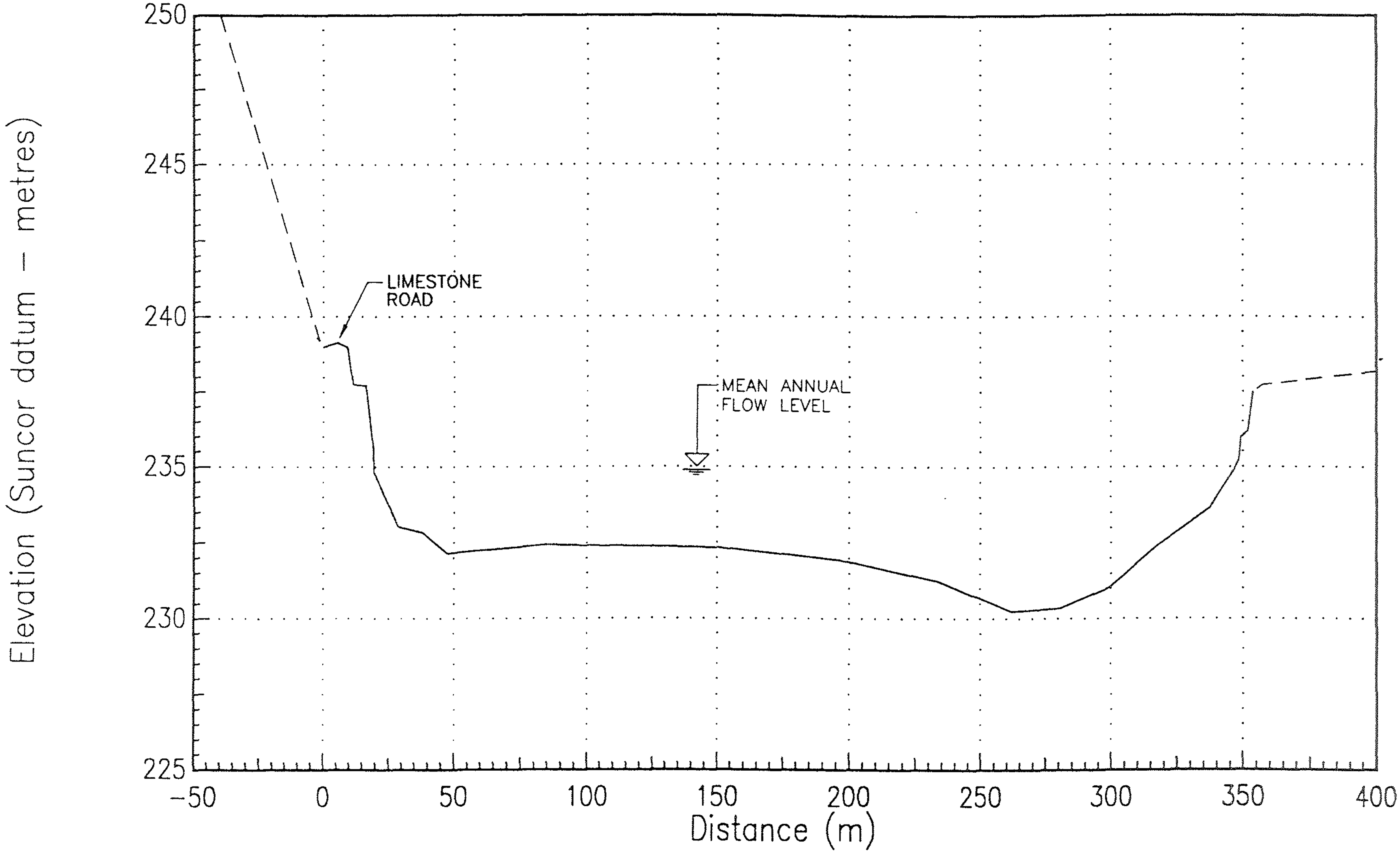
Cross-Section 7

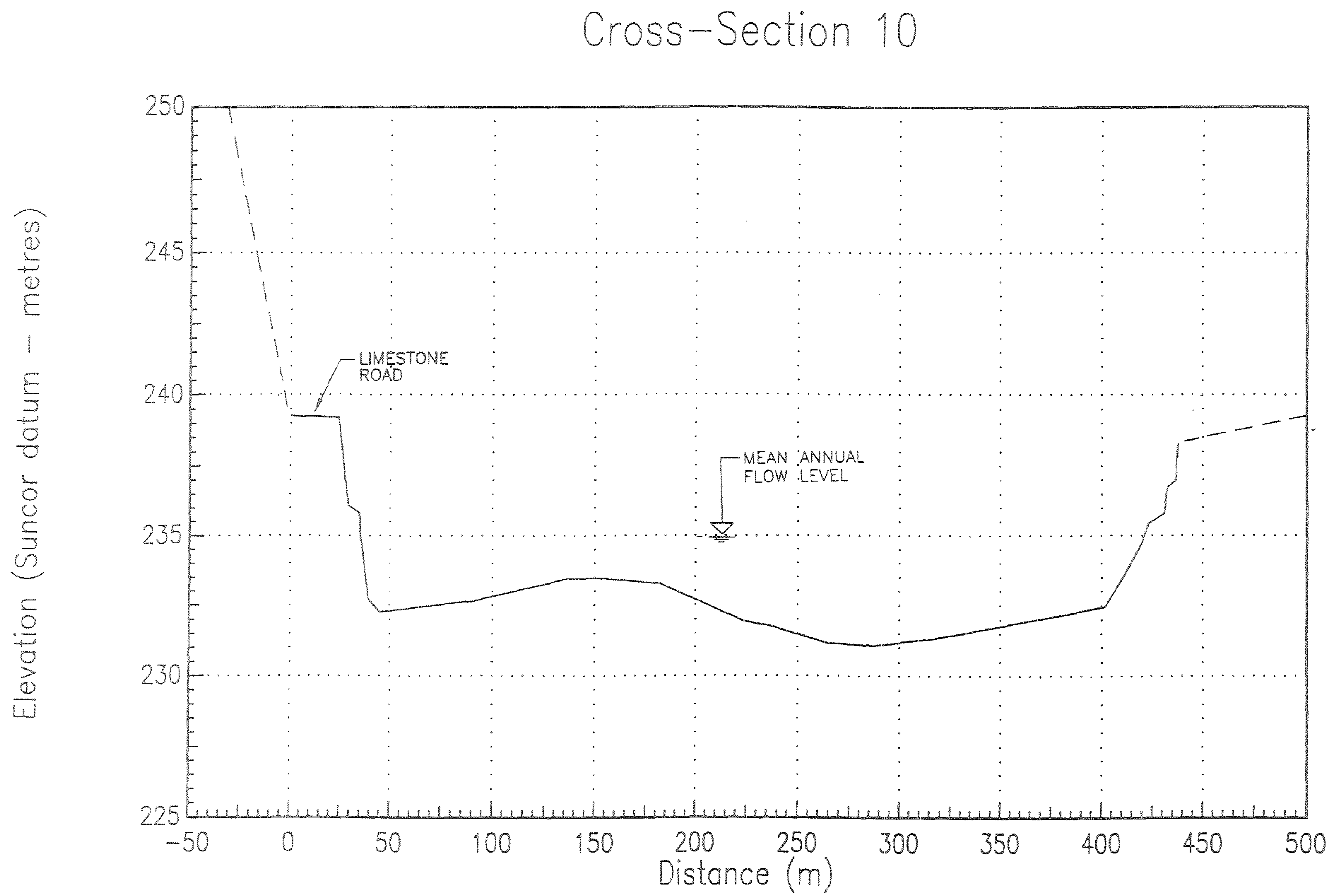




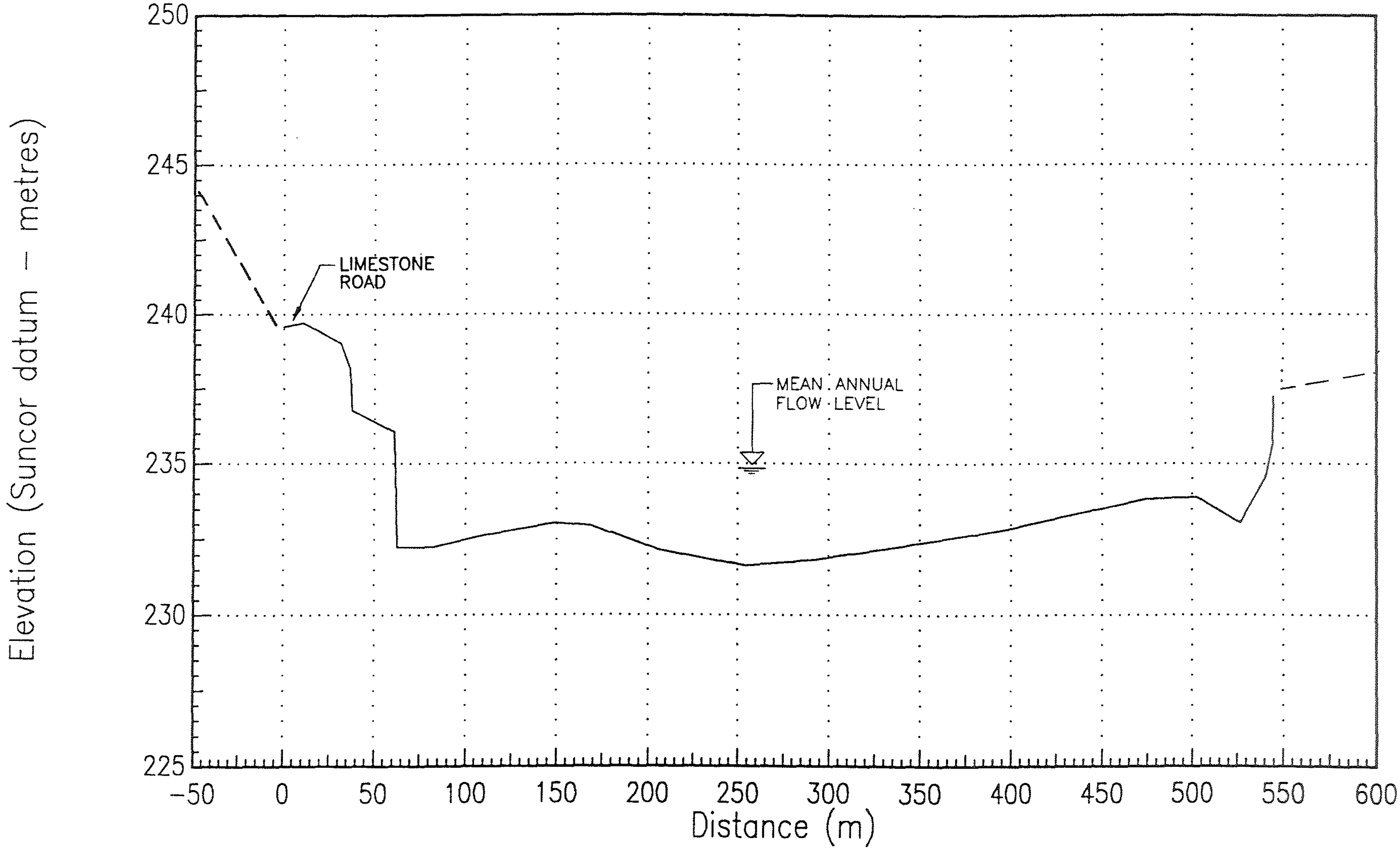
Cross-Section 8
Elevation (Suncor datum - metres)
Distance (m)
LIMESTONE ROAD
MEAN ANNUAL FLOW LEVEL
250
245
240
235
230
225
-50
0
50
100
150
200
250
300
350

Cross-Section 9

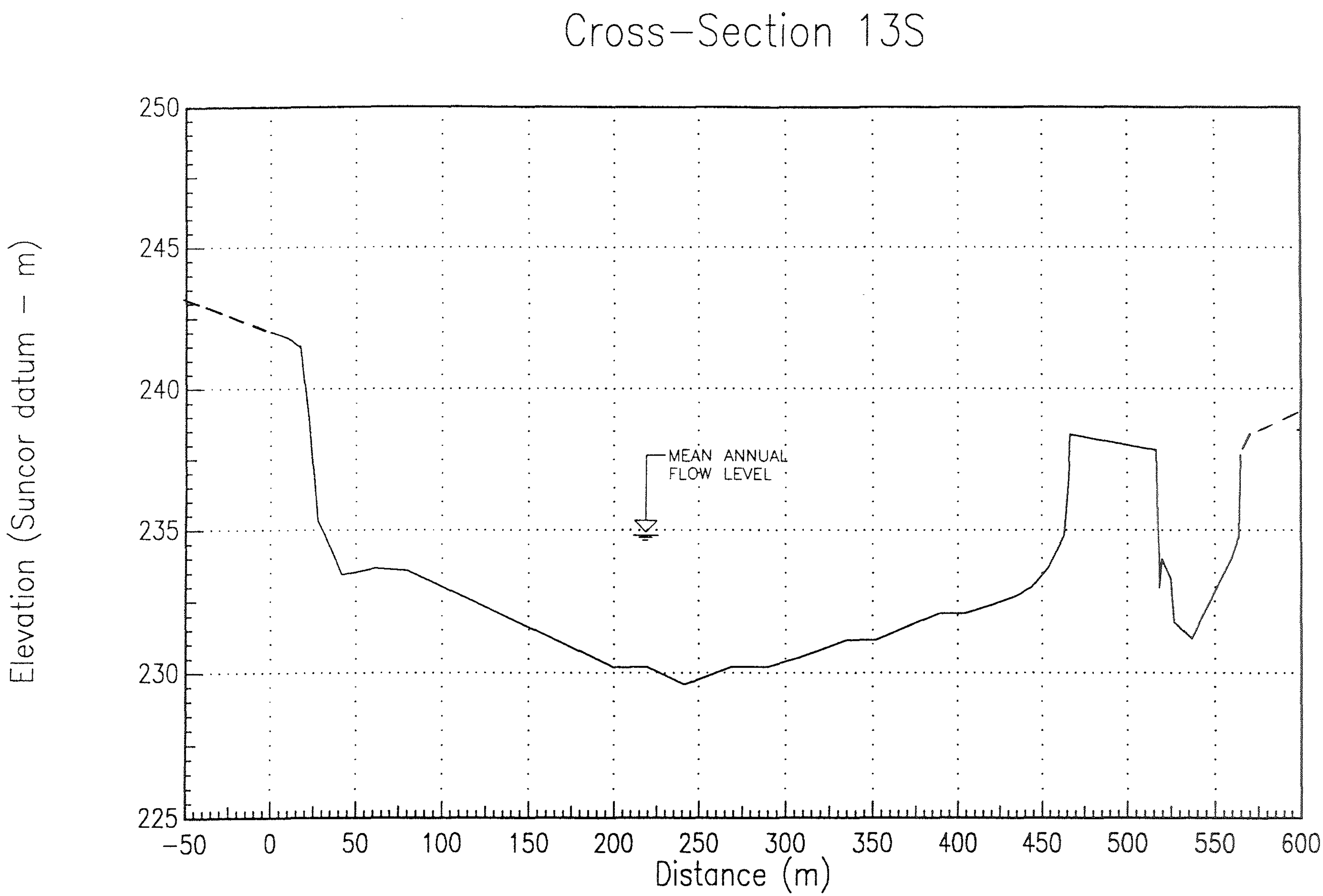



Cross-Section 10
LIMESTONE ROAD
MEAN ANNUAL FLOW LEVEL
Elevation (Suncor datum - metres)
250
245
240
235
230
225
-50
0
50
100
150
200
250
300
350
400
450
500
Distance (m)

Cross-Section 11




Cross-Section 12
Elevation (Suncor datum - metres)
250
245
240
235
230
225
-50
0
50
100
150
200
250
300
350
400
450
500
550
600
Distance (m)
MEAN ANNUAL
FLOW LEVEL


Cross-Section 13S
Elevation (Suncor datum - m)
Distance (m)
MEAN ANNUAL FLOW LEVEL
250
245
240
235
230
225
-50
0
50
100
150
200
250
300
350
400
450
500
550
600

Cross-Section 13