Ice jams and flood forecasting, Hay River, N.W.T. - Phase 2:
surges and interactive computer program

by

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SUMMARY

In Phase 1 of this study an operational ice jam flood forecast algorithm was developed. However, there were at least two important deficiencies in this algorithm. Firstly, the problem of surges released by ice jam failure upstream, which have been responsible for floods in Hay River in the past, were only considered in an empirical and indirect way. Secondly, the algorithm reflected the quite complex situation in the Hay River delta at break-up and was therefore difficult for the non-technical personnel of the Hay River Flood Watch to use.

To remedy these problems, this Phase 2 study was undertaken. The first component of this phase was to evaluate potential surges which could be released by sudden ice jam failure upstream in detail, using a sophisticated finite element unsteady flow analysis required for a river reach such as that of the Hay River. This analysis also necessitated a limited amount of field survey to document the hydraulic geometry of the Hay River.

The results of the surge analysis were simplified through dimensional analysis and some approximations to a series of influence lines which allow a ready assessment of the magnitude and timing of such surges when they reach Hay River, knowing only the river discharge and the ice jam location and length.

To overcome the problem of the application of the rather complex flood forecast algorithm, the other component of the Phase 2 study was development of a user-friendly interactive personal computer program which incorporated the algorithms developed in both the Phase 1 and 2 studies.

While the flood forecast algorithm is quite operational as it now stands, incorporation of two further components should improve it further. Both are relatively minor. The first is an algorithm to allow prediction of ice jam melt which in some years reduces the potential flood threat. The other is a precipitation-runoff algorithm which will allow the forecast period of about 1 day to be extended somewhat and will relieve the current reliance on estimates of the discharge from the WSC gauge at the border.

However, the most important future requirement is for systematic break-up observations in Hay River to assess and become familiar with the application of the flood forecast procedure and possibly to lead to its improvement.

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As for the previous phase of this study, thanks are due to Mr. Jesse Jasper and Dr. Terry Prowse of Environment Canada, who supervised the contract and provided timely guidance and assistance in its performance. Again Mr. Brian Latham of Indian and Northern Affairs provided day-to-day coordination of the study, assistance in the field and in the procurement of some of the background data, and advice, especially in the development of the interactive program.

Water Survey of Canada staff also provided support to the project. The Fort Smith office under Mr. Murray Jones was responsible for the development of the rating curves for the Hay River at the border and at Hay River, as well as the reach survey at the border station. The Peace River office under Mr. Vance Elder provided the rating curves for the Chinchaga River and the Hay River near Meander River, and carried out the reach survey of the Chinchaga River at the Highway 58 crossing.

The development of the finite element unsteady flow analysis was carried out under the supervision of Dr. Peter Steffler of the Department of Civil Engineering at the University of Alberta. Mr. Sheldon Lovell, Senior Technologist in the Department of Civil Engineering, and Mr. Jeff Rabinovitch, summer student, provided assistance with the summer field surveys.

1.0 INTRODUCTION

An ice jam flood forecast algorithm was developed for Hay River by Gerard and Stanley (1988) as Phase 1 of this study. During this phase of the study it became apparent that surges released by ice jam formation and failure can play a significant role in ice jam floods in Hay River. This was especially apparent during the flood of 1985. As reported by Wedel (1989): "May 7 - Just after midnight, at 00:30, a huge surge of water and ice was reported moving down the West Channel. Within 15 minutes roads in [sic] the West Channel were flooded to depths exceeding one metre". Such rapid increases in water level can only be explained by the action of surges released by ice jam failure.

Some allowance for surges should therefore be incorporated into any flood forecast procedure for Hay River. In the Phase 1 report this was handled in only an empirical, and indirect, way. Hence it was the intent of this Phase 2 investigation to develop a supplement to the flood forecast procedure which would take explicit account of the possibility of surges released by ice jam failure.

Figure 1.1 shows the Hay River upstream of Hay River. Ice jams have formed in various locations along this reach and, as the break-up observations of 1989 indicate (Gerard and Jasek, 1990), the failure of these jams can have a significant influence on break-up over large distances downstream. Consequently, in this Phase 2 study, various ice jam locations were considered over some 300 km upstream of Hay River.

The final product of this investigation had to be in a form convenient for use by the Town Flood Watch. For this reason the intent was to develop a procedure which could be readily adapted to a user-friendly interactive computer program. To do this, 'influence' lines were developed which gave the magnitude and timing of the discharge increase in Hay River caused by the arrival of a surge, as a function of the three most important variables influencing the impact of an upstream ice jam release on the Town of Hay River. These variables were taken to be the jam location, the ('carrier') discharge in the river immediately prior to jam failure, and the jam length. However, while this product is reasonably simple, the analysis behind it of unsteady flow in the relatively steep lower Hay River, over reaches interrupted by rapids and waterfalls, required application of complex state-of-the-art unsteady flow analysis.

A fundamental need for this analysis was definition of the channel geometry over the 300 km considered. This could have been estimated roughly from available topographic maps. However, in an effort to 'ground truth' the information from these maps, field measurements were carried out to measure the hydraulic geometry of the river at salient locations. The sites selected included the Water Survey of Canada (WSC) stations along the reach and other locations thought to be representative of the various geomorphic units along the river, as well as observed ice jam sites.

Figure 1.1. Plan of Hay River showing river distances (see also larger version on last page foldout).

2.0 SCOPE OF WORK

The terms of reference for the study are given in Appendix A. The scope of work developed from these included the following:

1. River surveys over selected river reaches to define the hydraulic geometry of those reaches;

2. Synthesis of the field survey results with the information available from topographic maps and other sources to get a coherent, continuous and smooth definition of the variation in hydraulic geometry over the 300 km reach of interest;

3. Adaptation of a recently-developed finite element unsteady flow algorithm to the situation on the Hay River;

4. Multiple simulations using the finite element algorithm to determine the effects in Hay River of jam failure of different jam locations, lengths and carrier discharges.

5. Development of an appropriate dimensionless representation of the results of the unsteady flow analysis and generation of appropriate, but simplified, influence lines on the basis of these dimensionless parameters; and

6. Development of an interactive and user-friendly computer program incorporating the results of the Phase 1 and 2 studies for use by the non-technical personnel of the Hay River Flood Watch.

These steps are discussed in the following in the above order.

3.0 FIELD SURVEYS

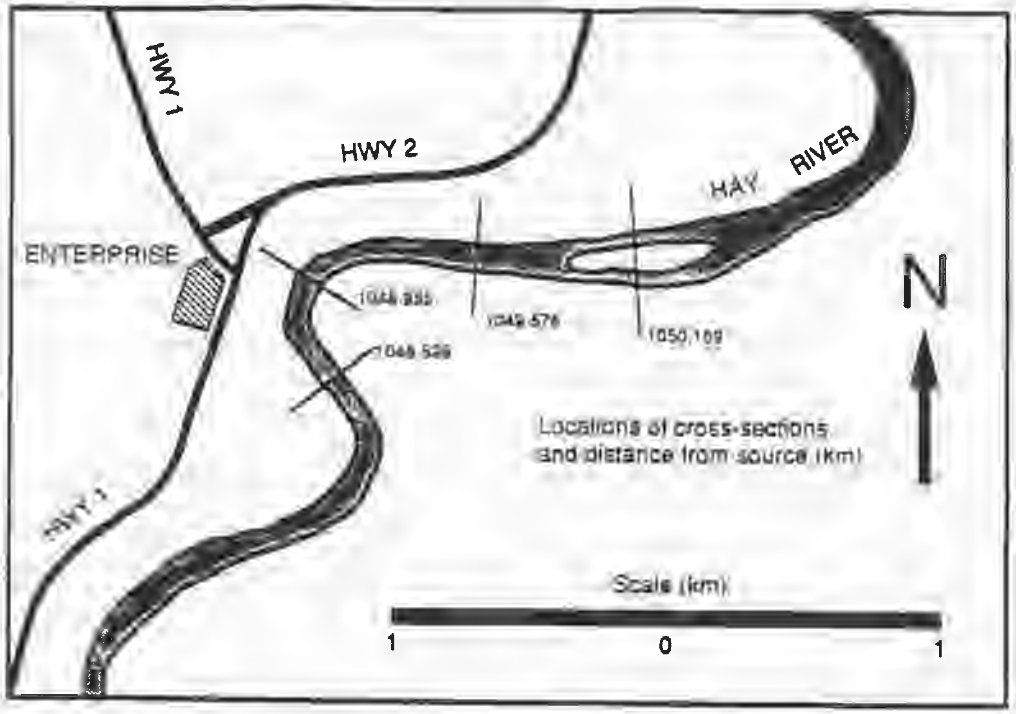
For the unsteady flow analysis it was necessary to recast the natural and quite irregular Hay River as an hydraulically-equivalent rectangular channel. This required preparation of a coherent, continuous and somewhat smoothed approximation of the channel width, bed elevation and hydraulic roughness over the 300 km reach of interest. To do this it was planned to define the hydraulically-equivalent rectangular channel at a limited number of characteristic reaches, on the basis of detailed field surveys of these sites, and then to interpolate the hydraulic characteristic variation between these surveyed reaches using information gleaned from topographic maps.

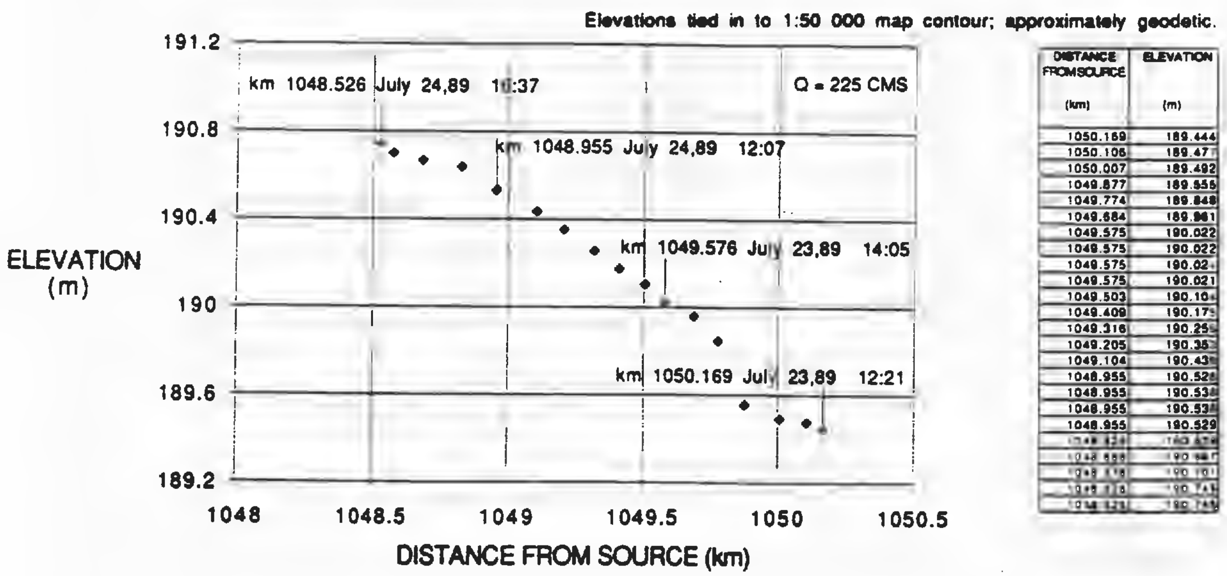
These field surveys were carried out at 8 locations between the Highway 58 crossing of the Chinchaga River and the Town of Hay River. The locations are indicated in Figure 1.1 and have been designated as follows: Chinchaga River at Highway 58; Hay River at Meander River; near Steen River; at the NWT/Alberta border; near Swede Creek; near Enterprise; at Paradise Gardens; and at Hay River. The data for two of these sites - the WSC stations at the Highway 58 crossing of the Chinchaga River and at the border - were collected by WSC staff. The remainder were surveyed by University of Alberta personnel.

The intent of the survey of each site was to determine the hydraulic geometry such that an hydraulically-equivalent reach-averaged rectangular channel could be defined for the finite element analysis. This entailed surveying sufficient cross-sections over each reach to define a reach-averaged width, hydraulic roughness and bed elevation. Hence, at each site the intent was to survey 4 or 5 cross-sections distributed over some 20 river widths.

The below-water portion of each cross-section was surveyed using sonar, and the above-water portion using standard land survey techniques. A longitudinal water surface profile was surveyed along the reach and the levels at each section tied to a common datum. Where possible the surveys were tied into geodetic datum. At the one reach where this was impractical, at Enterprise, the survey was approximately tied into geodetic using information from the topographic maps. The discharge in each reach on the day(s) of survey was estimated from WSC data.

Exceptions to this general pattern were as follows: at the 2 stations surveyed by WSC their standard slope-area surveys were carried out so only 3 cross-sections were taken over a relatively short reach of river; due to a serious injury sustained by one of the field crew, at Steen River only one full cross-section was surveyed, with only the below-water portion being surveyed at the other sites; and, at Paradise Gardens, only one full cross-section was surveyed as a check on the sections surveyed by UMA in 1977 (UMA,1979). The results of a typical survey - that of the Enterprise reach - are shown in Figures 3.1. The remainder are given in Appendix B.





DISTANCE FROM SOURCE (km)	ELEVATION (m)
1050.169	189.444
1050.106	189.47
1050.007	189.492
1049.877	189.555
1049.774	189.848
1049.684	189.961
1049.575	190.022
1049.575	190.022
1049.575	190.02
1049.575	190.021
1049.503	190.10
1049.409	190.17
1049.316	190.25
1049.205	190.35
1049.104	190.43
1048.955	190.52
1048.955	190.53
1048.955	190.53
1048.955	190.529
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]

Figure 3.1. Hydraulic geometry of the Enterprise reach: (a) Plan (b) Longitudinal water surface profile on the day of survey.

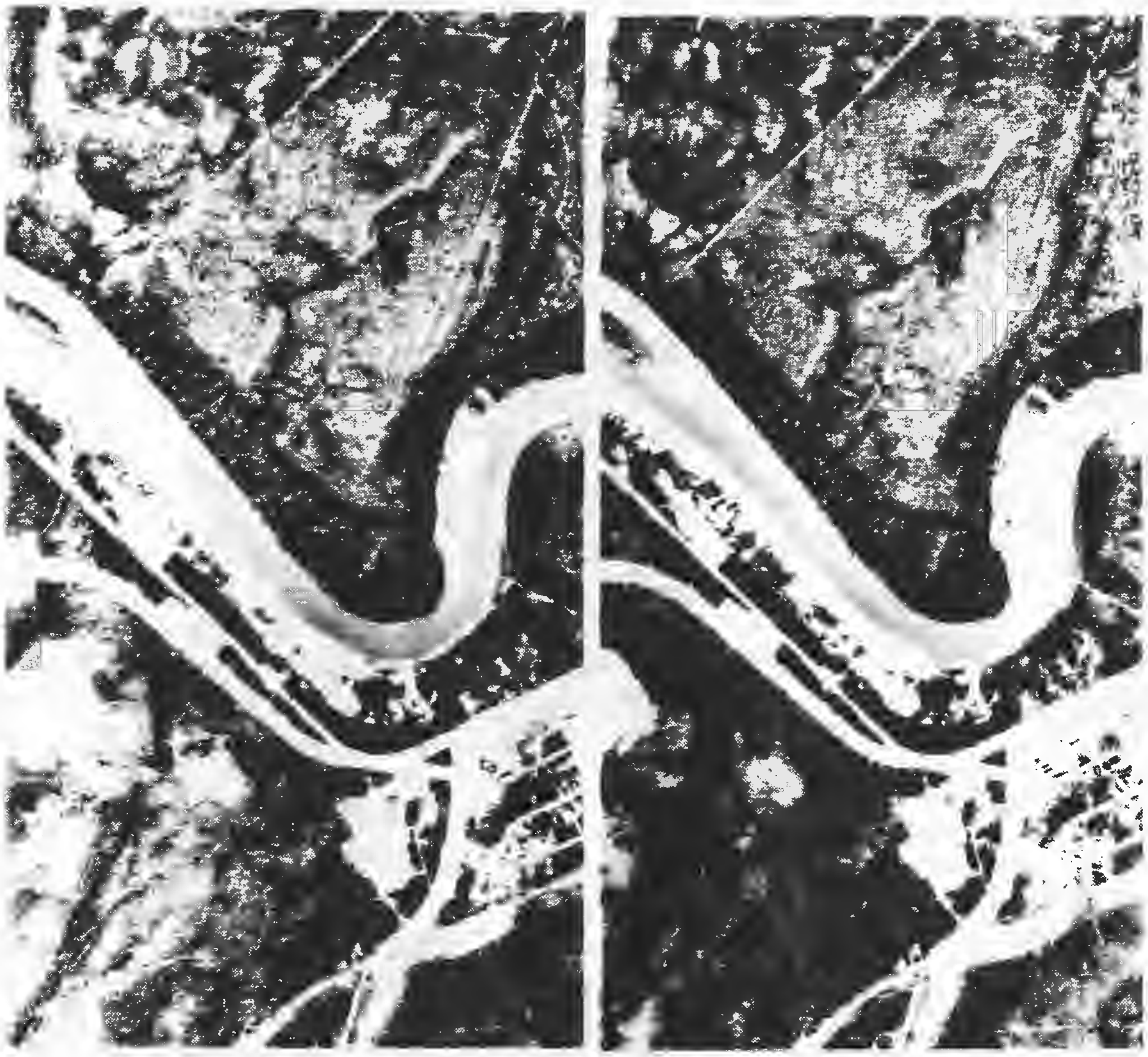


Figure 3.1 (continued) Hydraulic geometry of the Enterprise reach: (c) Airphoto of the reach, June 18, 1979, scale 1:20 000

Figure 3.1 (continued) Hydraulic geometry of the Enterprise reach: (d) Panorama of reach from the left bank, July 23, 1990, Q = 225 m^3/s.

Figure 3.1 (continued) Hydraulic geometry of the Enterprise reach Photograph of (e) Left bank, (f) Right bank at km 1048.53.

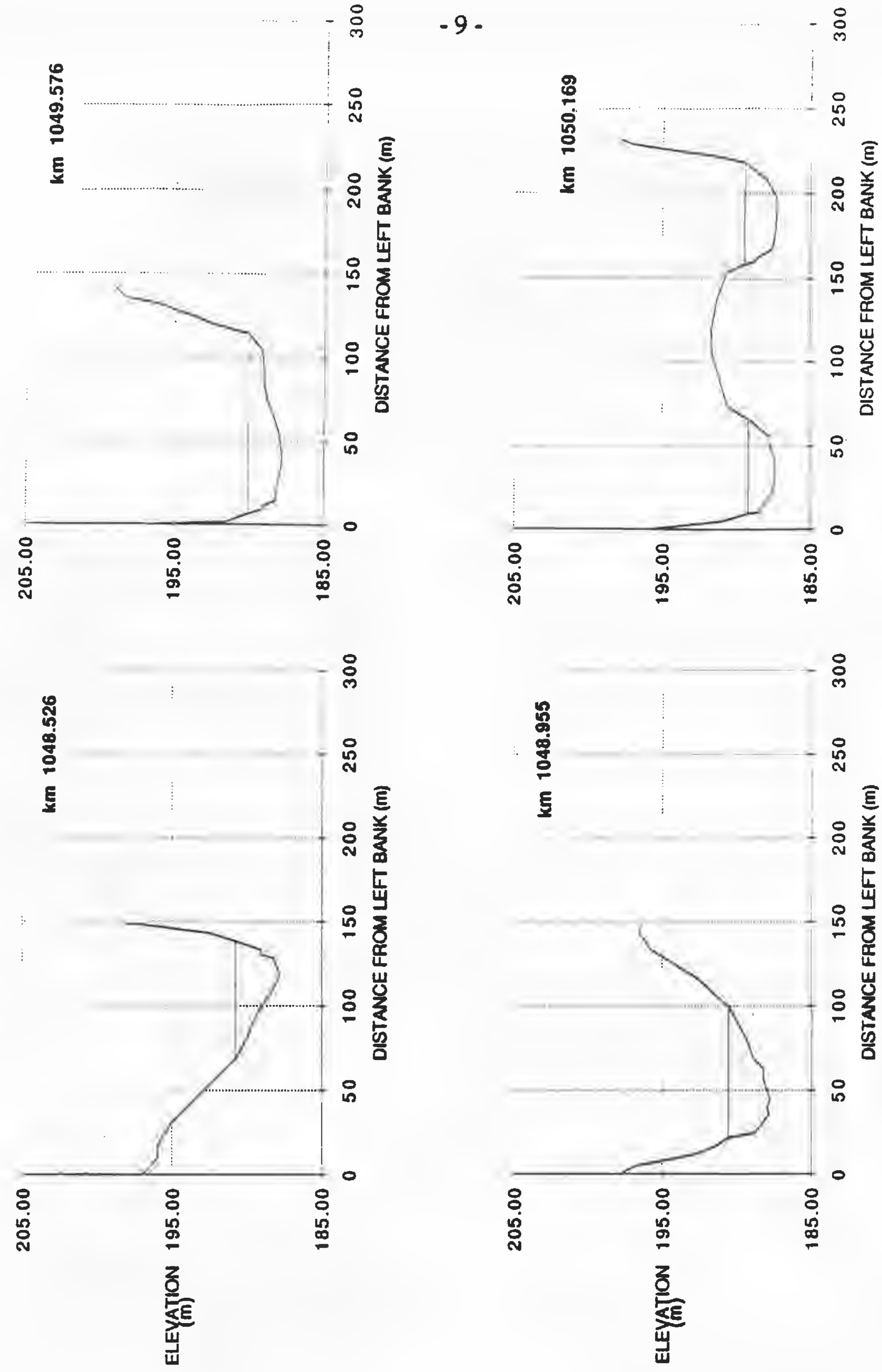


Figure 3.1 (continued). Hydraulic geometry of the Enterprise reach: (h) Surveyed cross-sections

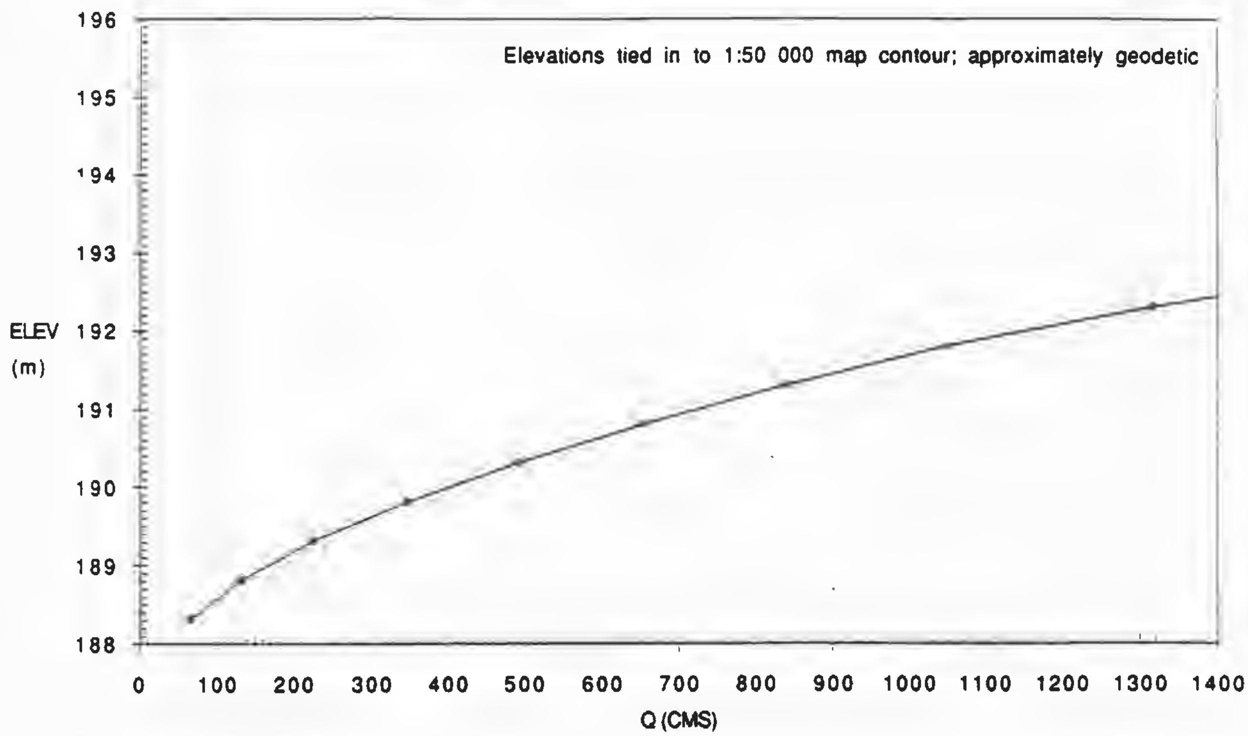


Figure 3.1 (continued). Hydraulic geometry of the Enterprise reach: (i) Calculated rating curve based on the hydraulic roughness deduced from the reach-averaged hydraulic geometry on the day of survey.

4.0 CHARACTERISTICS OF AN EQUIVALENT CHANNEL

To define an equivalent 'rectangular' channel, the variation in reach-averaged width, bed and/or water surface elevation, and hydraulic roughness over the 300 km reach must be estimated. As indicated earlier, this was done through a synthesis of the above field data and information from topographic maps. In addition, to provide an hydraulic datum, an estimate was required as to how a reference discharge, of a given probability, varied along this reach.

4.1 Reference discharge variation

The reference discharge was chosen to be the 50% probability flood (2 year flood) as this approximates the bankfull flow and, in combination with the hydraulic analysis of each surveyed reach, allows determination of a consistent and hydraulically significant reference water surface along the river. The definition of the reference discharge variation along the river was based on a probability analysis of the annual peak discharges of the Chinchaga River at the Highway 58 crossing, the Hay River near Meander River and the Hay River at Hay River. The probability distributions for these locations are shown in Figure 4.1.

The reference discharge variation between these stations was estimated by interpolation on the basis of the length of tributaries discharging into each reach. The result is shown in Figure 4.2.

4.2 Longitudinal profile of the reach

The effective bed elevation of the equivalent rectangular channel was defined at each surveyed reach by determining the reach-averaged mean depth for the reference discharge and then subtracting this from the elevation of the geodetic water level for this discharge, as determined from either the measured or a synthesized rating curve for the reach. This gave a series of fixed mean bed elevations along the reach. Between these points the profile of the mean bed elevation was interpolated on the basis of the longitudinal profile determined from 1:50000 topographic maps.

The resulting mean bed elevation profile is plotted in Figure 4.3. The lower line represents the bed profile and the upper line the water surfaces for the reference discharge. In reality the steps evident in the latter profile would be smoothed out by backwater curves. The estimated bed elevations for each half-kilometre of the total reach are tabulated in Appendix C. The resulting slopes of the various reaches are plotted in Figure 4.4.

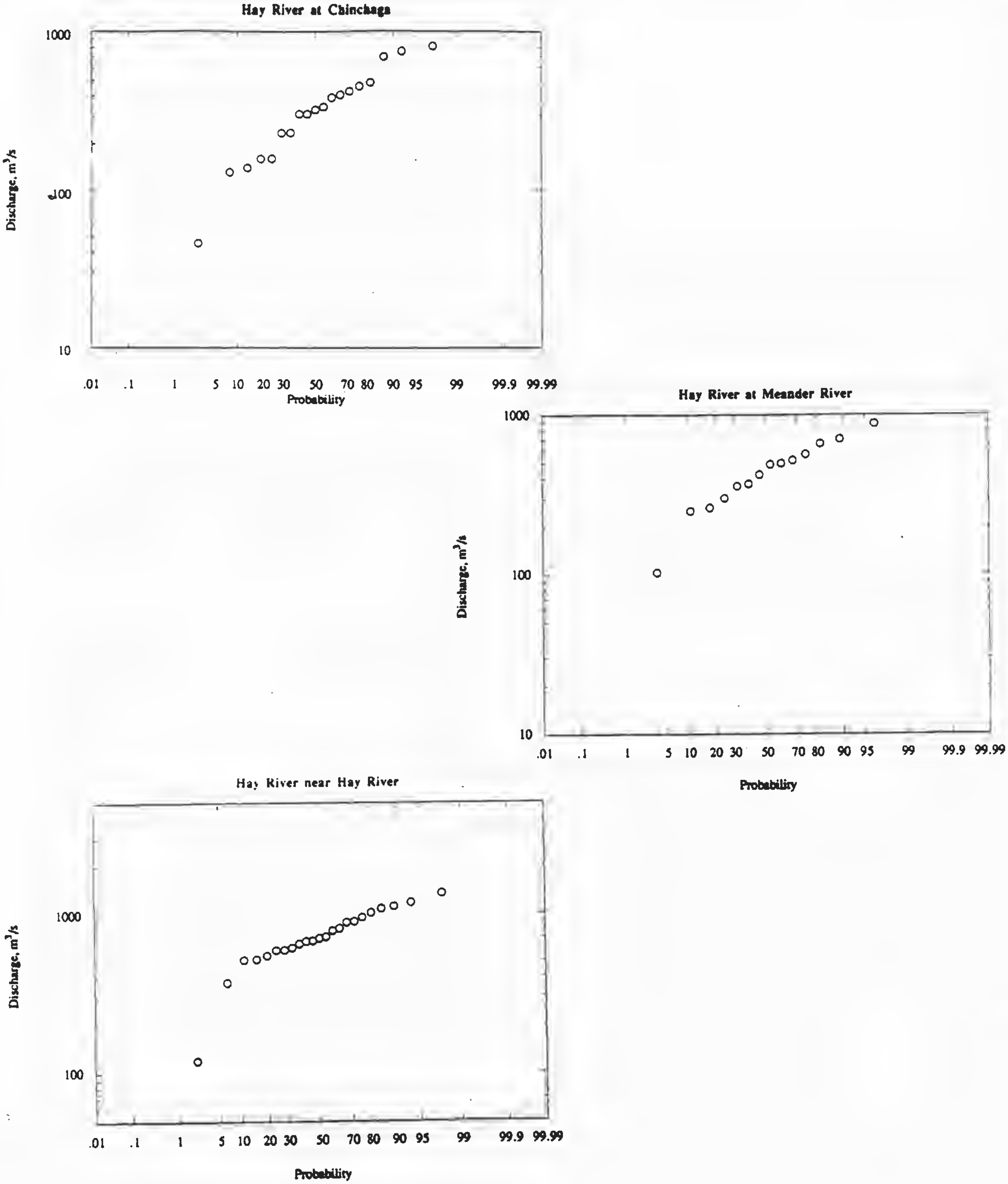


Figure 4.1. Probability distributions of the annual peak mean-daily flows at the three WSC stations along the Hay River : (a) Chinchaga River at the Highway 58 crossing , (b) Hay River at Meander River and (c) Hay River near Hay River.

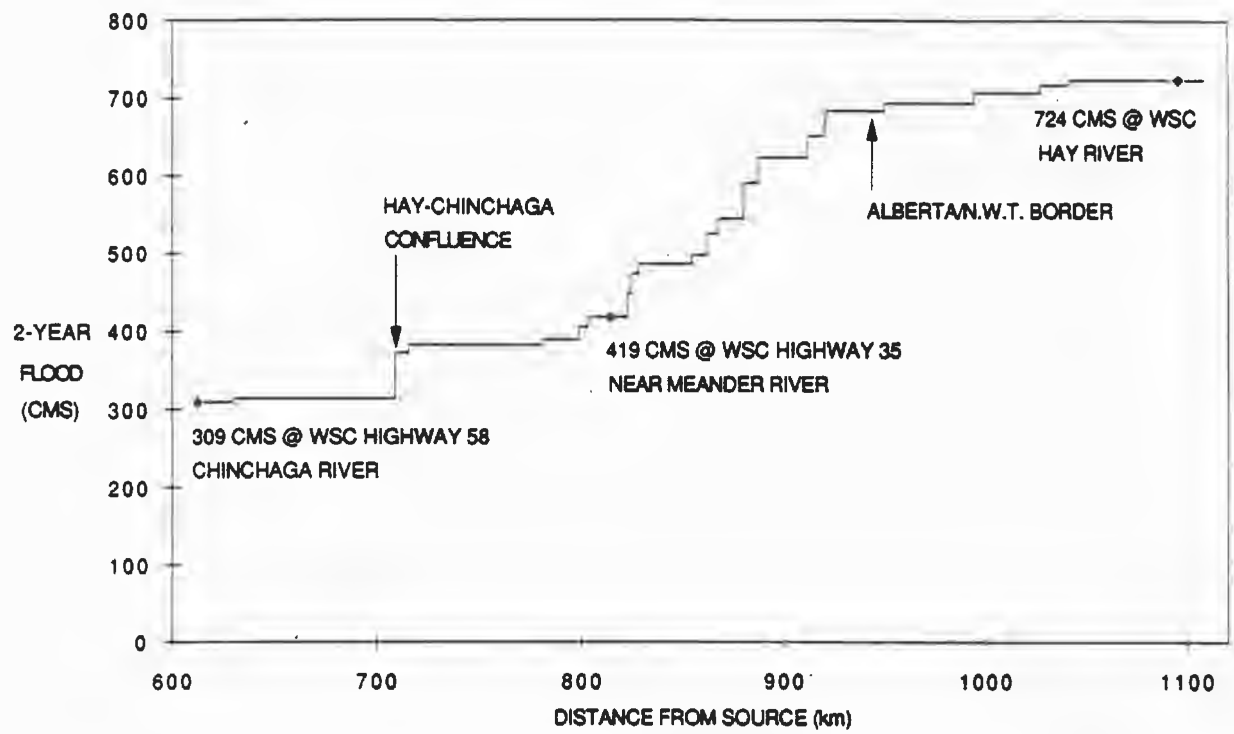


Figure 4.2. Estimated variation in the 50% (2 year) flood along the Hay River.

4.3 Variation in width of the equivalent channel

The Hay River displays considerable variation in channel width along its length, with often sudden changes over relatively short distances. For the purposes of the numerical study it was necessary to derive a somewhat smoothed version of this variation.

The appropriate width was taken to be that of the water surface at the reference discharge - approximately the bankfull width. The only locations where this was known was in the surveyed reaches, but relative widths could be determined from 1:50000 topographic maps or air photos. The ratios of the surveyed widths to those determined from the topographic maps is illustrated in Figure 4.5. An average value of the ratio of 1.25 was accepted and used to interpolate widths at one kilometre intervals. These widths are plotted in Figure 4.6. The points shown are the **surveyed** water surface widths at the reference discharge.

To smooth this data a moving average over variable lengths was used until it appeared visually satisfactory for the purposes of the numerical model. The result of this exercise is shown in Figure 4.7, and the resulting widths are tabulated in Appendix C. Some runs of the unsteady flow model were done to assess the impact of this smoothing on the results. It was negligible.

4.4 Effective hydraulic roughness

This parameter is determined from the relations

$$Q = C^{*}A\sqrt{gRS}$$

in which $C^{*} = 2.5 \ln 12R/k$

and where Q is the discharge, C^{*} the conveyance (dimensionless Chezy coefficient), A the reach-averaged waterway area, g gravity, R the reach-averaged hydraulic radius and S the reach-averaged slope of the energy line (approximated here by the water surface slope). The effective hydraulic roughness is k.

The hydraulic roughness was first estimated for each surveyed reach on the basis of the reach-averaged geometric parameters A, R, and S and the known discharge on the day of survey. The values found are shown in Figure 4.8. As is evident, they displayed a large variation, with the most extreme values being the result of idiosyncrasies of the reach. For example, at Swede Creek, at the discharge on the day of survey, the reach was basically a pool upstream of some rapids, making it very difficult to define the slope accurately enough to get a good determination of roughness.

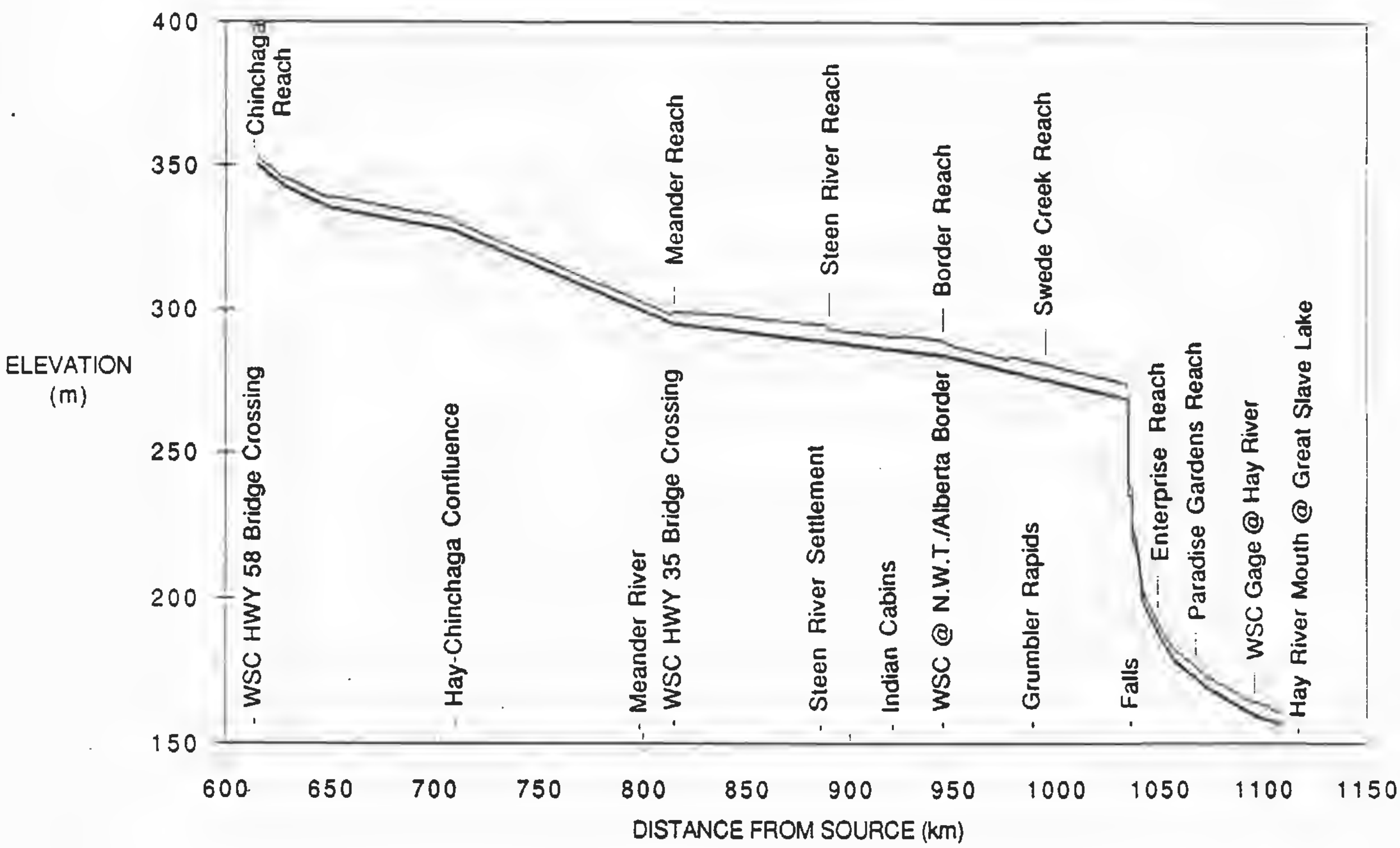


Figure 4.3. Longitudinal profile of the study reach. The lower line represents the bed, the upper line the water surface for the reference discharge.

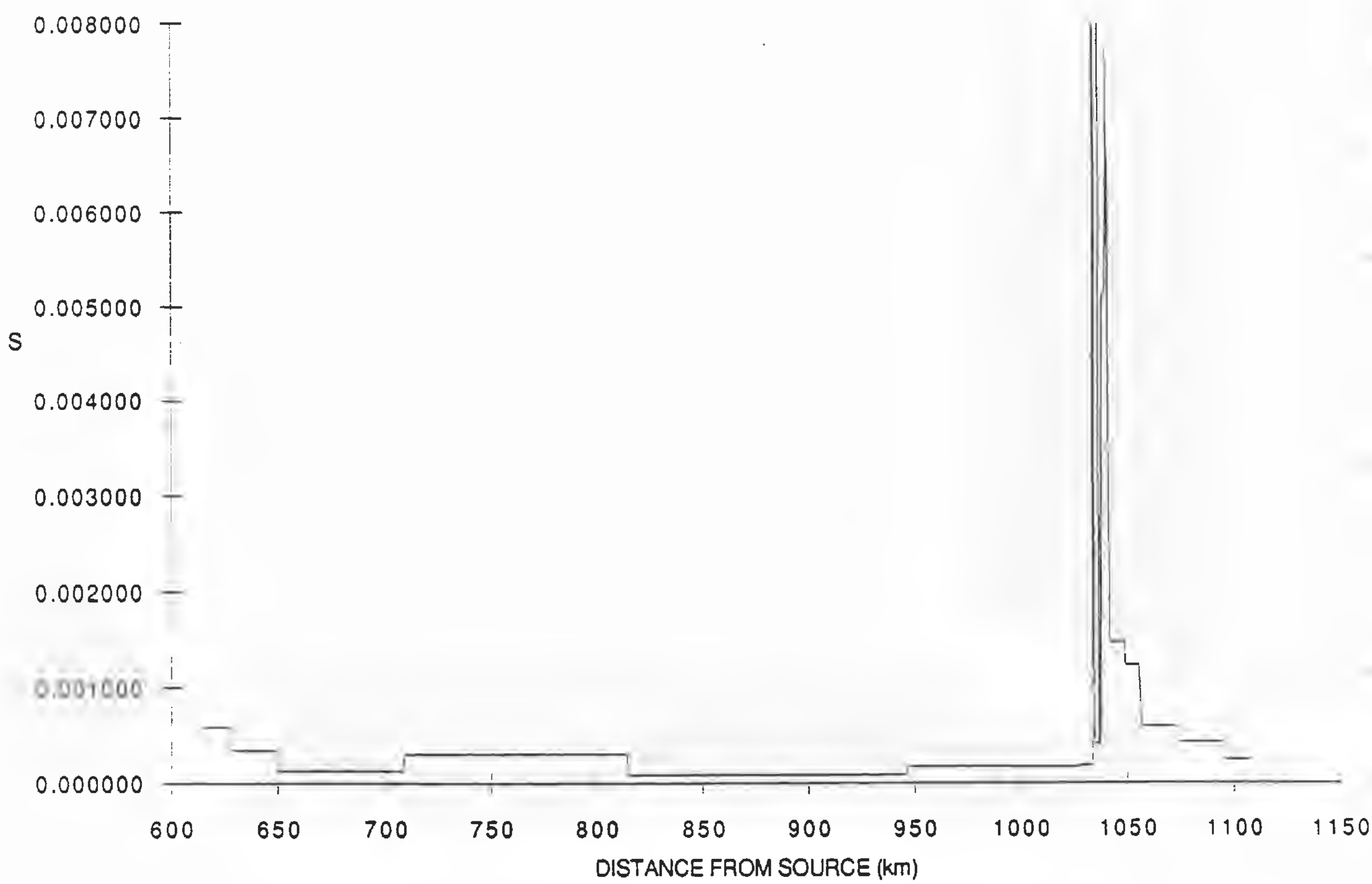


Figure 4.4 Slope variation along the study reach.

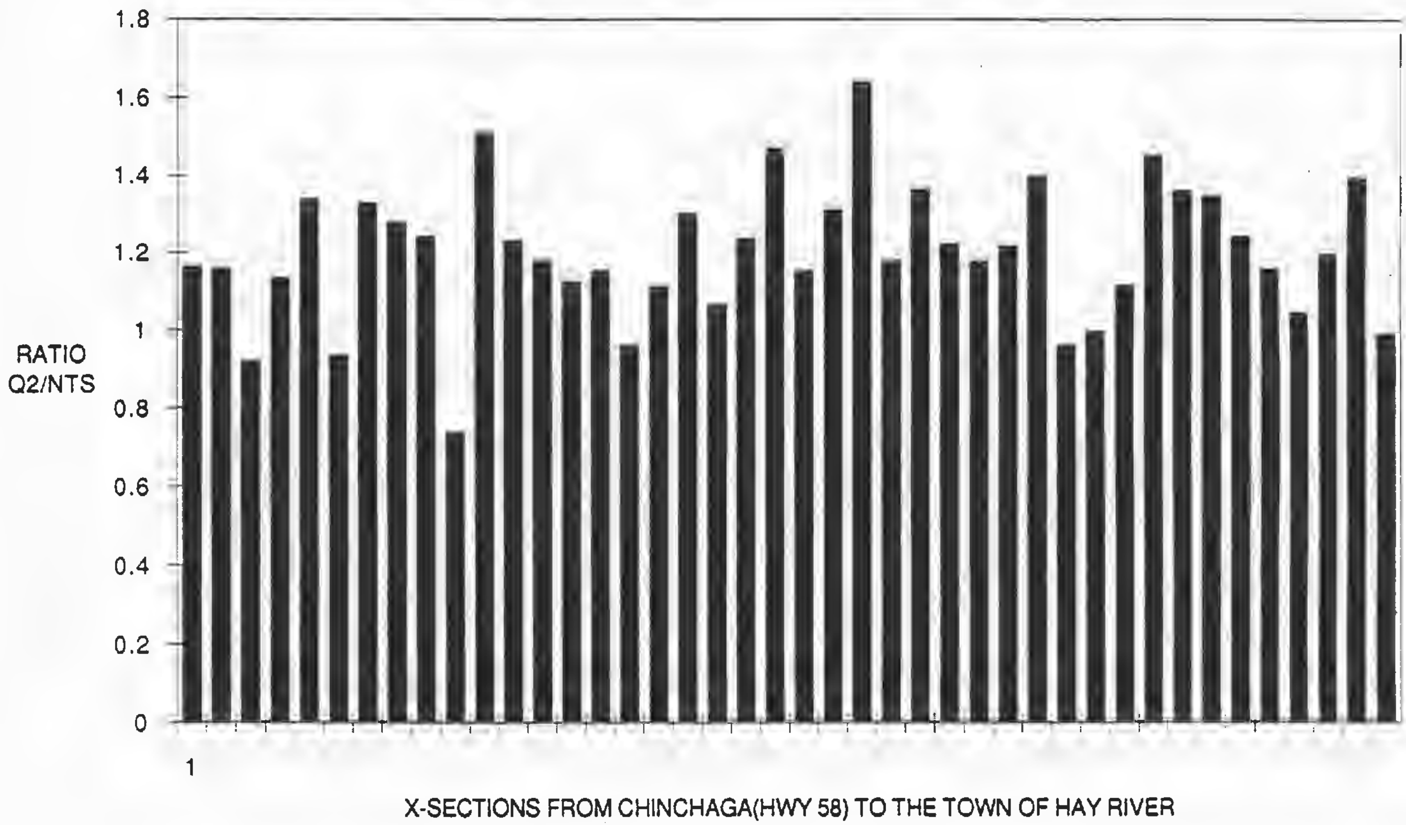


Figure 4.5. Plot of the ratio of surveyed water surface widths to those determined from 1: 50000 NTS maps.

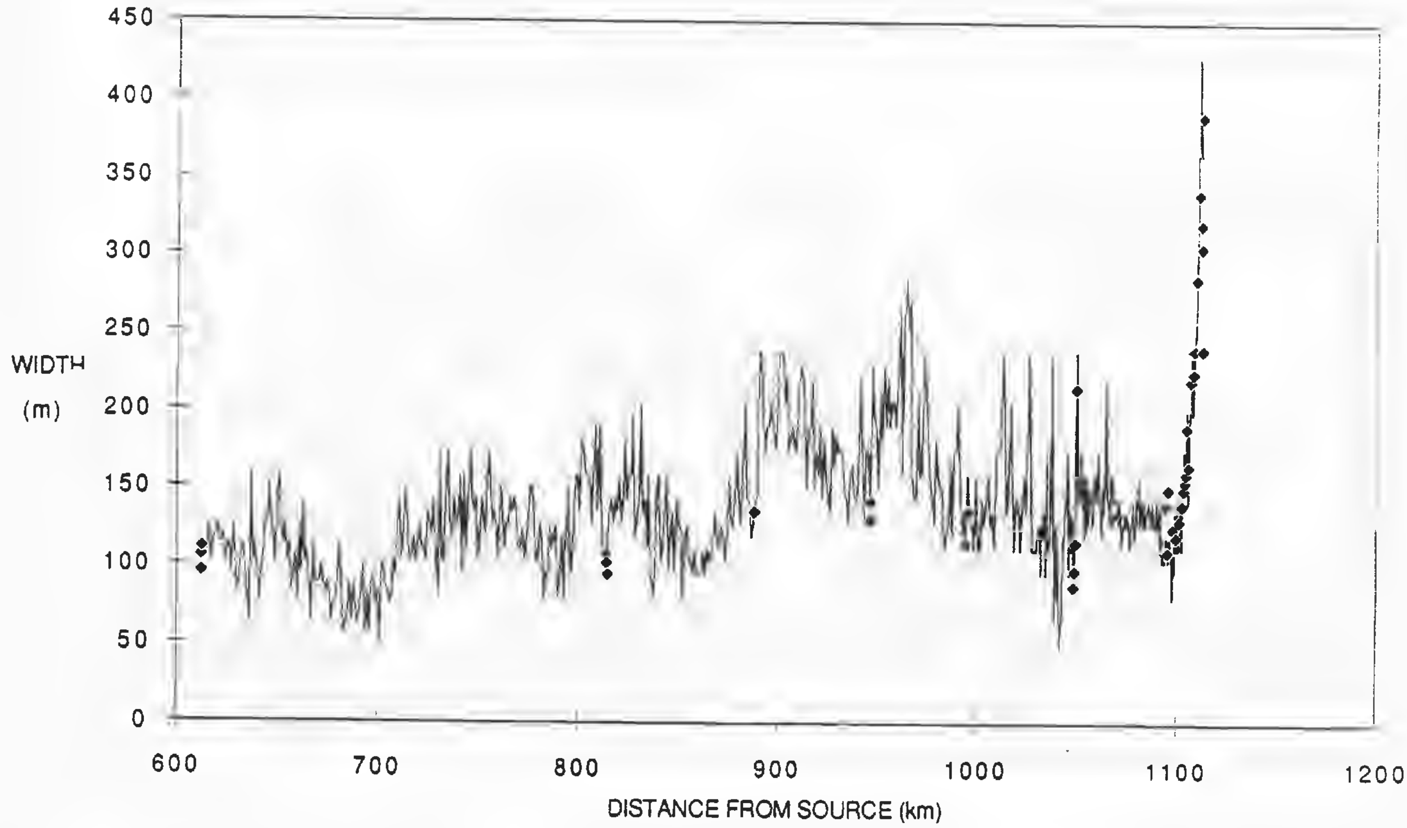


Figure 4.6. Variation in water surface width. The discrete points shown are the surveyed water surface widths at the reference discharge.

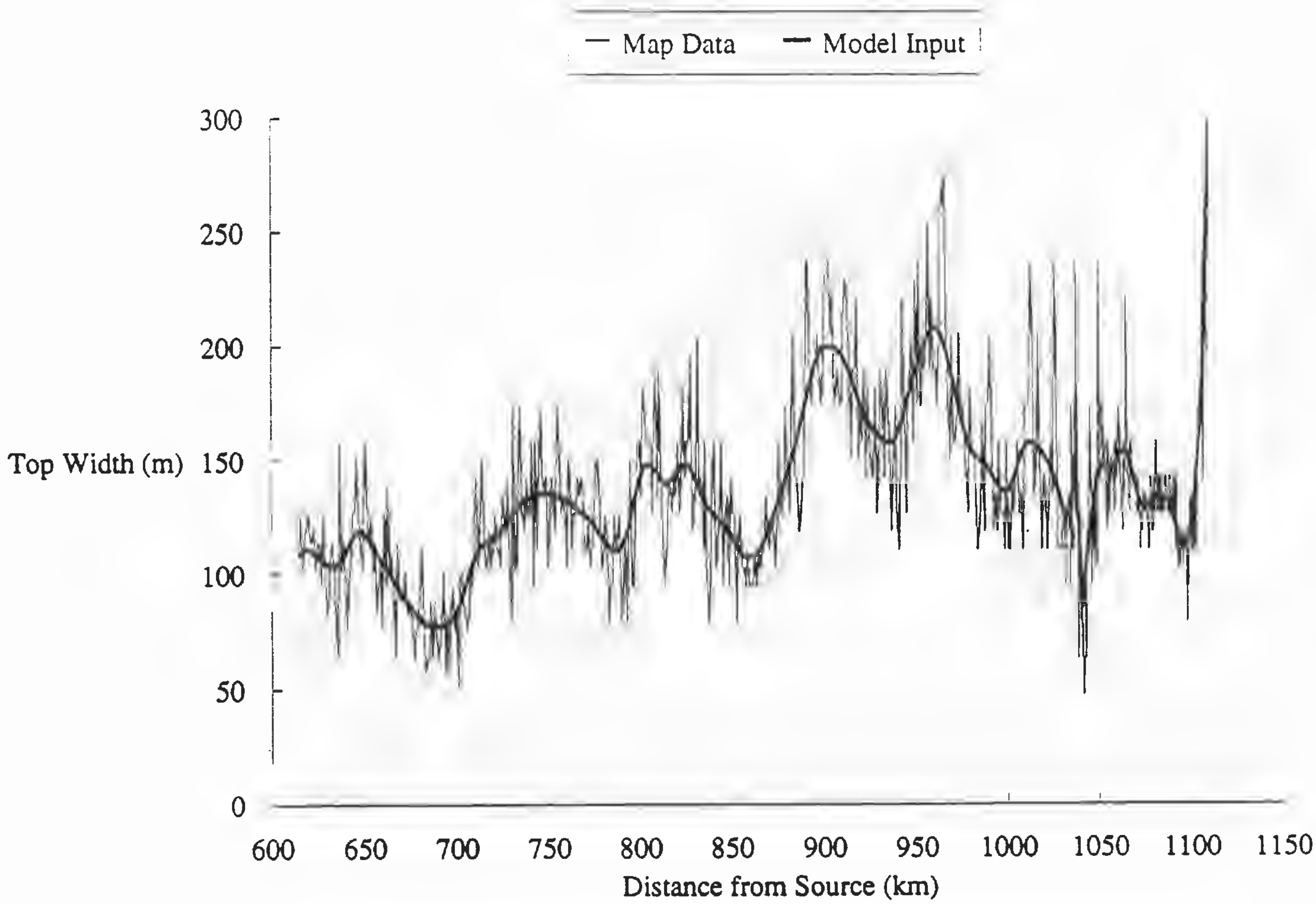


Figure 4.7. Variation in channel width of the Hay River and the smoothed distribution used in the numerical model.

Based on the values shown in Figure 4.8, and a general appraisal of the nature of the bed material (see Figure 3.1e, for example), a compromise value of 0.2 m was accepted as reasonable for the whole reach.

The impact of this 'rounding off' of the hydraulic roughness on the rating curves of the reaches is illustrated in Figure 4.9. This shows the rating curves for the Hay River at Hay River for the following circumstances: the roughness of 0.27 m calculated from the discharge on the day of survey; the compromise roughness of 0.2 m; and the rating curve measured by WSC over a large range of discharge. The latter corresponds to a roughness of 0.04 m, which is the value plotted in Figure 4.8. The quite different compromise value is responsible for only about 0.3 m difference in stage at the higher discharges, but is less at discharges of 500 - 1000 m^3/s which are more typical for this reach.

The above definitions of the variation in reference discharge, bed elevation, channel width and roughness of an equivalent rectangular channel allow application of the unsteady flow modelling described in the next section to estimate the consequences of ice jam failure on the variation of discharge in Hay River.

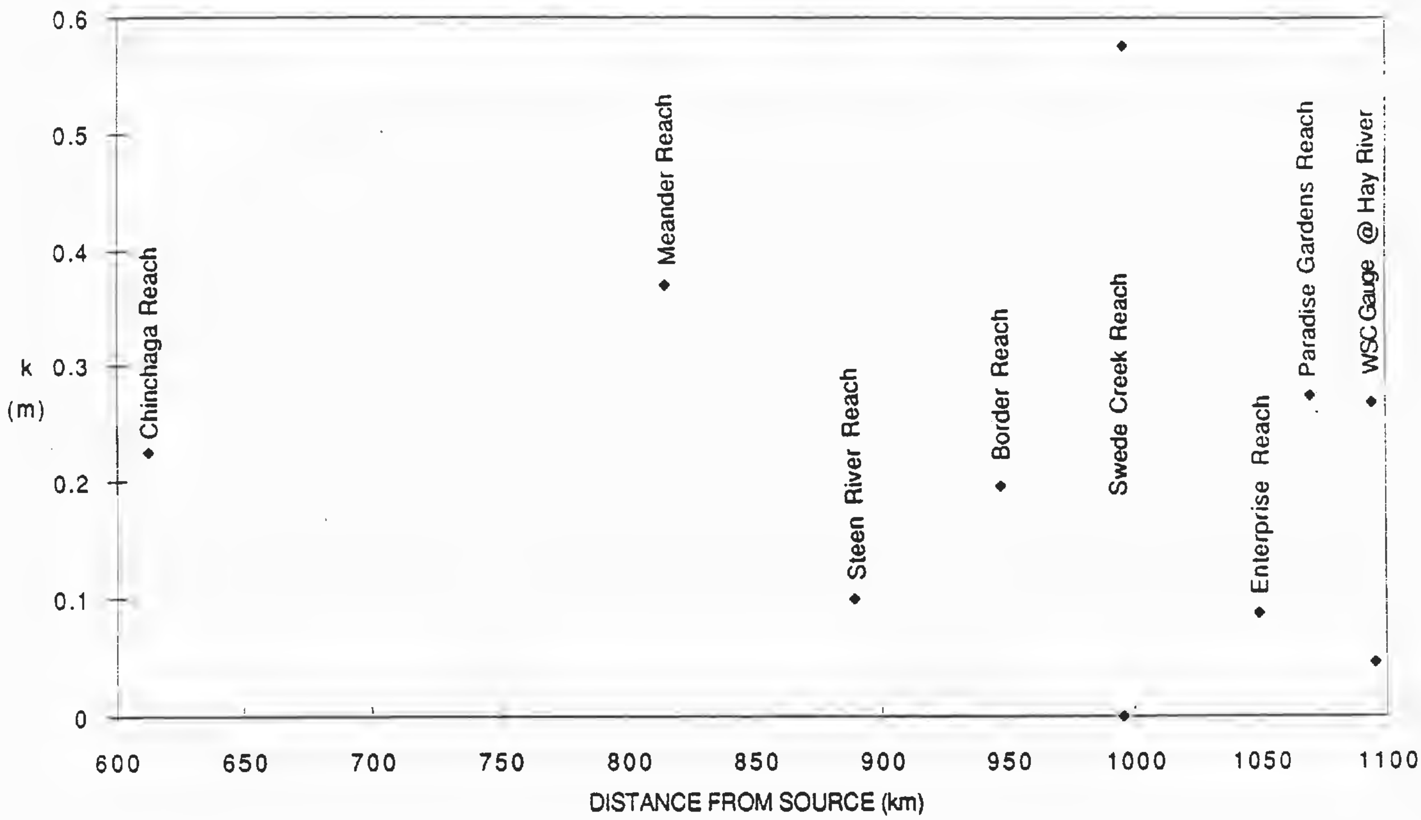


Figure 4.8. Variation in hydraulic roughness along the reach.

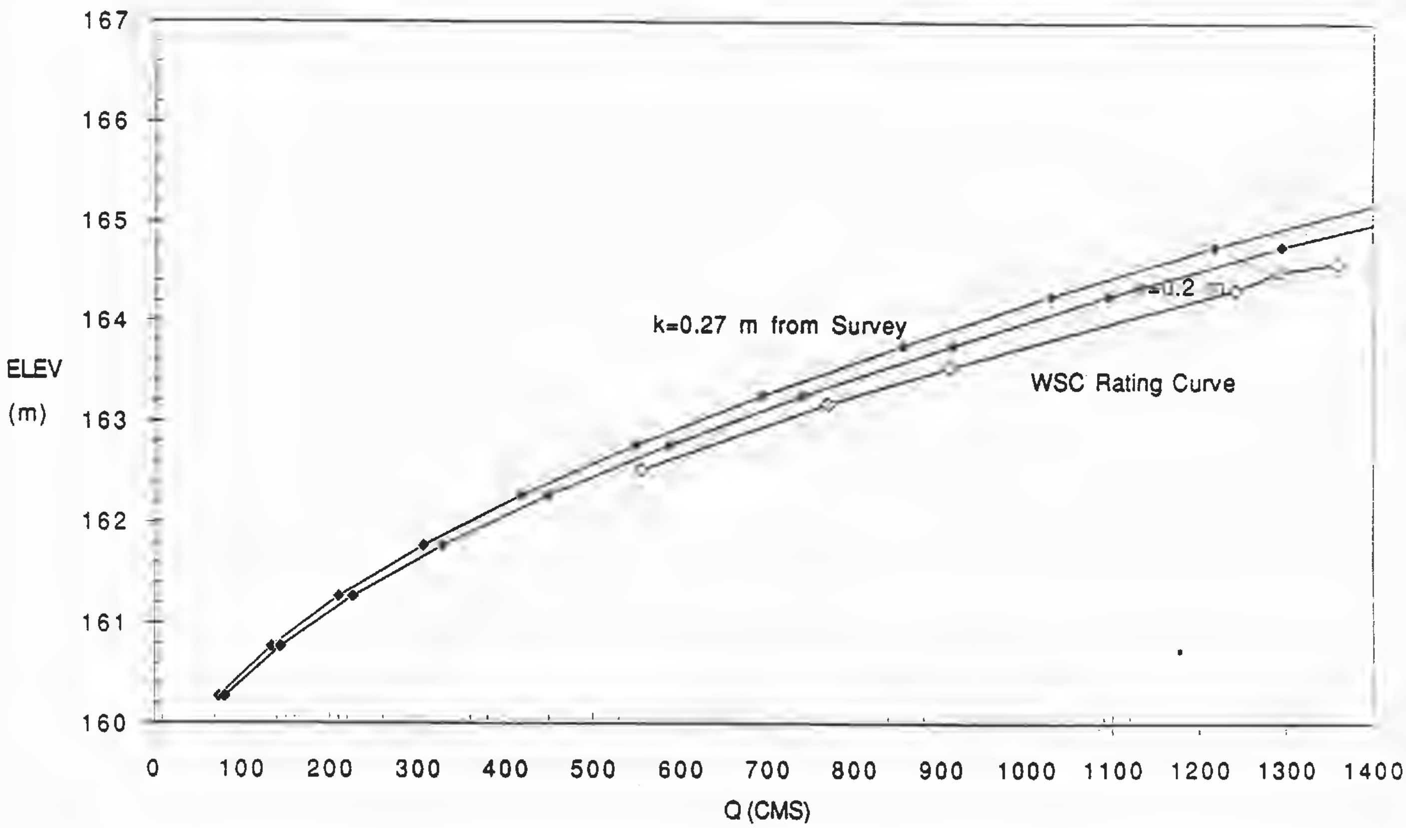


Figure 4.9 Variation in rating curve for the Hay River at Hay River caused by a change in the hydraulic roughness.

5.0 UNSTEADY FLOW MODELLING

5.1 Basic objectives

The purpose of this analysis was to quantify the effects of the sudden release of an upstream ice jam on discharges in the Hay River at the Town of Hay River. Specifically, the objective was to be able to determine the magnitude and timing of the potential maximum water level within the town, given the (carrier) discharge in the Hay River and the location and length of an upstream ice jam.

To achieve this, a numerical model of the equations of one-dimensional unsteady open channel flow was used to route the surges resulting from a sudden jam release down the Hay River to the town (Hicks, 1990). To encompass a wide range of ice jam and flow scenarios a number of jam lengths, locations and carrier discharges were considered in the analysis. Based on this analysis, a set of influence lines was developed that related the magnitude and timing of the peak flow to the three input variables.

5.2 Assumptions for ice jam simulation

A number of simplifying assumptions were used in conducting the unsteady flow analysis. These included not only an approximation of the channel geometry and the initial steady flow profiles, but also of the initial jam shape and failure mechanism.

As described earlier, a rectangular cross section was used to approximate the channel geometry in this analysis. To establish the initial conditions for each unsteady flow simulation, a steady varied flow profile was required over the entire study reach. This was determined using the same numerical model employed for the unsteady flow calculation.

After the steady flow profile was determined throughout the reach, an ice jam was superimposed on the flow at a selected location. As illustrated in Figure 5.1, the ice jam was assumed to have a toe length equal to the grid spacing of the numerical model which was generally a kilometre or less. The gradually varied (M1) flow profile upstream of the jam was approximated by projecting a horizontal line back from the jam to an intersection with the upstream water level. The flow depth to the phreatic surface within the jam was determined based on an equilibrium section of a fully developed jam, using the following relationship developed from the approach suggested by Beltaos (1983):

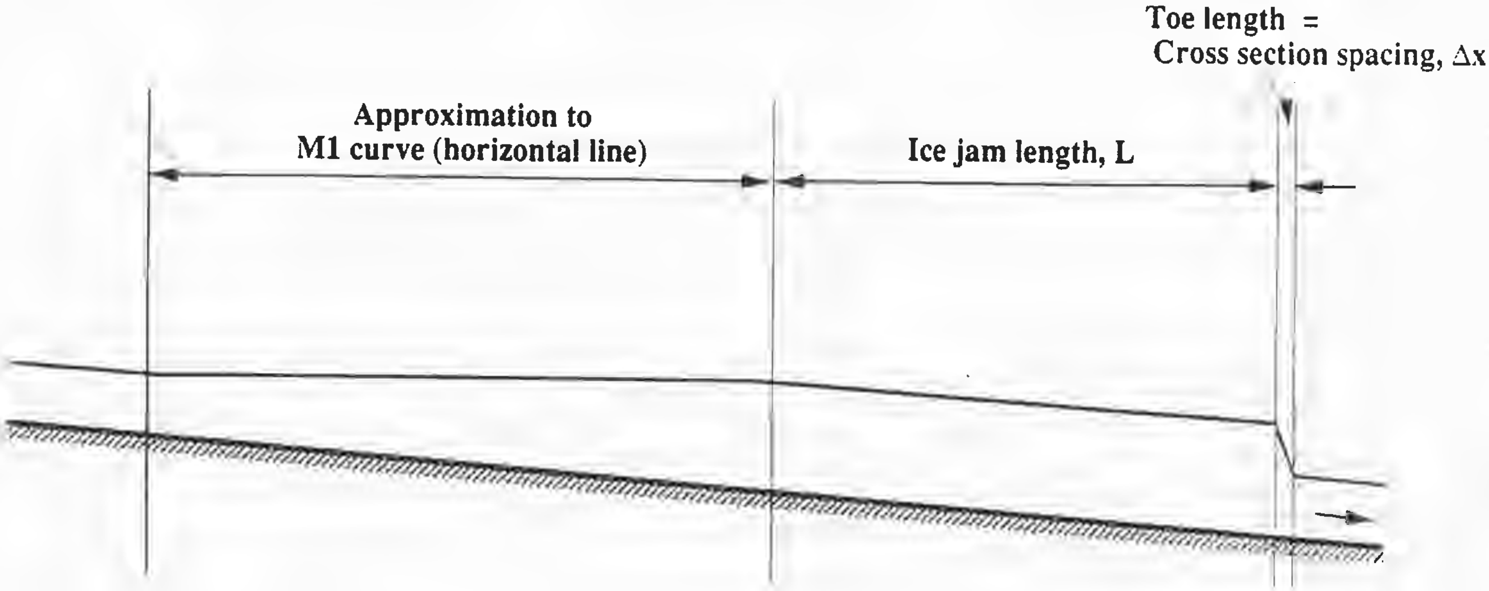


Figure 5.1. Schematic of the ice jam configuration assumed in the numerical analysis.

$$\eta = 0.38\,\zeta + \frac{5.75}{\mu}\left\{1 + \sqrt{1 + 0.07\,\mu\,\zeta\left(\frac{k_i}{k}\right)^{1/4}}\right\} \qquad [5.1]$$

where

$$\eta \equiv \frac{d}{SB} \qquad [5.2]$$

and

$$\zeta \equiv \frac{\left(qk^{1/6}/\sqrt{gS}\right)^{3/5}}{SB} \qquad [5.3]$$

in which μ is an internal friction-porosity coefficient (taken as 1.0 for this investigation), d the depth to the phreatic surface, S the slope of the energy line (equal to the channel slope in an equilibrium section), B the pack width, and q the discharge per unit width of channel. The composite roughness k is based on the ice roughness, k_i and the bed roughness, k_b, such that:

$$k = \left(\frac{k_i^4 + k_b^4}{2}\right)^{1/4} \qquad [5.4]$$

A constant value of 2 m was used for the hydraulic roughness of the pack for all runs conducted in this analysis. For the bed, a constant value of 0.2 m was used throughout the study reach as discussed in the previous section. The solid ice cover roughness was taken equal to that of the bed.

The release of the ice jam was assumed to be instantaneous, with the consequent acceleration of the water occurring over its entire length. In addition, it was assumed the ice in the pack followed the water and therefore had no resistance effect on the flow after the time of release. Finally, it was assumed that the entire reach downstream of the toe of the jam was either open (no ice cover), giving a 'worst case' scenario for the speed and magnitude of the peak discharge, or that an ice cover was in place over the entire reach downstream of the jam, which remained after passage of the surge. The intent of the latter was to give an indication of the range of possibilities and to assess the sensitivity of the results to the presence of an ice cover.

5.3 Finite element model for one-dimensional unsteady flow

5.3.1 Equations

The St. Venant equations describe conservation of mass and momentum for one dimensional, unsteady flow in an open channel. These conservation laws may be written as:

$$\frac{\partial H}{\partial t} + \frac{\partial (UH)}{\partial x} = 0 \qquad [5.5]$$

$$\frac{\partial U}{\partial t} + U\frac{\partial U}{\partial x} + g\frac{\partial H}{\partial x} = g(S_o - S_f) \qquad [5.6]$$

where H represents the water surface elevation, U the cross sectionally averaged longitudinal velocity, g the acceleration due to gravity, S_o the longitudinal bed slope; and S_f the longitudinal friction slope. The longitudinal and temporal coordinates are represented by x and t respectively.

These equations represent the complete equations for a dynamic wave analysis, though a number of simplifying assumptions are required in their derivation. For example, it is assumed that a hydrostatic pressure distribution exists at each cross section; that the vertical acceleration is negligible compared to the horizontal acceleration of the flow; that vertical shear stresses are negligible; that the velocity distribution at each cross section is uniform; and that the water surface elevation does not vary across the channel cross section. All of these assumptions are reasonable in the situation of concern. In addition, it was assumed that no lateral inflow, such as would be contributed by tributary or overland flow, occurred, so that the carrier discharge was taken as constant over the whole reach below any jam site. While less defensible for jams well above the town, as Figure 4.2 shows, the error introduced is small compared to the other approximations made, such as that of the roughness variation.

5.3.2 Solution technique

The St. Venant equations are a system of non-linear partial differential equations which may be solved by a number of numerical methods, generally classified as either finite difference methods or finite element methods. Of the finite difference models, the four-point implicit scheme as presented by Amien (1968) has been used most frequently to solve these equations. However, the underlying consistency and generality of the finite element method provides a number of advantages over the finite difference approach that are particularly suited to this study.

One of the most restrictive limitations of finite difference methods is that they require separate computational algorithms for subcritical and supercritical flow. Therefore, in a case such as the Hay River, which involves alternating reaches of subcritical and supercritical flow, each reach must be modelled separately and then the final solutions pieced together. This would be a difficult task for a steady flow problem; for unsteady flow it becomes logistically insurmountable. In contrast, the finite element method handles subcritical and supercritical flow in the same way, so that it can solve for both flow regimes simultaneously. Furthermore, because the finite element solution of the equations involve an integration of the St. Venant equations over the solution domain, accurate solutions may be obtained even when some of the underlying assumptions are violated. Integration of the equations eliminates the requirement of a continuous derivative thus allowing the model to provide accurate solutions even in the vicinity of such discontinuities as hydraulic jumps.

Another advantage of the finite element method is that, unlike the finite difference method, non-uniform spacings between channel sections are easily handled. This means that sections can be concentrated in areas where gradients are large (such as in the vicinity of the falls), while fewer sections may be used in areas where the geometry is more uniform, thus allowing for optimization of computational effort. This is particularly valuable in large simulations such as those described herein, where the study reach extends for hundreds of kilometres and the simulation involves thousands of time steps.

Hence, for this study a new finite element model, developed in the Department of Civil Engineering of the University of Alberta (Hicks, 1990), was employed for the flow analysis. This model has been found to provide consistently more stable and accurate solution to both steady and unsteady open channel flow problems over a wide range of situations than either other finite element models or the four-point implicit finite difference scheme.

5.4 Adaptation of the numerical model to Hay River

5.4.1 Spatial discretization

The study reach for the unsteady flow analysis extended from the Highway 58 crossing of the Chinchaga River (615 km) to the town of Hay River (1108 km). Through the upstream portion of the study reach a section spacing of 1 km was used. This spacing was consistently decreased (15% decrements) to 200 m just upstream of the steep reach in the vicinity of the falls (between 1027 and 1032.5 km) and then increased again to 500 m further downstream (1045 km). A total of 617 cross sections were used in the analysis. Table C1 in Appendix C provides a listing of the data for the input sections. The determination of these values is described in Section 4. It is noted the vertical drop of the Falls was replaced by a steep reach defined by the given upstream and downstream bed elevations and the numerical solution grid distance of 200 m at this location.

5.4.2 Boundary conditions

The study domain encompassed a number of subcritical and supercritical reaches. However, the flow remained subcritical at both the upstream and downstream extremities. Therefore, one upstream and one downstream boundary condition were required for the simulation. The upstream boundary condition was taken as the carrier discharge, while at the downstream end a constant water level of 156.9 m, representing the water level of Great Slave Lake, was used.

5.4.3 Range of input conditions

Three jam lengths - 5,10, and 50 km - were used in the analysis. The ice jam locations used were at Meander River (800 km), Indian Cabins (920 km), and at Paradise Gardens (1070 km). Each case was run for three carrier discharges - 200, 500 and 900 m^3/s. This involved a total of 29 unsteady flow simulations, each covering a duration of 24 to 60 hours of prototype time. An additional 12 runs were conducted to examine the effects of an ice cover on the surge propagation and to evaluate intermediate jam lengths of 20 km at some locations.

5.5 Typical results

Figure 5.2 illustrates the variation in discharge at the Town of Hay River generated by the sudden release of a 20 km jam located at 1095.6 km (Hay River at Hay River WSC gauge) with a carrier discharge of 500 m^3/s and open water downstream of the jam. The hydrograph at the toe of the jam is shown as well. It is evident that such a jam failure would result in a 170% increase in discharge at the town approximately 1.6 hours after the jam release. The irregularities evident in the hydrograph at the toe of the jam are due to small unsteady flow waves generated by the irregular assumed jam profile. These spurious effects are lost a short distance downstream of the jam. Figure 5.3 illustrates the unsteady flow rating curves at the same sites. Also shown is the steady state curve of Figure 4.9, which was derived quite independently. Theoretically it should pass close to the apex of the unsteady flow rating curve, which it does. It is also to be noted that the unsteady flow effects on

the rating curve some distance from the toe are minor. Figure 5.4 and 5.5 illustrate the effects of an ice cover for the same situation. It is evident that the presence of an ice cover on the reach downstream of the jam can have a significant damping effect on the surge, with a much lower peak discharge and later time of arrival.

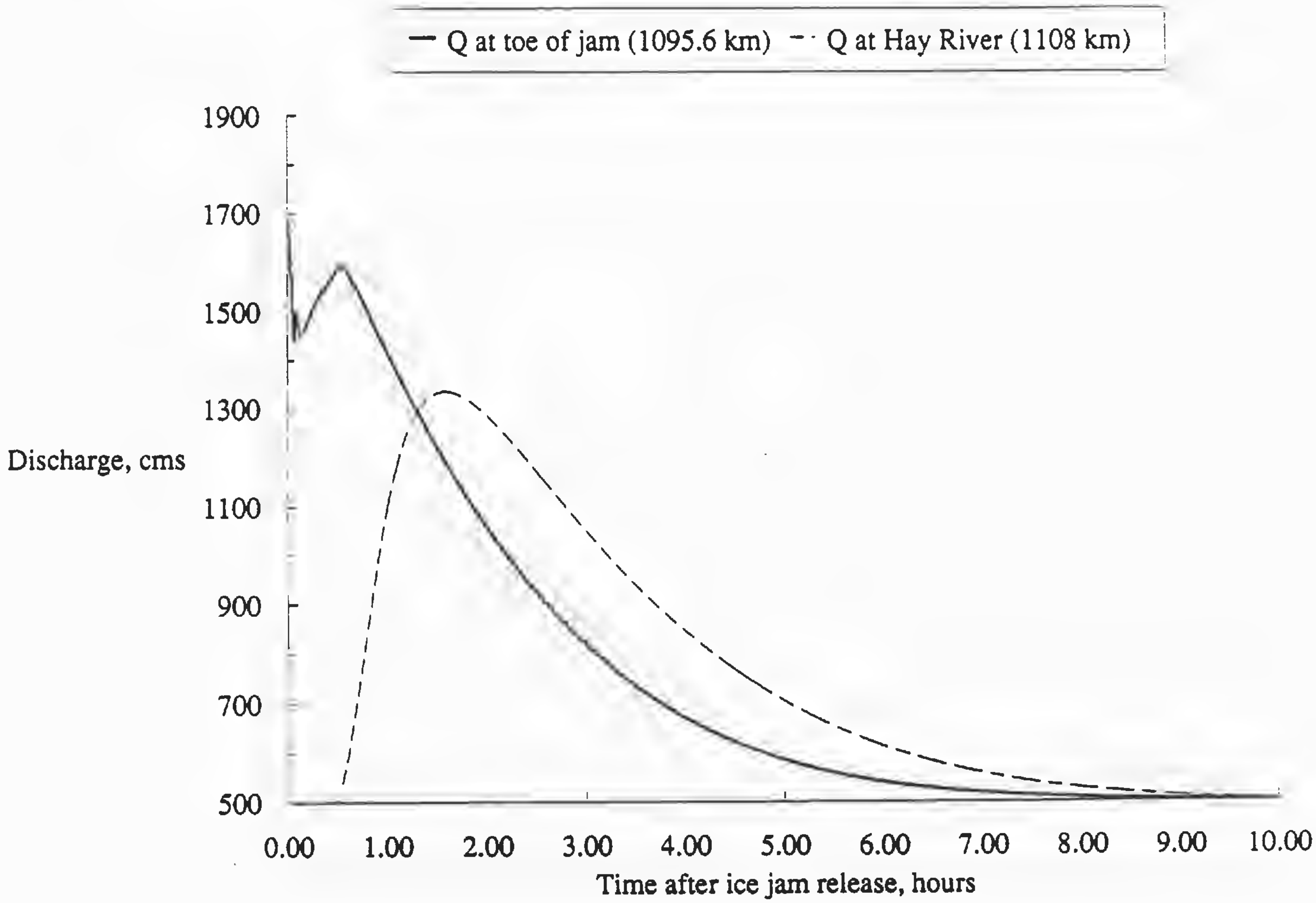


Figure 5.2. Hydrographs generated by 20 km ice jam release at 1096.6 km for open water conditions downstream of the jam and a carrier discharge of 500 m^3/s.

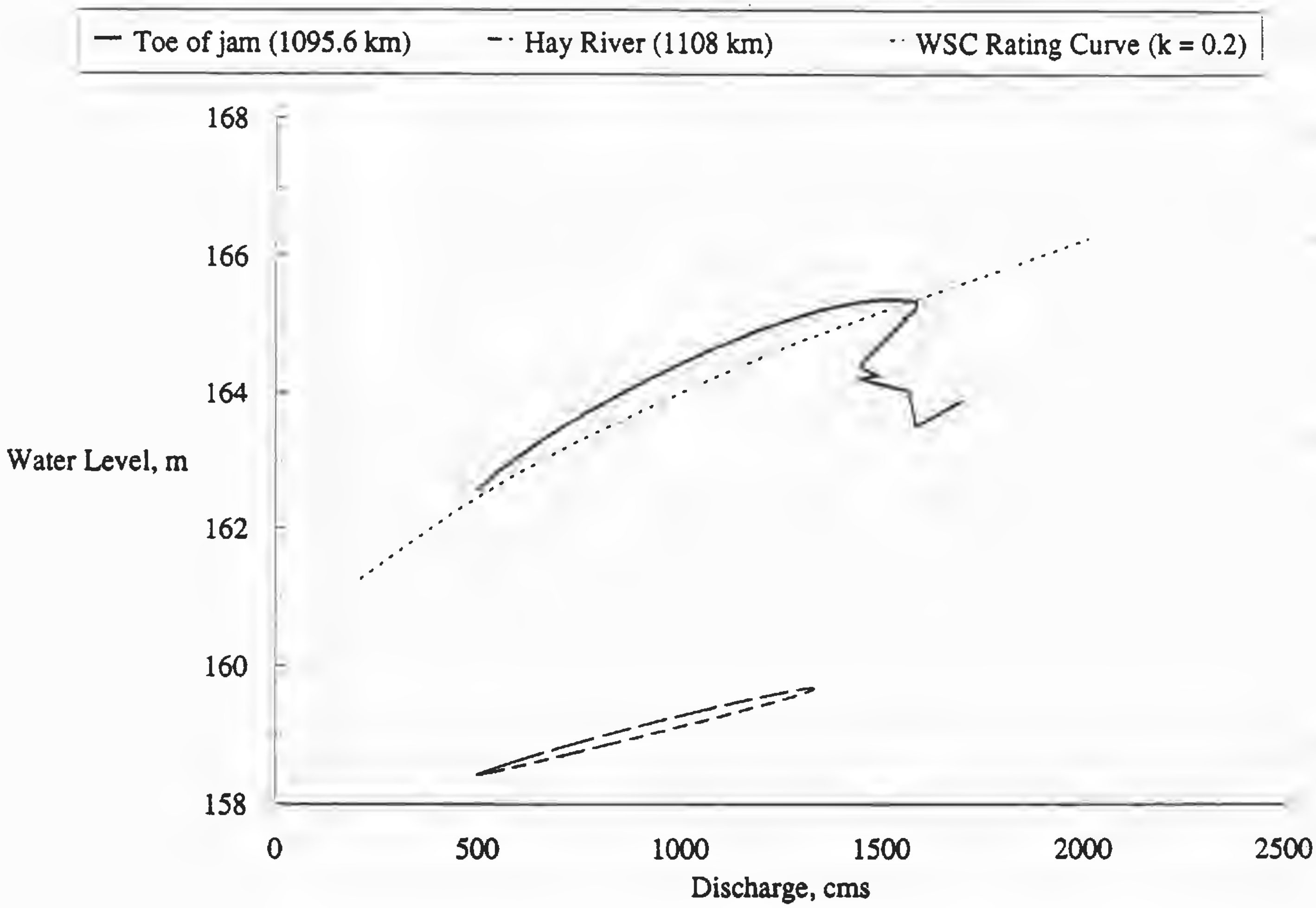


Figure 5.3. Variations in discharge with water level for open water conditions generated by the release of a 20 km long ice jam at 1095.6 km with a carrier discharge of 500 m^3/s.

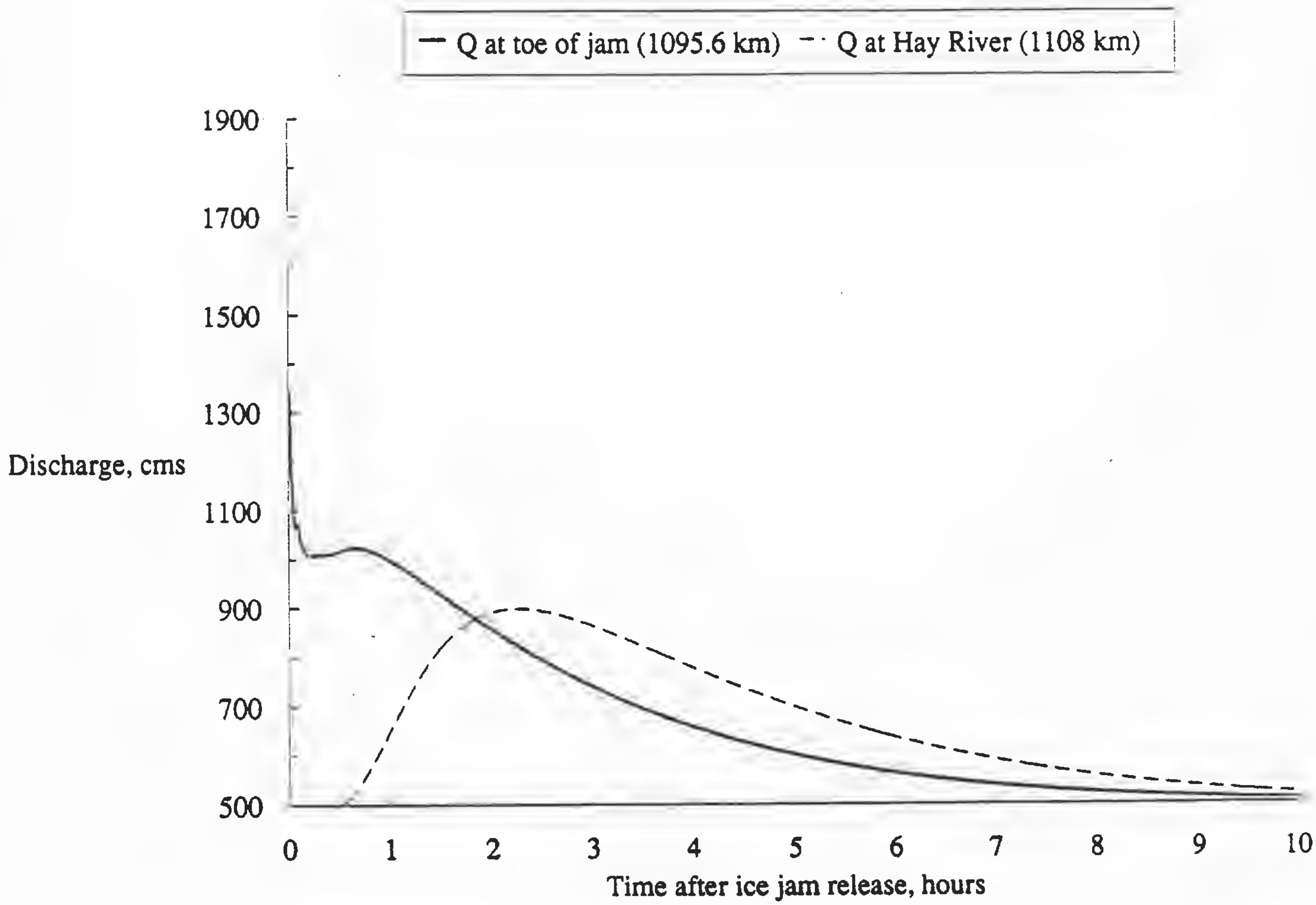


Figure 5.4. Hydrographs for ice-covered conditions following release of a 20 km long jam situated at km 1095.6 with a carrier discharge of 500 m^3/s.

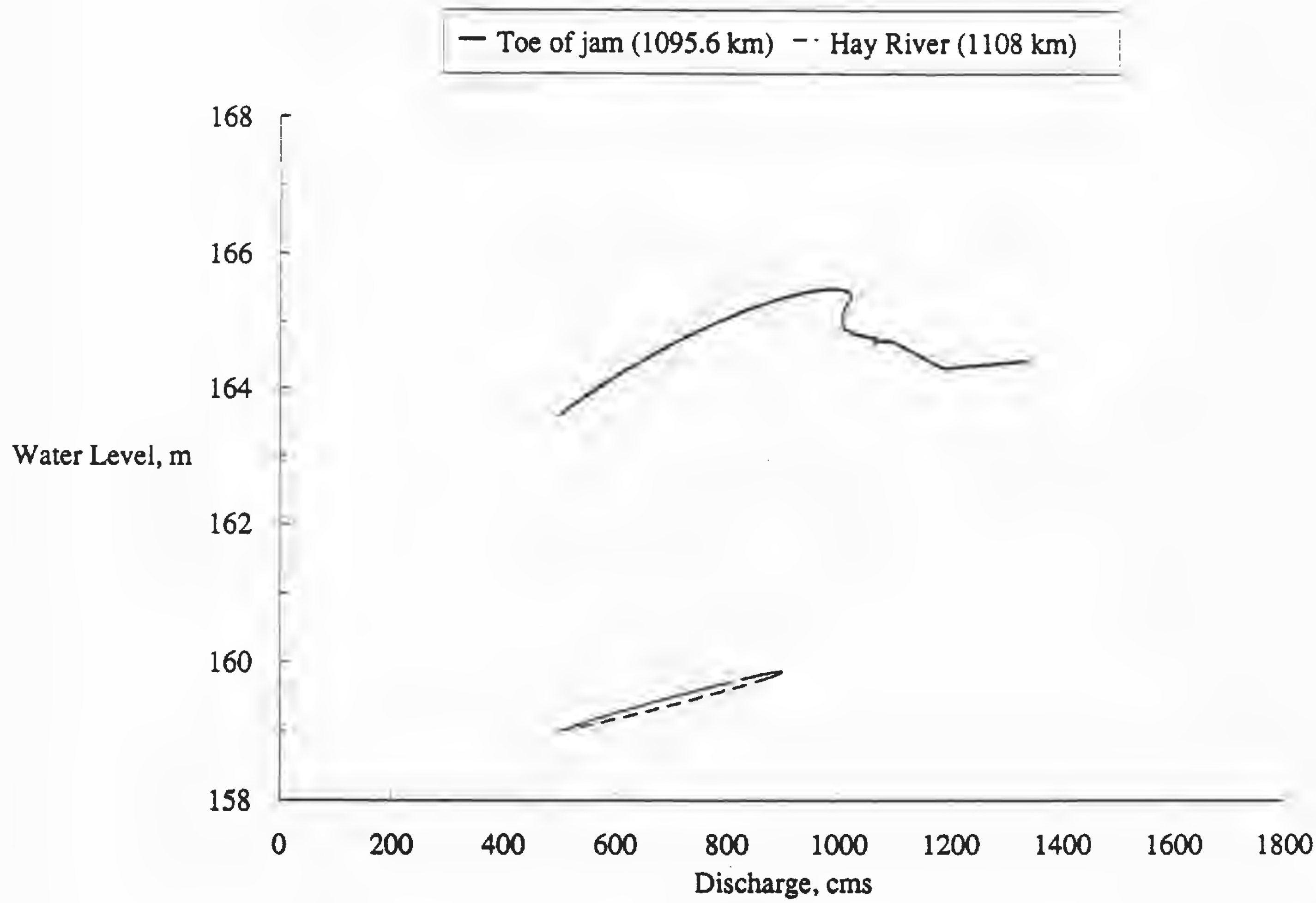


Figure 5.5. Rating curves for ice-covered conditions following release of a 20 km long jam situated at km 1095.6 with a carrier discharge of 500 m^3/s.

5.6 Comparison with Williamson results

As stated in Section 5.3.2, the four point implicit scheme has been used most frequently to solve these equations. However, due to the limitations of the finite difference approach, this method could not be applied to the Hay River problem. Because Williamson (1989) has used the Amien scheme to develop a series of non-dimensional curves for this type of analysis, his results provide a valuable comparison for this study. This comparison was carried out using simpler versions of the channel geometry to which Williamson's results could be applied and suggested that for most jam locations and lengths, the peak discharges predicted by these two methods were not significantly different. However, the FEM model used in this study indicated that jams located near the town of Hay River tended to cause much higher surge discharges at the town than would be predicted by the Amien scheme. These would be very dynamic events and it is expected that the dissipative nature of the Amien model would therefore tend to predict lower peak discharges.

6.0 INFLUENCE LINES FOR SURGES CAUSED BY ICE JAM RELEASE

6.1 Variation of the discharge at Hay River

Figure 6.1 shows the increase in discharge at Hay River (ΔQ) as a function of jam length (L), distance from Hay River (Δx) and carrier discharge at Hay River (Q_o). The results are based on the assumption of open water downstream of the jam. Attempts to derive a universal curve on the basis of these primary variables were not successful. The best that could be achieved were the lines indicated in Figure 6.1. These indicate that the downstream effects of an ice jam release depend on the jam location in more ways than just its distance from Hay River. It is felt that the significant effect left out of this simple dimensional analysis was that of the different volumes stored by jams of a given length in the different reaches, and wave dispersion by the characteristics of various channel segments, particularly the slope.

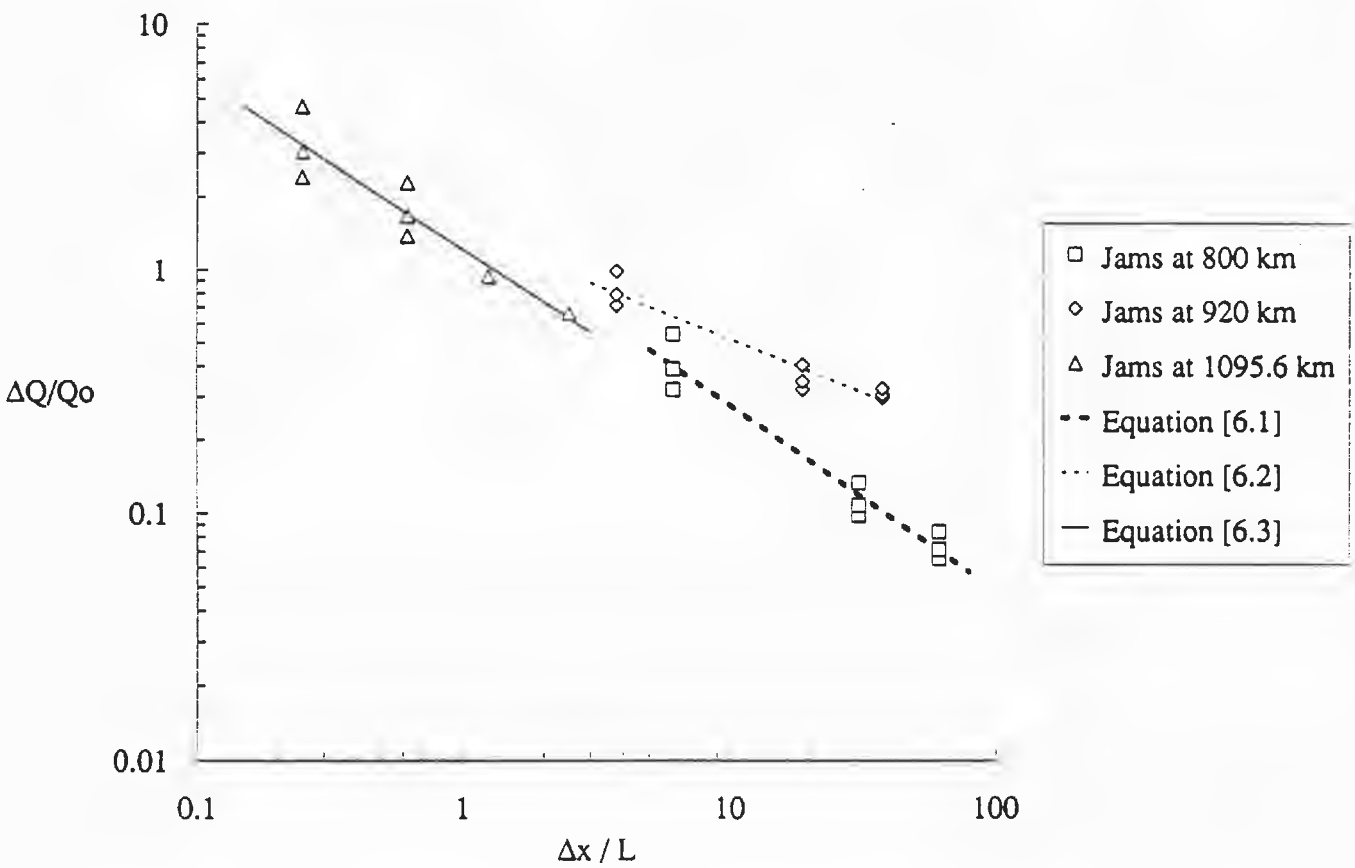


Figure 6.1. Discharge change as a function of jam location and length.

It is therefore recommended that the reach be split on the basis of slope, and that, for locations above about km 820, the following relation should be used

$$\frac{\Delta Q}{Q_0} = 1.61 \left(\frac{\Delta x}{L}\right)^{-0.7570} \qquad [6.1]$$

while for the reach below km 820, but above the Falls, Equation 6.2 is appropriate

$$\frac{\Delta Q}{Q_0} = 1.44 \left(\frac{\Delta x}{L}\right)^{-0.4391} \qquad [6.2]$$

and that below the Falls, Equation 6.3 is valid.

$$\frac{\Delta Q}{Q_0} = 1.22 \left(\frac{\Delta x}{L}\right)^{-0.7074} \qquad [6.3]$$

Figure 6.2 gives a comparison, for a carrier discharge of 500 m^3/s, between the results obtained assuming open water downstream of the jam and those obtained with an ice cover in place. As pointed out in an earlier section, an ice cover causes a substantial reduction in the magnitude of the increase in discharge in Hay River. The equations for the ice cover cases comparable to the above open water equations are Equations 6.4, 6.5, and 6.6.

$$\frac{\Delta Q}{Q_0} = 0.742 \left(\frac{\Delta x}{L}\right)^{-0.8013} \qquad [6.4]$$

$$\frac{\Delta Q}{Q_0} = 0.616 \left(\frac{\Delta x}{L}\right)^{-0.5590} \qquad [6.5]$$

$$\frac{\Delta Q}{Q_0} = 0.589 \left(\frac{\Delta x}{L}\right)^{-0.7043} \qquad [6.6]$$

6.2 Timing of surge arrival at Hay River

Figure 6.3 shows the variation in the time of arrival, (Δt), after the ice jam release of the peak flow at Hay River with jam length and distance to the jam. A distinct change in trend is evident. It is believed this is due to the gradual change from a dynamic wave to an essentially kinematic one. For $\Delta x/L < 1.0$, Equation 6.7 applies whereas for $\Delta x/L \geq 1.0$ Equation 6.8 is applicable.

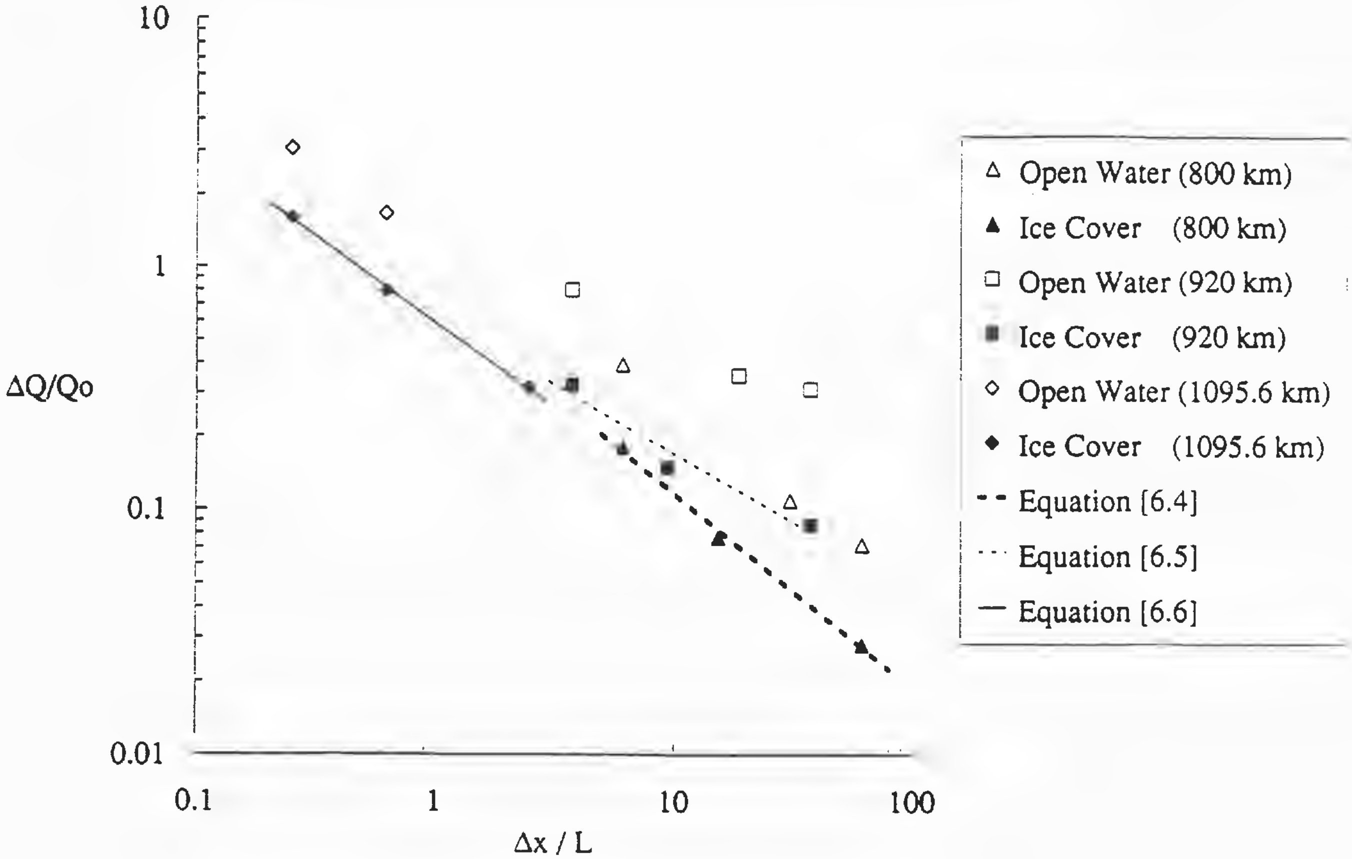


Figure 6.2. Comparison of the discharge change between open water and ice cover downstream of the jam.

$$\frac{\Delta t\, Q_0^{1/3}}{\Delta x} = 0.771 \left(\frac{\Delta x}{L}\right)^{-0.5918} \quad [6.7]$$

$$\frac{\Delta t\, Q_0^{1/3}}{\Delta x} = 0.796 \left(\frac{\Delta x}{L}\right)^{0.1123} \quad [6.8]$$

where Δx and L are in km, Q_0 is in m^3/s and Δt is in hours. Beyond the above distinction, however, the time lines do not display the stratification evident in the discharge plots.

These time parameters are dimensional. They were derived in dimensionless form but, on the basis that g is constant and C_* can be assumed approximately constant, this more direct form was used.

As might be expected, Equation 6.8 is almost independent of the jam length L. Accepting that it is independent of L, a new regression was run to yield

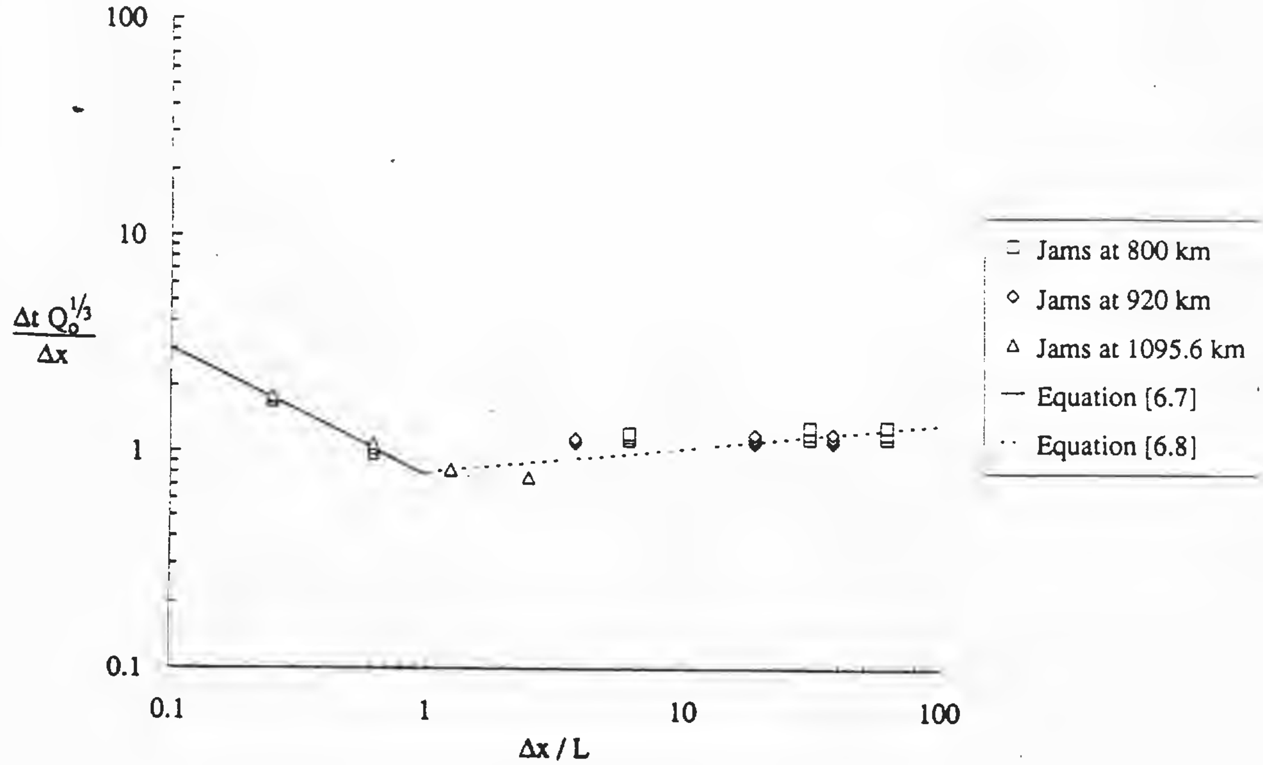


Figure 6.3 Variation in arrival time of surge released by ice jam failure.

$$\frac{\Delta t\, Q_0^{1/3}}{\Delta x} = 0.93 \qquad [6.9]$$

This can be rearranged to

$$V_W \approx 0.3\, Q_o^{1/3} \qquad [6.10]$$

where V_W is the velocity of the river wave in m/s. It is noted that while this velocity is surprisingly high, it is still lower than the observed velocity of the 1989 break-up front (Gerard and Jasek, 1989).

Figure 6.4 gives a comparison between the results obtained assuming open water downstream of the jam and those obtained with an ice cover in place, in both cases for a carrier discharge of 500 m^3/s. The ice cover slightly increases the time for the peak to arrive in Hay River but this small difference was not considered significant given the other approximations that were made in the study.

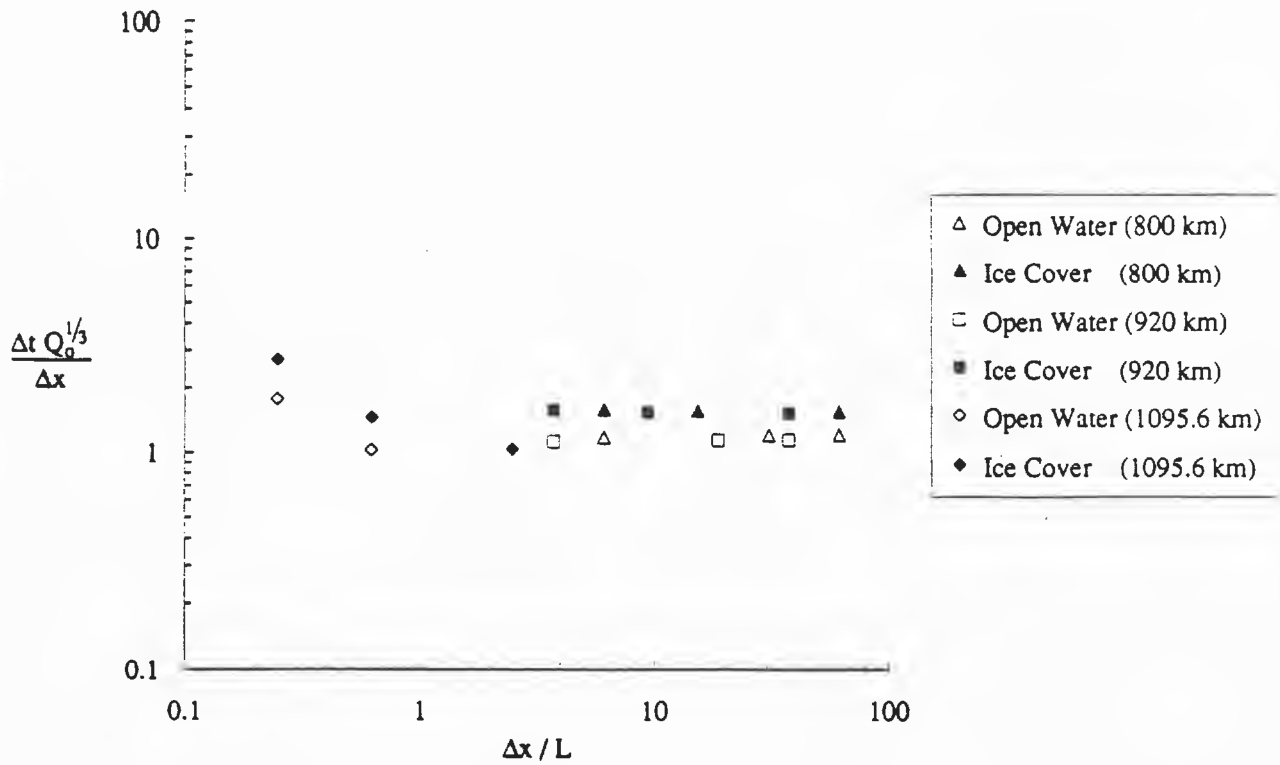


Figure 6.4. Comparison of arrival time between open water and ice-covered cases.

6.3 Comparison with field data

Unfortunately there is no definitive field data for surges on the Hay River. However, the dramatic events of 1985, which were clearly related to surge action, provide some basis for assessing the unsteady flow analysis.

On May 4, 1985, some 3 days before the second, and destructive, ice run in the West Channel, the ice jam pack extended from the mouth of the West Channel upstream to Paradise Gardens (Wedel, 1988). On May 5, sometime prior to 0700 h, a large jam at Indian Cabins was reported to have broken. Assuming the Indian Cabins jam was some 20 km long and the carrier discharge was about 1000 m^3/s on May 5 (Wedel, 1988), from Equations 6.2, 6.5 and 6.8 the surge released by this failure would have reached Hay River some 20 h later, and caused the discharge to increase between about 20 and 50 %, depending on the assumed downstream conditions. This indicates the peak discharge in Hay River should have been between 1200 and 1500 m^3/s. Given that at least half the distance between the jam and Hay River was covered with an ice accumulation considerably rougher than assumed for the ice cover in the unsteady flow analysis, the appropriate estimate of the peak discharge is likely a little less than halfway between the above bounds. Such a value is indeed close to the peak mean-daily discharge of 1350 m^3/s estimated by WSC.

While this was doubtless the surge which triggered the 'rapid flow' at Paradise Gardens at 0230 h on May 6 and the run down the East Channel in the early morning of this day, it is unlikely to have been the cause of the damaging surge action on the West Channel at 0300 h on May 7. Rather, this was likely caused by release of the pack which apparently existed through the Pine Point bridge, beginning somewhere downstream of the bridge but upstream of the split (the West Channel was open at the West Channel bridge and ended somewhere above the Pine Point bridge but below the Hay River WSC gauge (Wedel, 1988). From WSC records (Wedel, 1988), this release seems to have occurred about 2300 h on May 6, perhaps in response to the general increase in discharge and warm water temperatures. On the basis that this jam was, say, 5 km long and more-or-less centered on the Pine Point bridge, Equations 6.3 and 6.8 indicate that the discharge at the split would have almost doubled a bit less than an hour after the jam release. Wedel (1988) estimates the discharge just upstream (at the WSC gauge) increased by about 30%, and the timing of events indicates the peak at the split occurred 2 - 3 hours after the release. It is not clear why this period is so much longer than predicted, but it is probably explained somewhat by the fact that the surge released would not have been as dynamic as that predicted and the timing of events was not particularly precise.

As another example, when a jam at Desnoyer Estates, noted during the 1989 break-up, failed it was about 0.8 km long (Gerard and Jasek, 1990) . As Desnoyer Estates are only about 11 km upstream of the split, Equation 6.3 indicates the discharge in Hay River should have increased by some 19%, which should have caused a 0.5 m increase in water level at this location about an hour after the jam failed. No such increase was observed. There are likely several reasons for this: the ice jam at Desnoyer had been undergoing melt for some time and the roughness of the pack was likely less than the 2 m assumed in the unsteady flow analysis, so that the water stored by the jam in its final phase would have been much less; in the analysis the length of the jam is taken as that upstream of the toe, assumed to be 1 km long - hence the observed jam would fit within the assumed toe so that its effective length would be much less than presumed for Equation 6.3; the release was doubtless slower than the instantaneous release assumed in the analysis; and, finally, the surge would have had to pass under the ice jam pack that existed in Hay River, so attenuating the jam more than the open water assumed when developing Equation 6.3.

The above results indicate the conservative nature of the surge predictions, particularly when the jam release is close to the town.

7.0 DEVELOPMENT OF AN INTERACTIVE COMPUTER PROGRAM

The use of relations developed in Phase 1 of this study, together with those of this Phase, is quite laborious, and not at all viable for the non-technical personnel of the Town Flood Watch. Hence the other major component of this Phase 2 study was development of a user-friendly interactive computer program incorporating the flood forecast procedure developed in both Phase 1 and Phase 2. This task was contracted out to HYprocessing Ltd. of Edmonton. The program developed is designated ICEJAM and is programmed to run on an IBM personal computer.

At the discretion of the user, the program ICEJAM displays one of six screens which request input information and/or display water levels predicted for various reference locations around the delta. Copies of these screens are shown in Figure 7.1. Larger versions can be found in Appendix D, which also contains a brief user's manual. The detailed background to the program and displays is given in this report, the Phase 1 report (Gerard and Stanley, 1988) and the break-up and evaluation report (Gerard and Jasek, 1990). A summary is provided below.

The first screen, Figure 7.1a, is simply the title screen. By pressing any key the second screen, Figure 7.1b, is displayed. The second screen is the main menu screen. It can be returned to from any other screen at any time. This screen requests the user to choose between the first or second level flood watch, and asks whether the user knows the location and other details of a significant ice jam upstream of Hay River.

If the first level flood watch is chosen, the first level flood watch screen is displayed (Figure 7.1c). As explained in the Phase 1 report, this level is simply intended to give a long-range indication of whether a flood threat is possible during the following break-up. It asks for the expected snow accumulation at break-up at the three reference locations in the catchment - Hay River, High Level and Fort Nelson. The accumulation at each site is taken as equal to the recorded precipitation, when the mean daily temperature is below 0°C, at the stations on the day the forecast is made, plus the past average accumulation between then and break-up (see Gerard and Jasek, 1990). From this information an estimate is made of the range of discharges (the associated water levels at the West Channel bridge) possible during break-up, using the relation developed in the Phase 1 report. For convenient reference, this relation is reproduced here as Figure 7.2.

The average snowpack in the catchment, S_{av}, is determined from the relation:

$$S_{av} = 0.10\ S_{HR} + 0.34\ S_{FN} + 0.56\ S_{HL} \qquad [7.1]$$

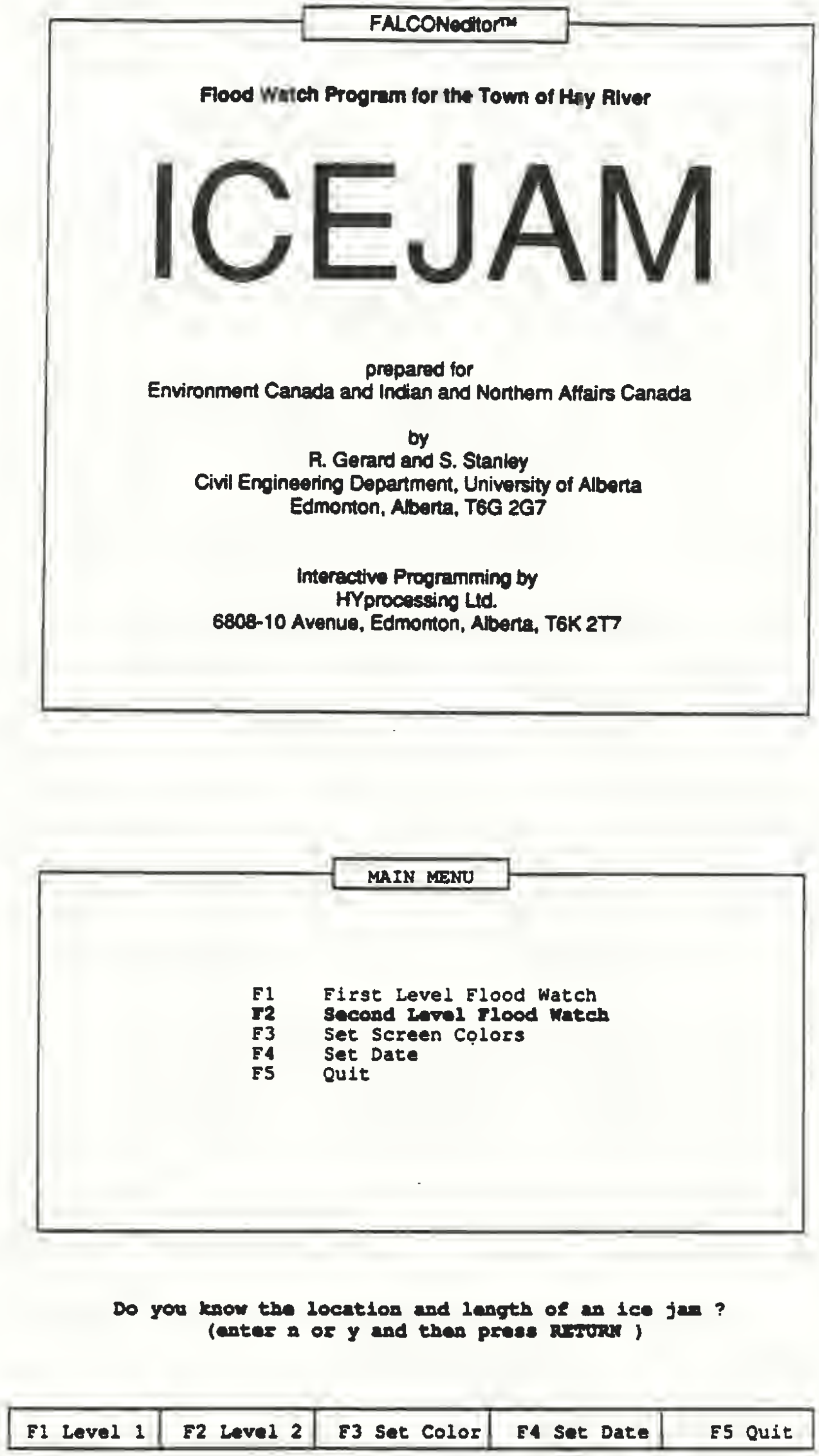


Figure 7.1. Copies of the displays of the interactive computer program ICEJAM: (a) Title screen and (b) Main menu screen.

```
                    FIRST LEVEL FLOOD WATCH

               LOCATION           SNOW DEPTH (cm)
          ---------------------------------------------
            1) Hay River      |        144
            2) Fort Nelson    |        128
            3) High Level     |        109

- Basin Average Accumulated Snowfall = 118  cm
- Maximum likely discharge at Hay River (based on accumulated
  snow only) =    397  to   960  cms
- Maximum likely water level upstream of the West Channel Bridge at
  breakup (assuming no ice deterioration)
              =  161.1   to   163.4   m   (geodetic)
              =  11.6    to   19.1    ft  (on the pier of W.C.B )
```

F1	F2 Level 2	F3 Print	F4 Main Menu	F5 Quit

```
          SECOND LEVEL FLOOD WATCH / 1-DAY FORECAST
                  WITHOUT ICE JAM DETAILS

        Current discharge at the Border      =  680  cms
        Bright sunshine with mean daily
              temperature above -5°C         =  200  hours
        Accumulated snow at Hay River        =  144  cm

West Channel    Datum                      Possible Range of Levels

Open Water      Geodetic (m)                    160.7  to  161.4
                On pier of W.C.B. (ft)           10.3  to   12.8
Jam at W.C.B.   Geodetic (m)                    161.4  to  162.3
                On pier of W.C.B. (ft)           12.6  to   15.5
```

F1 Level 1	F2 Oth. Sites	F3 Print	F4 Main Menu	F5 Quit

Figure 7.1 (continued). Copies of the displays of the interactive computer program ICEJAM: (c) First level flood watch screen and (d) Second level flood watch screen if no ice jam information is available.

SECOND LEVEL FLOOD WATCH

WITH ICE JAM DETAILS

Estimated discharge at Hay River = 680 cms
Distance of ice jam upstream of Hay River = 230 km
Length of ice jam = 25 km

Estimated surge discharge at Hay River = **801.2** to **1050.6** cms
(occurring approximately **26.7** hours after ice jam release)

West Channel	Datum	No Surge	Ice Cover	Open Water
Open Water	Geodetic (m)	**160.7**	**161.0**	**161.6**
	Pier of WCB (ft)	**10.3**	**11.4**	**13.2**
Jam at WCB	Geodetic (m)	**161.4**	**161.8**	**162.5**
	Pier of WCB (ft)	**12.6**	**13.9**	**16.2**

F1 Level 1	F2 Oth. Sites	F3 Print	F4 Main Menu	F5 Quit

DISCHARGE / WATER LEVEL ESTIMATES AT OTHER SITES

WEST CHANNEL CONDITIONS	ESTIMATE	LOCATION W.C.B.	F.V.	Fill C	E.C.*
Open Water	**Discharge (cms)**	**369**	**369**	**310**	310
	Water Level (m)	**161.4**	**160.2**	**160.7**	158.7
	Above Reference (ft)	**12.8**	**5.1**	**-7.4**	1.4
Ice Jam	**Discharge (cms)**	**240**	**240**	**438**	438
	Water Level (m)	**162.3**	**159.5**	**161.6**	159.4
	Above Reference (ft)	**15.5**	**3.0**	**-4.7**	3.4

NOTE - All levels geodetic and are upper value of range.
- Calculations assume ice jam in East Channel.

*** assumes jam at East Channel mouth**

W.C.B. - West Channel Bridge Pier (157.55m)
F.V. - Fishing Village Docks (158.6m)
Fill C - Strang's corner south edge of pavement (spike in pole 31 - 163.0m)
E.C. - East Channel Docks (158.3m)

F1 Level 1	F2 Last Page	F3 Print	F4 Main Menu	F5 Quit

Figure 7.1 (continued). Copies of the displays of the interactive computer program ICEJAM: (e) Second level flood watch screen if ice jam information is available and (f) 'Other sites' screen.

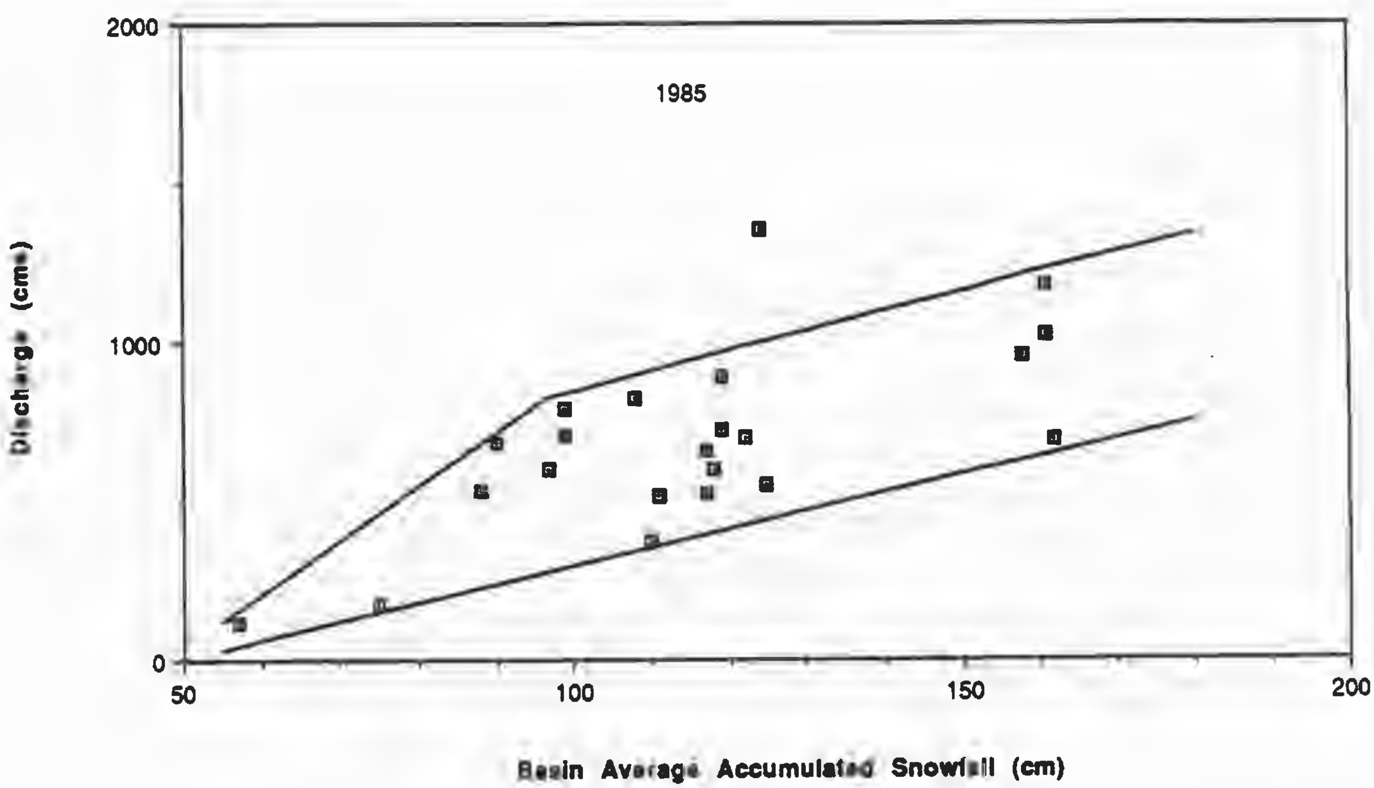


Figure 7.2. Relation between accumulated snow in the catchment at break-up and the possible peak mean-daily discharge at Hay River.

S_{HR} being the accumulated snowfall over the winter at Hay River, S_{FN} that at Fort Nelson and S_{HL} that at High Level. The range of values evident in Figure 7.2 have been bounded by the following relationships for the purposes of the computer program: for the upper bound, for $S_{av} \leq 97$ cm

$$Q = -800 + 16.7\, S_{av} \qquad [7.2a]$$

while for $S_{av} > 97$ cm

$$Q = 21\, S_{av}^{0.80} \qquad [7.2b]$$

where Q is the discharge to be expected in Hay River during break-up in m^3/s and S_{av} is in cm. The lower bound is taken as:

$$Q = -290 + 5.78\, S_{av} \qquad [7.3]$$

The results of the first level flood watch analysis are displayed on the same screen, Figure 7.1c.

This procedure is to be used as many times as appropriate as break-up approaches and is intended to be equivalent to the current practice of an experienced member of the Flood Watch touring portion of the catchment and making an assessment of the likely situation at break-up on the basis of experience and the depth of the snow pack.

As break-up becomes imminent, or perhaps more importantly, after break-up has moved through the town and the ice jams are in place, the second level flood watch screens can be called up from the main menu.

There are two possible screens for this level. The first (Figure 7.1d) is for use when there is no information on ice jams upstream. This utilizes the current estimate of the discharge at the border WSC station as at least a 1-day forecast of the discharge in Hay River* and, with an estimate of the bright sunshine hours which will have accumulated at Fort Smith** 1 day hence (a

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* There is some indication that high discharges can move from the border to Hay River in about 1 day while low discharges take some 2 days. To be conservative a 1 day interval is assumed (Gerard and Jasek, 1990). See also the earlier discussion of the 1985 event.

** The Fort Smith data was used in establishing this relation as it had the only long-term record of sunshine data to relate to the recorded discharges and water levels in Hay River. If a sunshine recorder is installed in Hay River this data could likely be used instead. Such data would also afford the means to come up with a more direct relation for Hay River in the future. The constraint that the accumulated sunshine only be for days when the mean daily temperature is above -5°C is an estimate of the temperature required for significant snowmelt due to radiation absorption to take place during the day.

measure of the potential state of decay of the ice) and the accumulated winter precipitation at Hay River (a measure of the protection of the ice from decay by sunshine) an estimate is made of the water levels likely a day or so later at the West Channel bridge.

The estimate is based on the steady state ice jam rating curves for the split given in Figure 7.3. It is noted that the water level at the West Channel bridge will depend on whether or not there is an ice jam in the upstream portion of the West Channel; it is always assumed there will be an ice jam pack extending at least up to the East-West Channel split in the East Channel. The curves of Figure 7.3 are described by the following equations, which were fitted by regression:

$$\$ = 0.219\, Q^{0.4489}\ \mathrm{m} \qquad [7.4]$$

for the West Channel open below the split, where \$ is the stage in m above a reference water level of 156.6 m and Q is the discharge in m^3/s. For an ice jam pack in the West Channel below the split:

$$\$ = 0.222\, Q^{0.4713}\ \mathrm{m} \qquad [7.5]$$

above the reference water level. The water level on the West Channel bridge pier is determined by adjusting these stages to a reference water level of 157.55 m, this being the geodetic elevation of the zero of the water level scale painted on the bridge pier.

A 'variability' correction is then applied to these stages, where this is given by:

$$R = 1.2 - 0.000024\, E^2 \quad \text{for } R \leq 1 \qquad [7.6]$$

a relation displayed in Figure 7.4, where R is the ratio of the estimated stage to the steady state stage determined from Equations 7.4 or 7.5, and E is given by:

$$E = B - 1.2\, S_n \qquad [7.7]$$

where B is the hours of bright sunshine accumulated at Fort Smith after the mean daily temperature at Hay River rises above -5°C, and S_n is the accumulated snowfall at Hay River over the winter in cm.

From the variability factor, R, a likely range of water levels is determined, the low value being the level estimated from Equations 7.4 or 7.5, and the high value being the maximum likely instantaneous value determined from the value of R given by Equation 7.6.

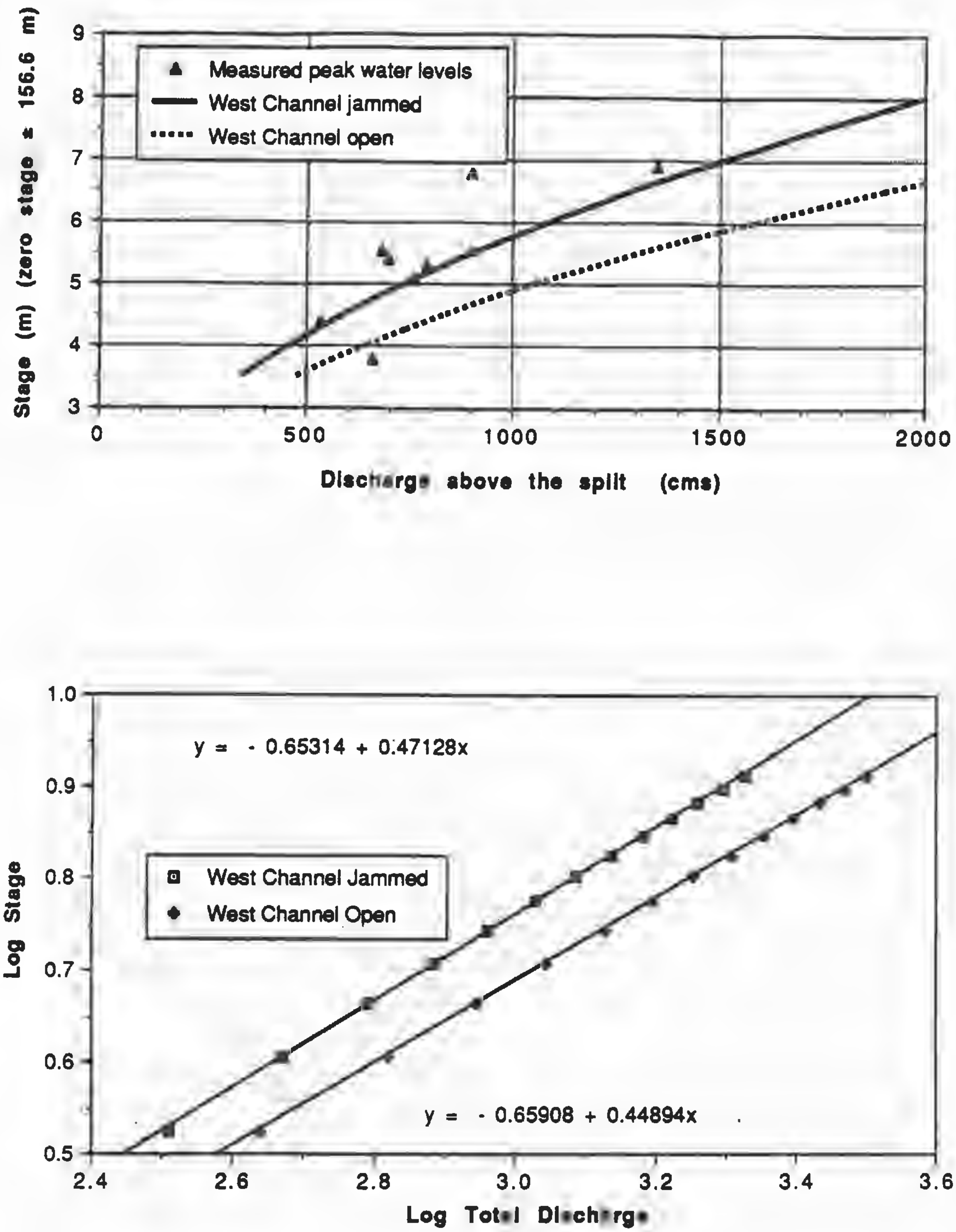


Figure 7.3. (a) Arithmetic and (b) transformed ice jam rating curves for the Hay River at the East-West Channel split.

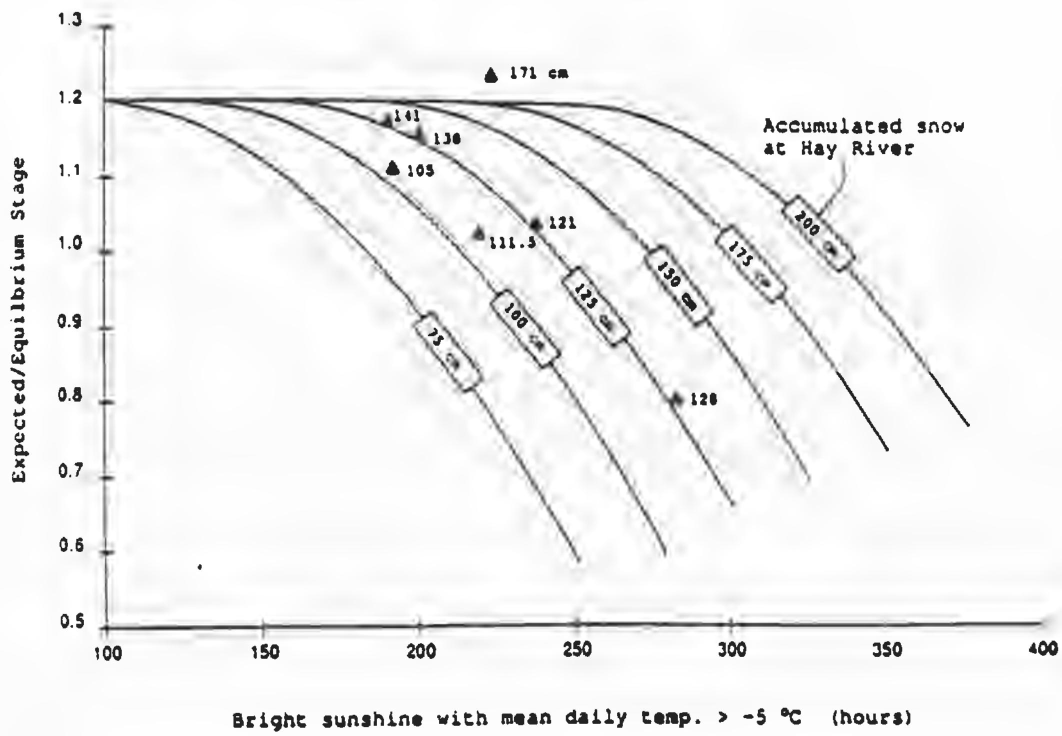


Figure 7.4. Variation of stage from the mean-daily level with ice decay parameters.

On requesting 'Other sites' on this second level flood watch screen, the screen shown in Figure 7.1 (f) is displayed, which gives the discharge estimates in the East and West Channels together with the associated water levels expected at the various reference locations throughout the delta, these being

(i) the Fishing Village dock, this being a convenient reference location for the Fishing Village ;

(ii) the vicinity of Fill C, or Strang's corner, where there is a low point, relative to river water levels, in the road to Vale Island; and

(iii) the Government docks on the East Channel, this being a convenient reference location for Old Town.

To determine these levels the discharge split between the East and West Channels is determined from ice jam rating curves for the East and West Channels at the split. The discharge in the West Channel is given by Equation 7.8:

$$Q = 15.05\,\$^{2.269}\ \mathrm{m^3/s} \qquad \text{for } 200 < Q < 900\ \mathrm{m^3/s} \qquad [7.8]$$

for the West Channel open below the split, where $ is in m and refers to the reference water level at the split of 156.60 m, or Equation 7.9:

$$Q = 10.06\,\$^{2.022}\ \mathrm{m^3/s} \qquad [7.9]$$

for an ice jam pack on the West Channel below the split. These relations are shown in Figures 7.5 (a) and (b).

For the East Channel below the split *

$$Q = 14.42\,\$^{2.176}\ \mathrm{m^3/s} \qquad [7.10]$$

---

* In the Phase 1 report this discharge was determined from the difference between the total Hay River flow and that estimated for the West Channel. Taking such differences can lead to large errors so it was decided to develop a relation, Equation 7.10, which gave the East Channel discharge directly from the stage at the split. Small errors can therefore be expected if the estimated East and West Channel discharges are added to get the total Hay River discharge.

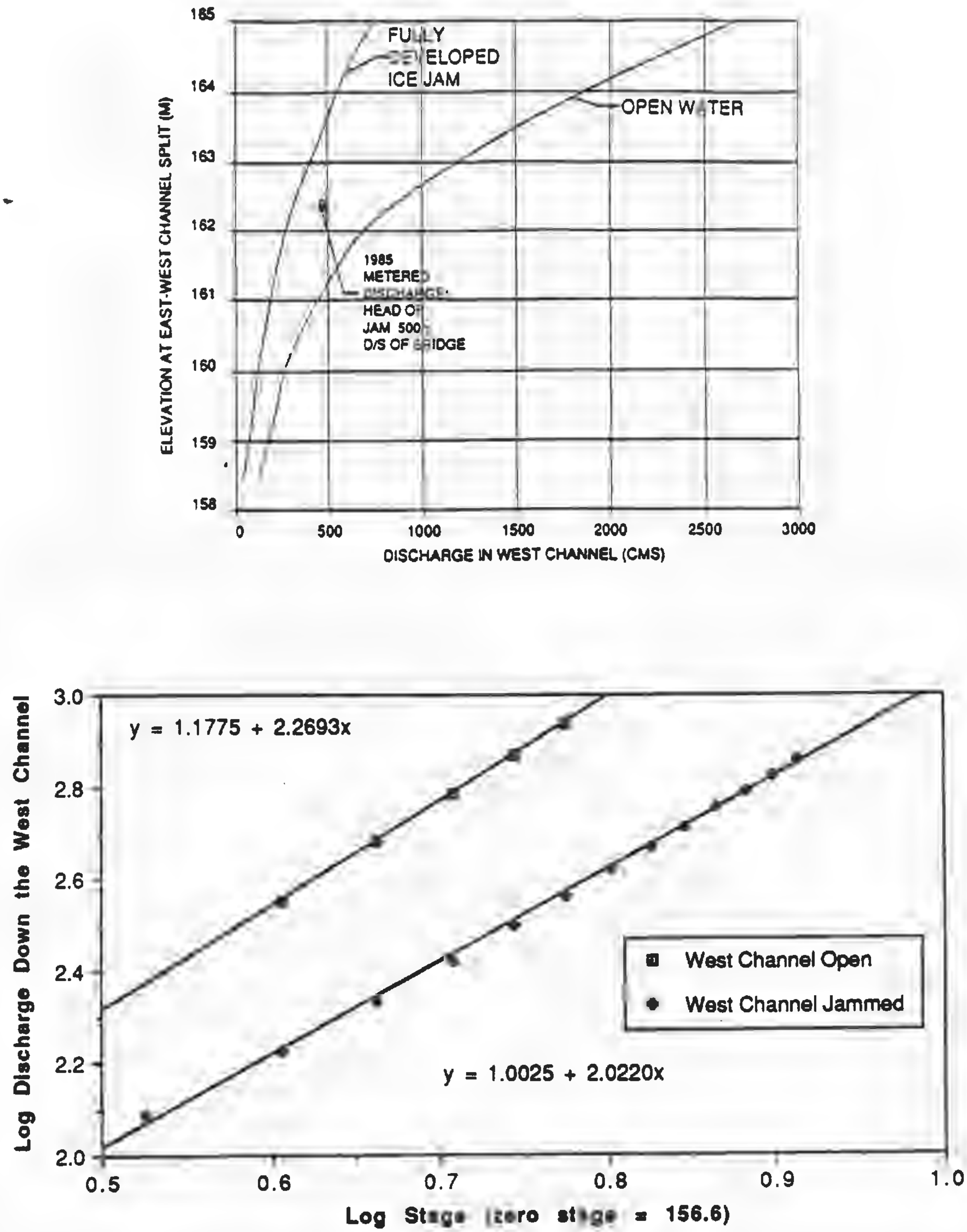
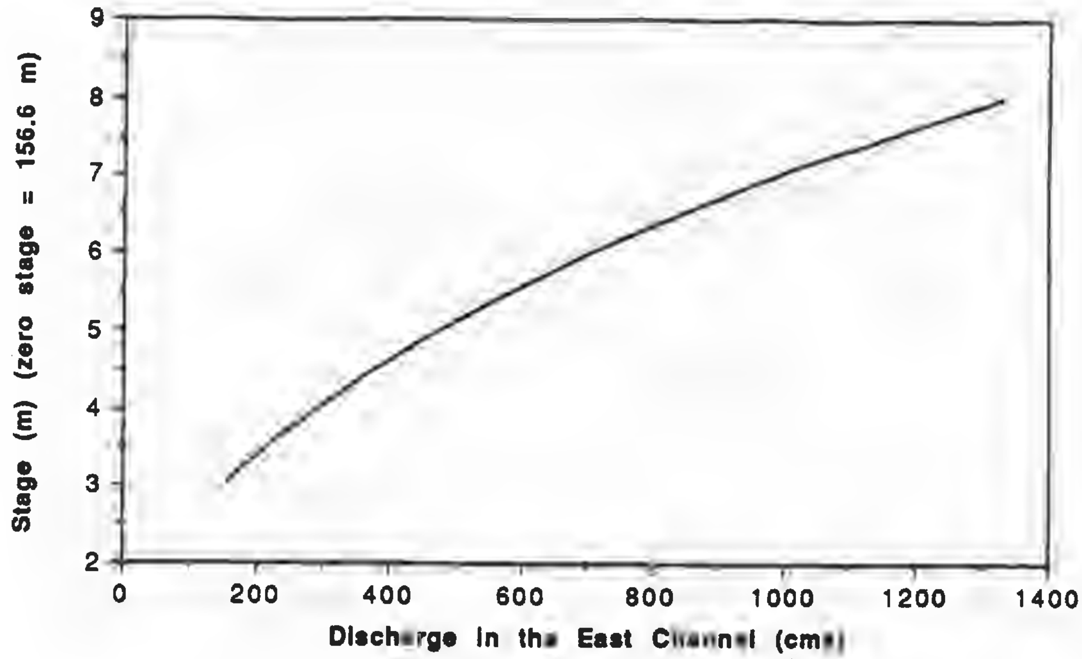


Figure 7.5 (a) Arithmetic and (b) transformed variation of the discharge in the West Channel with stage for both ice jammed and open water conditions in the West Channel



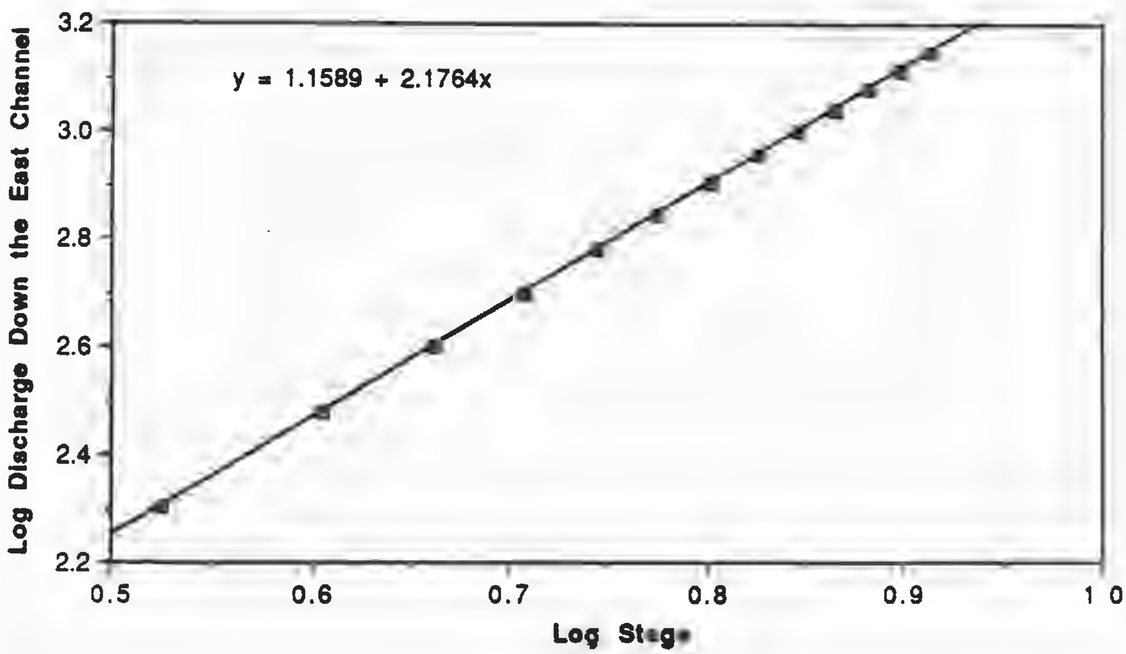


Figure 7.5 (c) Arithmetic and (d) transformed variation of the discharge in the East Channel.

This relation is shown in Figures 7.5 (c) and (d).

From these estimated discharges in the East and West Channels, water levels at the Fishing Village and East Channel docks are estimated from the rating curves developed in Phase 1, with the possible range determined by adding the same water level variations as for the split. However, unlike the display for the West Channel bridge, the 'Other sites' display only gives the upper value of the range estimated for each site.

The level at Fill C, or Strang's corner, is determined from the level at the East-West Channel split by subtracting the fall in water level from this site to Fill C which, for an equilibrium pack, is that of the channel - 0.5 m/km or 0.71 m. The level of the spike in power pole #31, which was taken as the reference level at this site, is 163.0 m; this is about the same level as the centre of the road at the low spot near here.

The relation used for the Fishing Village dock is that given in Figure 7.6 and Equations 7.11:

$$\$ = 0.112\,Q^{0.5841}\ \mathrm{m} \qquad \text{if } Q < 440\ \mathrm{m^3/s} \qquad [7.11a]$$

and

$$\$ = 0.668\,Q^{0.2911}\ \mathrm{m} \qquad \text{if } Q \geq 440\ \mathrm{m^3/s} \qquad [7.11b]$$

which apply to the **West Channel split** near the mouth, corrected to the Fishing Village by subtracting the surveyed fall in the water level over this reach of 0.9 m/km, or 0.72 m. The distinction between the above two equations is overbank flow, which is important at this site. The reference water level for the stage in Equations 7.11 is 156.6 m. The level of the docks is 158.6 m (which, it is noted, is about 2 m - or 6.6 ft - above the mean lake level in spring).

The stage at the East Channel docks is estimated from the ice jam rating curve shown in Figure 7.7, which is described by Equation 7.12:

$$\$ = 0.0409\,Q^{0.6678}\ \mathrm{m} \qquad [7.12]$$

where the reference water level for the stage is 156.1 m. The level of the East Channel docks is 158.3 m.

It is to be noted that these levels for the East Channel docks assume the toe of the jam has moved to near the mouth while the pack is still reasonably long. While this provides an estimate of the worst case possible, in most years it will not happen. Judgement must therefore be used each year as to whether it is a possibility, before a flood warning is issued for the Old Town. The possibility of 'crying wolf' too many times for this site must be accepted as an unavoidable aspect of the problem at this time.

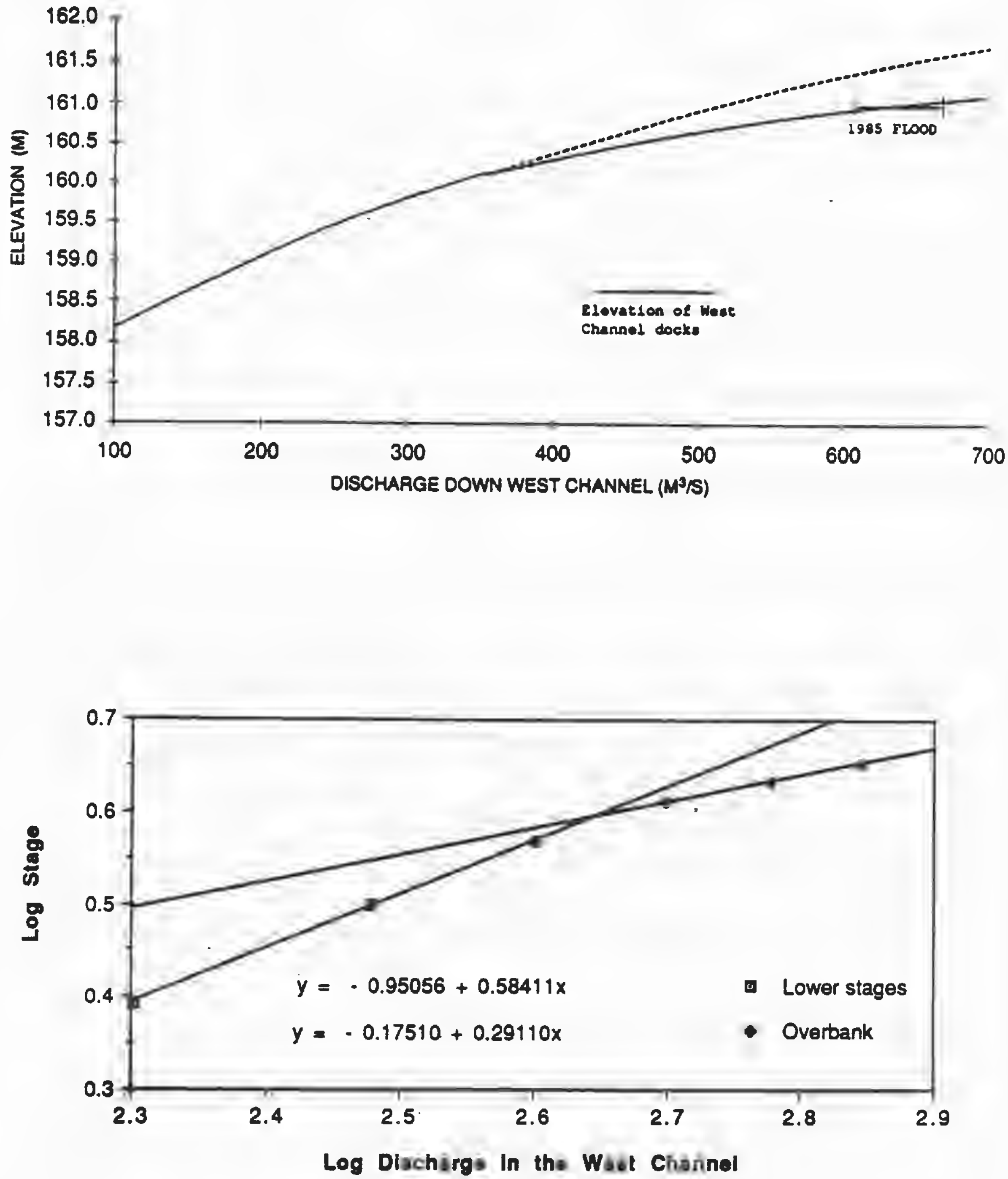


Figure 7.6. (a) Arithmetic and (b) transformed ice jam rating curve for the West Channel just above the West Channel split, at the north end of the main airport runway.

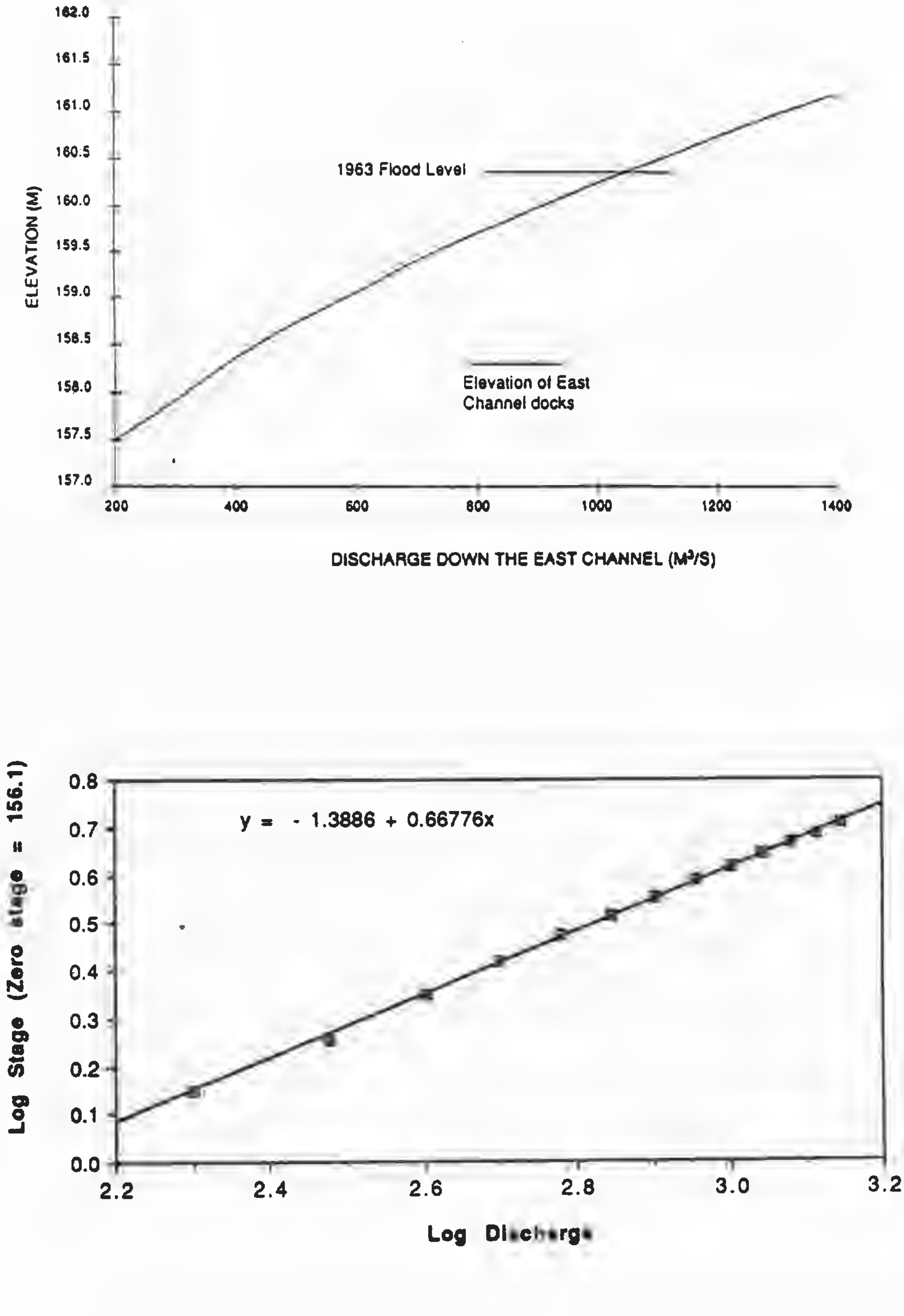


Figure 7.7. (a) Arithmetic and (b) transformed ice jam rating curve for the East Channel at the Government Docks (opposite Island A).

If details of ice jams upstream are known from aerial reconnaissance, the second 'Second level' flood watch screen (Figure 7.1e) can be mobilized. This screen is based on the study described in this Phase 2 report. It requires estimates of the discharge at Hay River (the carrier discharge), the distance upstream to the jam and the jam length. In return the screen will display a range of discharges to be expected in Hay River near the West Channel bridge. The screen also provides an estimate of the time of arrival of the surge. As discussed, the small difference in the arrival time due to the presence or not of an ice cover is small and has been ignored in this procedure The equations used to derive these values are described in Section 6 of this report.. Importantly, **no account has been taken of the possibility of break-up, of a sequence of ice jams possibly interacting, or even of the presence of a long rough ice jam pack upstream of Hay River.** These features must be left to judgement at this time. An example of the judgement required is given in Section 6.3.

As well as the estimates of discharge and timing, this screen provides estimates of the water level to be expected at the West Channel bridge as a result of the surge. As before, the values distinguish between whether there is a jam in the West Channel at the split or not. For each circumstance at the split, three values of water level are given: the level to be expected for steady flow at the carrier discharge (these are the same as the lower values of screen 4 shown in Figure 7.1 (d)); the level to be expected due to surge action , with a solid ice cover between the jam and the town; and the level if this reach is totally open.If the 'with ice jam details' option for the second level flood watch is selected, the screen which gives the estimates of levels at 'Other sites' is based on the surge discharge to be expected from the open-water-below-the-jam scenario. The surge discharge has been distributed between the delta channels on the basis of a quasi-steady assumption and, as this likely represents an upper bound already, no allowance has been made for the 'variability' incorporated into the without-ice-jam-detail screen.

The user's manual developed for ICEJAM forms Appendix D of this report. Sample screen displays are shown there for worked examples. The program listing is given in Appendix E.

8.0 DISCUSSION

8.1 Surge analysis

The objective of the Phase 2 study was to develop an algorithm which allows some direct account of surges that may be released by ice jams upstream. To do this a sophisticated finite element unsteady flow analysis algorithm was used which had recently been developed at the University of Alberta. While the unsteady flow analysis is believed to be quite rugged and accurate, there are still limitations imposed by the complexity and poor understanding of the break-up process in general and the lack of detailed field data on surge propagation in the Hay River. For example, the surge analysis considered just one ice jam in the reach at a time, with either open water or a solid ice cover over the whole reach downstream of the jam site.

However reality is generally more complex than this. It is not uncommon for there to be more than one ice jam in the reach, and for the failure of one ice jam to cause the failure of others. An example of this in the Hay River is described by Gerard and Jasek (1990). Even with only one ice jam in the reach, the analysis does not consider the complexities possible. For example, the analysis assumes the ice jam fails over its whole length at once. In reality the ice jam will likely fail in stages, with a negative river wave propagating along the pack. This would cause the resultant surge to be somewhat less severe. However, little is known of the processes associated with ice jam failure so the conservative assumption of instantaneous failure was used.

It is also unlikely there will be either wholly open water or solid ice cover downstream of the ice jam. The more likely circumstance is to have an intermittent (in distance) ice cover, with an ice jam pack of some length through the town. If the actual ice cover distribution and type was the same each year and was known, an analysis specific to this situation could be carried out. However, the situation differs from year to year and from moment to moment in any given year. Furthermore, the action of the surge itself would likely change the configuration of any ice cover or ice jam downstream, which in turn would change the nature of the surge, in a presently unpredictable fashion.

Hence the unsteady flow analysis was constrained to the analysis of the two situations that should bound the situation in any given year. It was difficult enough to generalize these simpler situations for use in the interactive program. The surge predictions produced by this portion of the program must therefore be used with some judgement, somewhat along the lines described in Section 6.3.

8.2 Appraisal of the flood forecast program ICEJAM

The basis for the components of the interactive program were developed during the Phase 1 study and their limitations are described in that report (Gerard and Stanley, 1988). The success of these components in forecasting water levels was evaluated during the 1989 break-up (Gerard and Jasek, 1990) and resulted in some modifications. As a result of the surge analysis component of this Phase 2 investigation, a component was added to the flood forecast algorithm to allow explicit account to be taken of the action of surges that may be released by ice jam failure upstream. Because of the assumptions required in the surge analysis (eg. instantaneous failure over the whole length of the jam) it is believed the discharge, and hence water level, estimates using this portion of the algorithm should be quite conservative, especially if the jam is close to Hay River. This is somewhat confirmed by the very limited field data available, as discussed in Section 6.3.

It is therefore expected that the upper water levels forecast at the West Channel bridge, the Fishing Village and Fill C will be at or above the water levels that will actually occur for a given situation and therefore represent a plausible worst case. However, as indicated by the range of water levels displayed for the West Channel bridge, in some years the water levels may be up to a metre lower than these worst case values. This could make a significant difference in the nature of any flooding that may occur but at present it is not possible to give more refined estimates. Hence in planning any flood damage mitigation activities some judgement will be required to avoid too many false alarms. Some assistance with this judgement will be afforded by a comparison of the forecast water levels with those actually observed at the West Channel bridge in any given year.

The water level for the East Channel docks will almost invariably be high because of the assumption that the ice jam will move to the mouth, which happens only rarely, so the situation at this site must be assessed with particular care before evacuation orders are given. The limited information that does exist suggests that if the ice jam stalls at the usual location near Island CD, and stays there for a day or so, the toe will not then move to the mouth until the pack is so short as to preclude development of the full increase in water level at the East Channel docks.

An important feature that has been suggested by the observations and calculations in the last year or so is that a high discharge may take significantly less than two days to move from the border to Hay River, two days being the period accepted in the Phase 1 study. For example, Equation 6.9 indicates the time for a surge released at the border to travel to Hay River is about 16 hours, while observations of the 1989 break-up indicated break-up could move over the same distance in just 12 hours! Although these two situations are quite dynamic in nature and propagate faster than typical snowmelt flood discharges, the indications are that even the latter will take less than two days to move from the border to Hay River. Consequently application of ICEJAM should assume only a one-day forecast.

The possibility of a shorter travel time should be confirmed by field measurements of well-defined discharge events in the summer. If it proves to be correct, thought should be given to development of a relatively simple precipitation-runoff model of the Hay River catchment to allow the forecast period to be extended. Such a model would also allow consideration of rainfall such as that which is supposed to have played such a significant role in the 1985 flood event (Wedel, 1988). However, this would also require a more extensive meteorological network than is presently available in the Hay River catchment.

It is believed the components of the ice jam flood forecast algorithm developed in the two phases of this study are as sophisticated as the current state of understanding of break-up and ice jams will allow. However, there are other components that would likely be worthwhile to develop and would provide some refinement of the present flood forecast procedure. One is the precipitation-runoff model. Another would be a simple ice jam melt algorithm, based on observed water temperatures. The latter would be used to predict when the ice jam pack in Hay River would be melted sufficiently that any higher discharge expected from upstream would not cause higher water levels. Furthermore, observations over the last few years have suggested - but as yet the evidence is very tentative - that the toe of the ice jams in the East and West Channels are prone to move when the pack is short enough that above 0°C water from upstream can reach the toe and begin the melt process there. Prediction of such an occurrence could be simply worked into the melt algorithm. Development of these two modules - precipitation/runoff and ice jam melt - would complete the suite of programs required to allow as comprehensive and accurate flood forecasting for Hay River as is possible at this time.

Beyond this is the need for continuing, systematic evaluation of the algorithm against actual events, much as was done for the 1989 break-up. Only after several years of such observations will the success of the algorithm be able to be judged and possibly refined.

9.0 CONCLUSIONS

Field work and the application of a sophisticated finite element unsteady flow analysis has allowed analysis of the results of ice jam failure in the complex reach of the Hay River above Hay River. While the analysis has many limitations, it is believed to be the best that can be developed within the constraints of the current understanding of ice jam and break-up processes.

The results of this analysis have been reduced through dimensional analysis to a series of simplified 'influence lines' which allow estimates of the magnitude and timing of increases in discharge in Hay River due to ice jam failure upstream, and which can be incorporated into the interactive computer program developed as part of this study.

This interactive, user-friendly computer program incorporates the results of both the Phase 1 and 2 studies. It will allow full application of the somewhat complex ice jam flood forecast algorithm by the non-technical personnel of the Hay River Flood Watch. The surge component of this program will likely be conservative and must, at this time, be used with considerable judgement. Despite its limitations, this interactive program, if applied systematically, will allow forecasts to be formalized, should assist the work of the Flood Watch, and should reduce the present dependence on experienced personnel. It should also provide a firm base against which to compare future events and assess the possibility of improvements to the procedure.

It is believed two relatively simple avenues of work would provide further improvement of the flood forecast procedure and complete the suite of relations required for flood forecasting in Hay River. These are the development of an ice jam melt algorithm and of a precipitation-runoff algorithm for the Hay River.

Most importantly, continued systematic evaluation of the algorithm against events in the field is required to assess its efficacy and to lead to its improvement.

10.0 REFERENCES

Amien, M. 1968. An implicit method for natural flood routing. Water Resources Research, Vol. 4, No. 4, pp. 719-726.

Beltaos, S. 1983. River ice jams: theory, case studies and applications. Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol. 109, No. 10: pp. 1338-1359 and discussion.

Gerard, R. and Stanley, S. 1988. Ice jams and flood forecasting, Hay River, N.W.T. Water Resources Engineering Report No. 88-6, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, prepared for Environment Canada and Indian and Northern Affairs Canada, Yellowknife, N.W.T., October, 1988.

Gerard, R. and Jasek, M. 1990. Break-up observations and ice jam flood forecast algorithm evaluation, Hay River, N.W.T., 1989. Water Resources Engineering Report No. 90-3, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, prepared for Indian and Northern Affairs Canada, Yellowknife, N.W.T., June 1990.

Hicks, F.E. and Steffler, P. 1990. Finite element modeling of open channel flow. Water Resources Engineering Report No. 90-6, Department of Civil Engineering, University of Alberta, Edmonton, Alberta.

Wedel, J.H. 1988. 1985 Hay River flood report. Report No. IWD-NWT-WPM-002-88, prepared by the Water Planning and Management Branch, NWT Program, Inland Waters Directorate, Western and Northern Region, Environment Canada: 68 p.

APPENDIX A

Terms of Reference

The objective of this study is to develop a method to add the effects of surges to the existing flood forecast system developed during Phase I and to computerize both the original flood level algorithm and the surge estimates method.

Tasks:

1. Adapt the Phase I flood forecast algorithm to a user-friendly personal computer format, suitable for easy use by the Hay River Town Floodwatch Committee and future flood forecasters.

2. Develop an algorithm to account for and predict the occurrence and effects of surges on water levels through the delta.

3. Incorporate the surge algorithm into the computer program of Task 1.

4. Carry out at least 5 cross-sectional surveys and other necessary observations at each of 3 selected reaches of the Hay River to define the hydraulic characteristics of the river for use in the surge routing calculations. The recommended reaches are at Indian Cabins, below Alexandra Falls and Paradise Gardens.

Results and Deliverables:

Since most of the historical background data for the basin were collected and presented in the Phase I contract, the results of this contract are of an applied rather than a descriptive nature. In particular, the contract should produce:

1. an accurate and functioning user-friendly personal computer version of the entire flood forecast algorithm, in a computer format compatible with the computers available to the Hay River Floodwatch Committee, which includes the results of the Phase I work and the surge work of Phase II. This will be suitable for use by the Hay River Floodwatch Committee and others and is not to require proprietary software to run. The program will become public property on completion of the contract.

2. field cross-section data for the three representative reaches of the Hay River mentioned above or an equivalent for other reaches, as required for development of a surge algorithm.

3. a surge algorithm that, given data readily obtainable by the Hay River Floodwatch observers, will account for and predict the potential for

surges and the effects of the surges on the water levels throughout the delta.

4. reports as outlined below.

Required Reporting

1. Concise (1 page) progress work reports to support invoices.

2. Five copies and the camera-ready original of two final reports as described below:

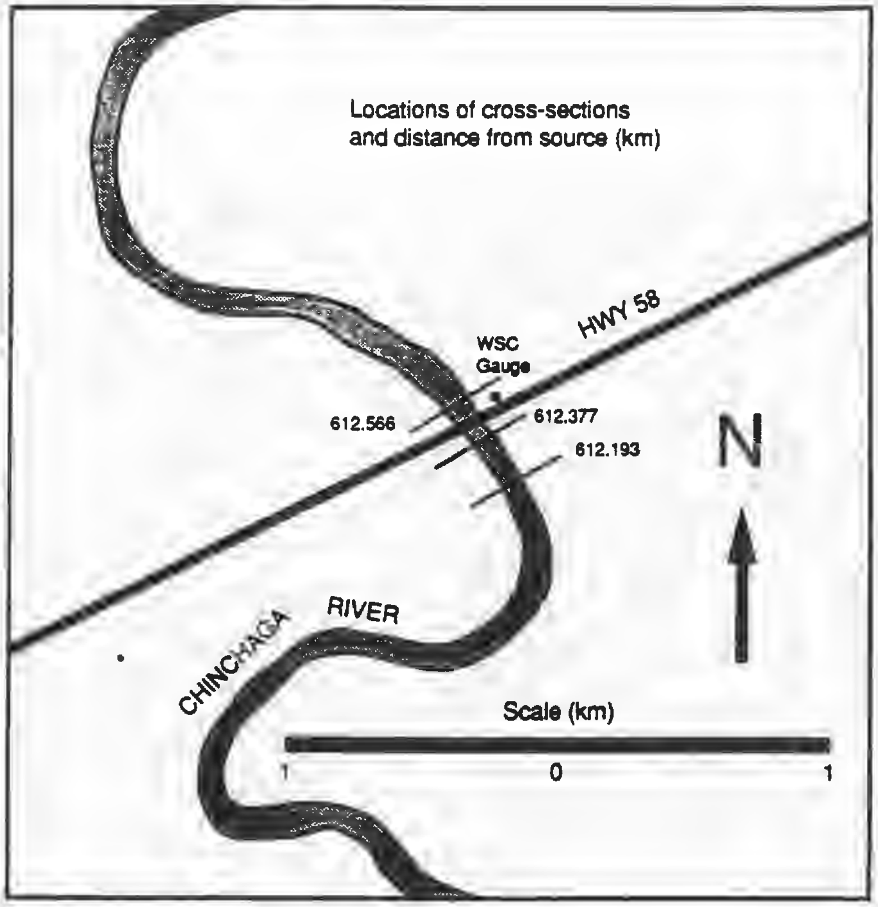
   a) A technical report including:

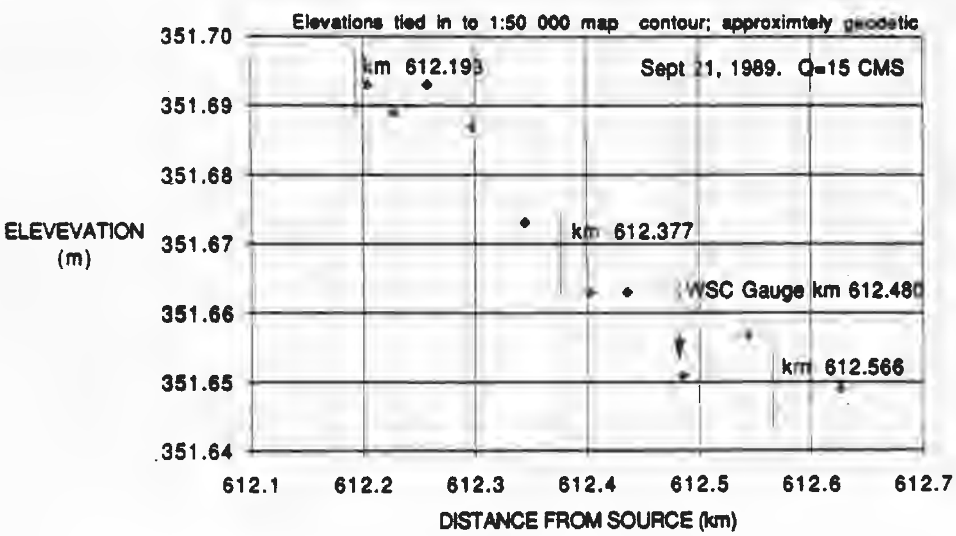
      - data collected as part of this study
      - a description of the scientific knowledge and reasoning employed in the development of the surge algorithm
      - a programmer's guide to the software which will permit the maintenance of this software by future programmers
      - a clear hard copy printout of the computer program
      - a description of the methods and formulae used to computerize graphs and other parts of the algorithm
      - adequate user instructions for operation of the computerized model
      - a computer diskette containing the program.

   b) A short public report prepared in cooperation with DOE and DIAND that includes a brief that includes a brief section on the background of the Hay River flood situation and a general description of what has been produced (i.e. reports, model and computer program) for distribution to the general public. The length of the report is to be 8-15 pages.

APPENDIX B

Reach Surveys





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[illegible]12.[illegible]7	351.6[illegible]
61[illegible]7	351.687
61[illegible].34[illegible]	351.6[illegible]
61[illegible]3	351.[illegible]
61[illegible].438	3[illegible]1.6[illegible]
612.48[illegible]	351.6[illegible]1
61[illegible].544	351.6[illegible]7
[illegible]1[illegible].6[illegible]6	351.64[illegible]

Figure B1. Chinchaga River at Highway 58 crossing (a) Plan (b) Longitudinal water surface profile on day of survey.

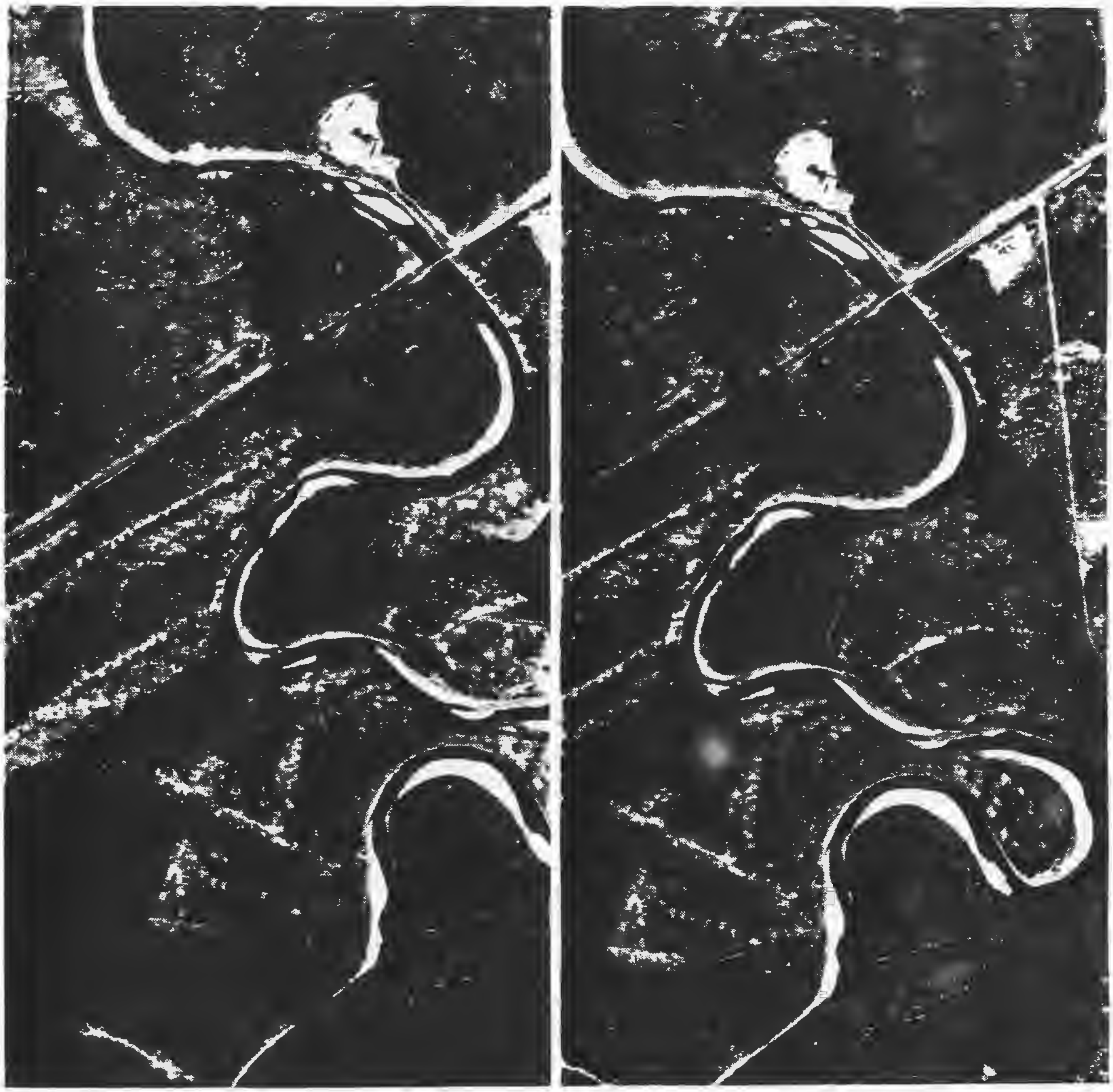
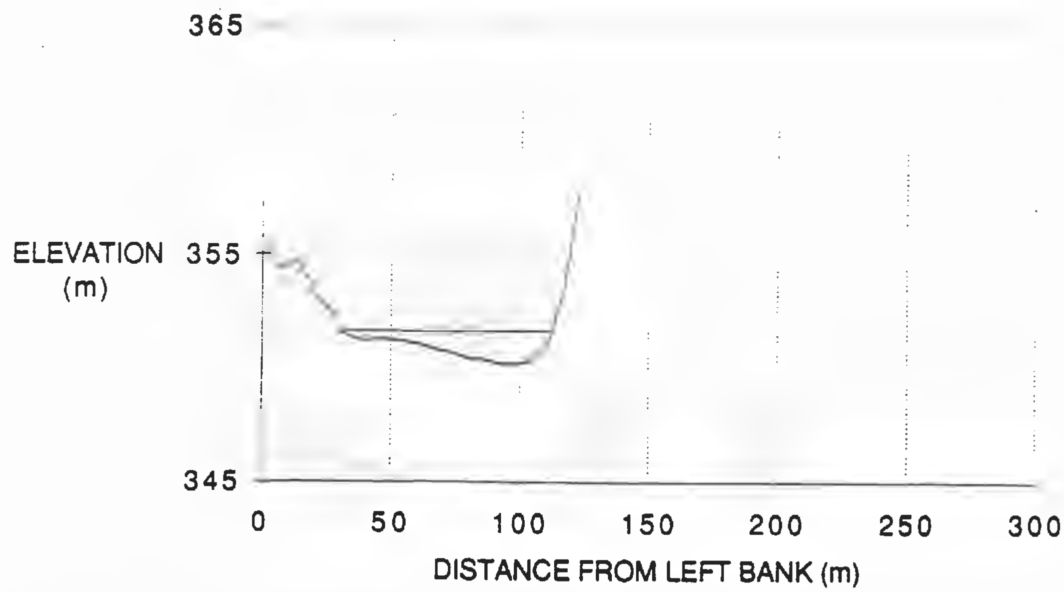


Figure B1 (continued) Chinchaga River at Highway 58 crossing (c) Airphoto of reach, 26 Sept. 87, scale 1:25000.

Figure B1 (continued) Chinchaga River at Highway 58 crossing (d) Reach photos (i) Right bank, and (ii) Left bank upstream of bridge.

Figure B1 (continued). Chinchaga River at Highway 58 crossing (d) Reach photos (continued)(iii) Bed material, left bank downstream of bridge.



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	355.35
3.0	355.37
7.0	354.41
14.0	354.78
17.0	354.08
19.0	353.85
22.0	352.97
29.0	352.29
29.2	351.68
38.0	351.26
44.0	351.29
50.0	351.33
56.0	351.24
62.0	351.07
68.0	350.87
74.0	350.73
80.0	350.47
86.0	350.43
92.0	350.21
98.0	350.20
104.0	350.33
110.0	350.98
112.5	351.68
114.5	352.68
116.5	353.27
120.5	356.21
122.4	357.67

CROSS SECTION km 612.193

Location: 0.28 km U/S from Highway 58 bridge crossing

Left bank:

Heavy brush and tree growth to within 10 m of the waters edge. Generlly flat land beyond section which has likely flooded overbank in the past.

Right bank:

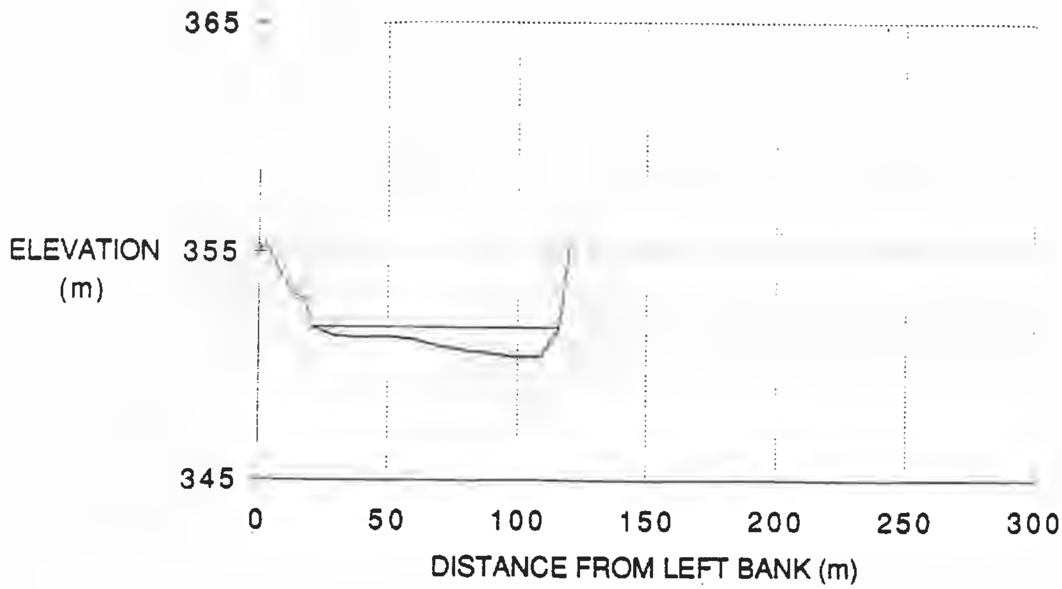
Steep slope to near waters edge. Some sloughing. Approximaitly 15-20 m to top of bank. Little vegatation on slopes.

Water level on Sept 20,1989: 351.681 m

Q = 17 CMS

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B1 (continued). Chinchaga River at Highway 58 crossing (e) Cross section km 612.19



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	355.10
3.0	355.36
9.3	353.90
14.0	353.12
18.0	352.91
20.4	351.69
29.0	351.31
39.0	351.25
49.0	351.28
59.0	351.19
69.0	350.92
79.0	350.70
89.0	350.57
99.0	350.39
109.0	350.44
116.2	351.69
120.4	355.48

CROSS SECTION km 612.377

Location: 0.10 km U/S from Highway 58 bridge crossing

Left bank:

Heavy brush and tree growth to within 10 m of the waters edge. Generlly flat land beyond section which has likely flooded overbank in the past.

Right bank:

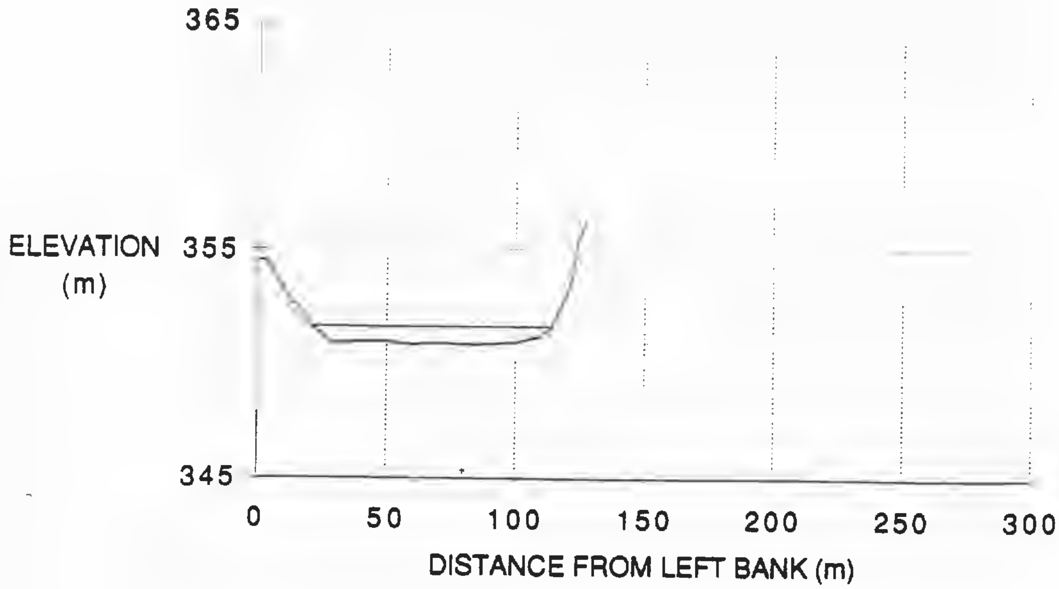
Steep slope to near waters edge. Some sloughing. Approximaitly 15-20 m to top of bank. Little vegatation on slopes.

Water level on Sept 20,1989: 351.689 m

Q = 17 CMS

Elevations tied in to 1:50 000 map contour; **approximately geodetic.**

Figure B1 (continued) Chinchaga River at Highway 58 crossing (f) Cross section km 612.38



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	354.54
3.0	354.51
13.0	352.72
18.0	352.29
20.5	351.65
29.0	350.91
39.0	350.98
49.0	351.01
59.0	350.86
69.0	350.93
79.0	350.86
89.0	350.88
99.0	350.94
109.0	351.17
114.0	351.64
119.0	352.88
122.0	353.89
123.6	355.17
127.0	356.32

CROSS SECTION km 612.566

Location: 0.09 km D/S from Highway 58 bridge crossing

Left bank:

Heavy brush and tree growth to within 10 m of the waters edge. Generlly flat land beyond section which has likely flooded overbank in the past.

Right bank:

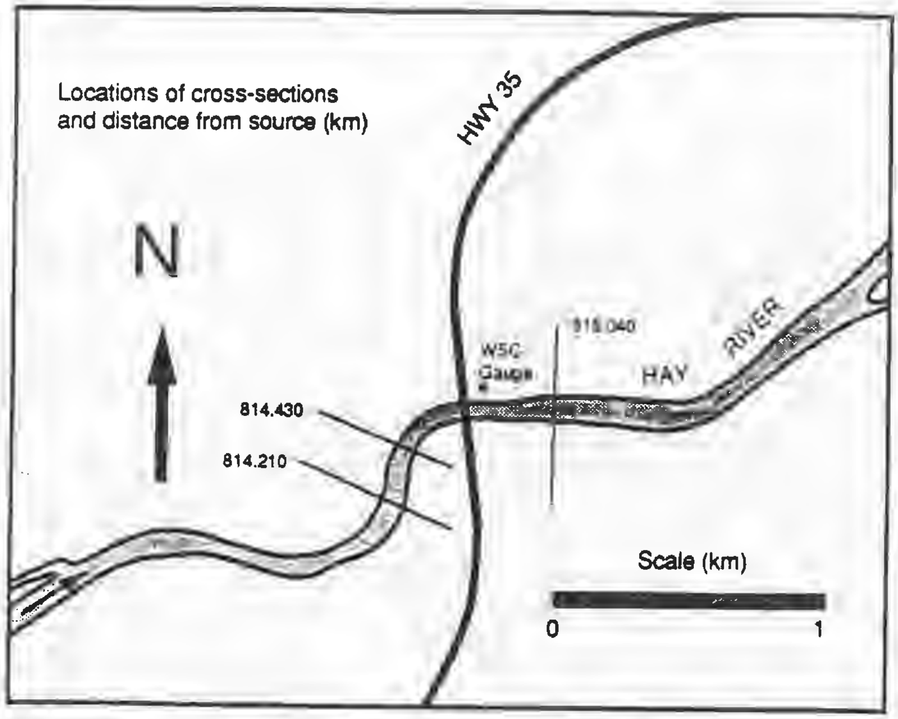
Steep slope to near waters edge. Some sloughing. Approximaitly 15-20 m to top of bank. Little vegatation on slopes.

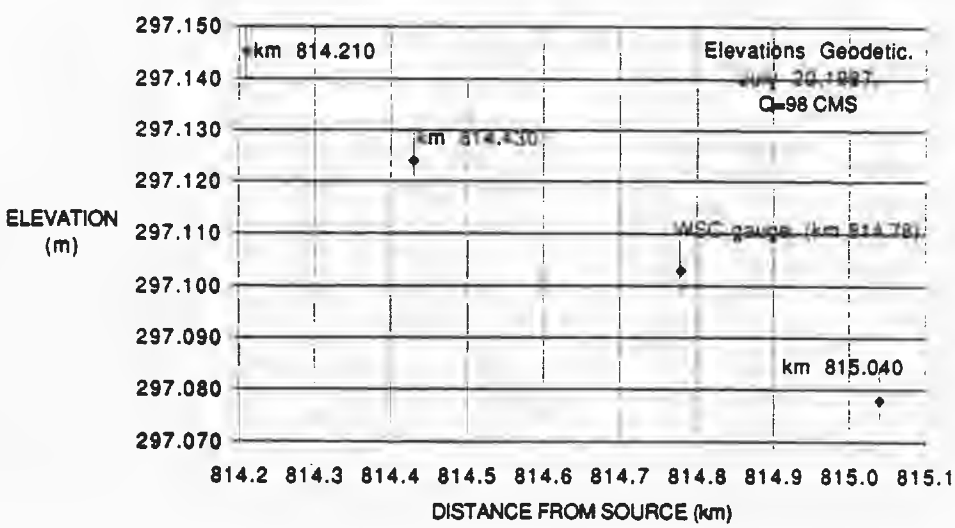
Water level on Sept 20,1989: 351.650 m

Q = 17 CMS

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B1 (continued) Chinchaga River at Highway 58 crossing (g) Cross section km 612.57





DISTANCE FROM SOURCE (km)	ELEVATION (m)
814.210	297.145
814.430	297.124
814.779	297.103
815.040	297.078

Figure B2. Hay River near Meander River: (a) Plan (b) Longitudinal profile of water surface on day of survey.

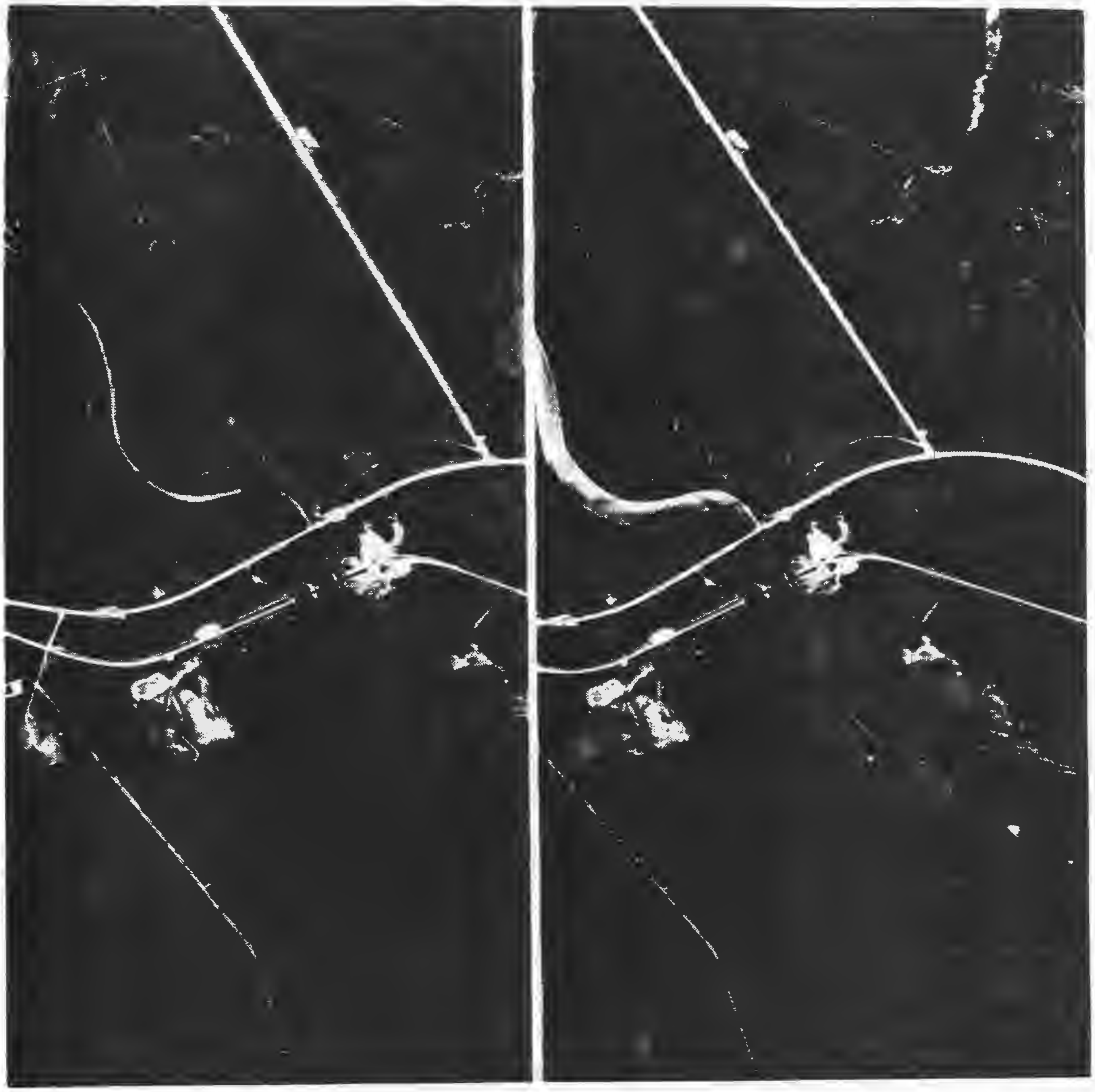
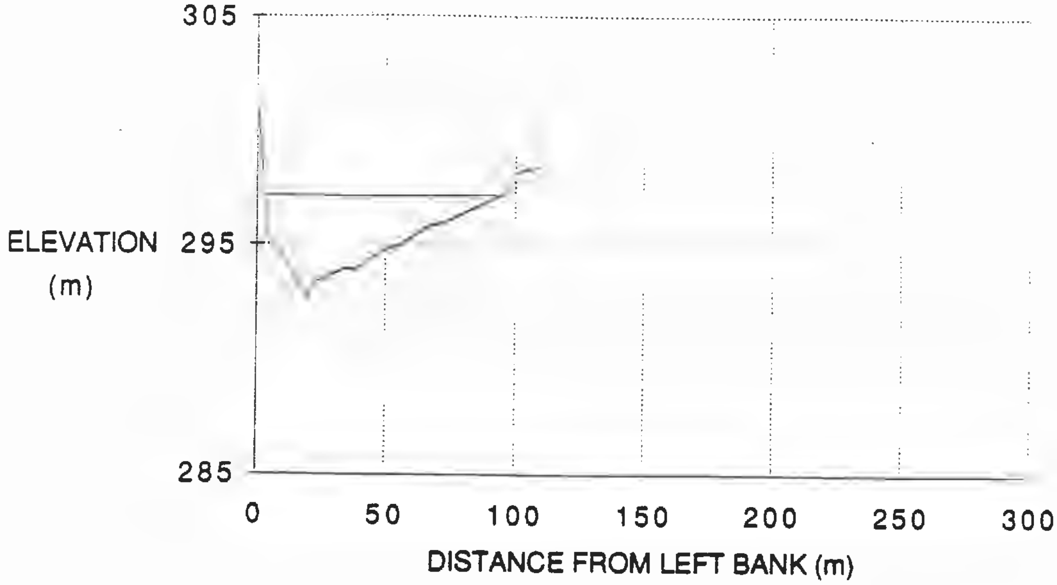


Figure B2 (continued). Hay River near Meander River (c) Airphoto of reach, 16 Aug 84, scale 1:25000

Figure B2 (continued). Hay River near Meander River (d) Reach photos (i) Looking downstream across section km 815.05 (ii) right bank at section km 814.21.

Figure B2 (continued). Hay River near Meander River (d) Reach photos (continued) (iii) Left bank and (iv) right bank at section km 814.21.



CROSS SECTION km 814.210

Location: 0.5 km U/S from Highway 35 bridge crossing
Left bank:
mature poplar
willow and brush to waters edge
Right bank:
cut bank
mature poplar at top of bank

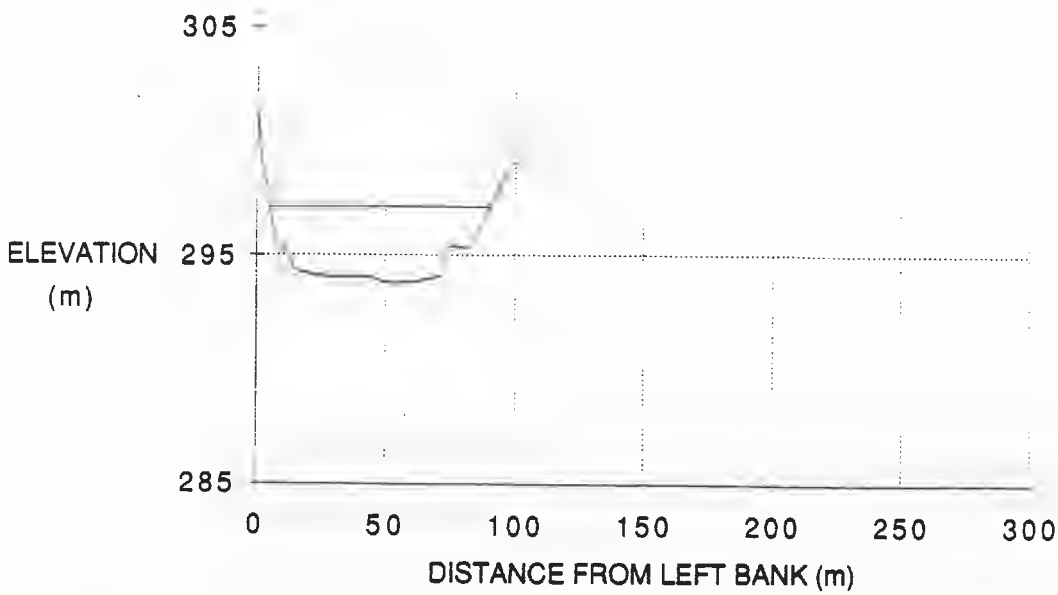
Water level on July 20,1987: 297.145 m

Q = 98 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	301.51
3.0	297.15
3.0	297.15
3.7	295.62
14.7	293.64
19.7	292.63
21.7	293.34
34.7	293.98
37.7	293.85
50.7	294.86
54.7	294.92
66.7	295.80
74.7	296.08
94.7	297.15
94.7	297.15
101.2	298.18
108.7	298.37
109.7	299.77

Elevations geodetic.

Figure B2 (continued). Hay River near Meander River (e) Cross section km 814.21



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	301.63
2.0	299.03
4.5	297.74
4.9	297.12
4.9	297.12
7.5	295.60
12.5	295.05
14.5	294.38
22.5	294.11
30.5	294.05
43.5	294.08
50.5	293.77
60.5	293.83
71.5	294.14
72.5	294.99
74.5	295.42
82.3	295.36
90.3	297.12
90.9	297.12
95.9	298.44
99.9	299.08

CROSS SECTION km 814.430

Location: 0.3 km U/S from Highway 35 bridge crossing

Left bank:

mature poplar
willow and brush to waters edge

Right bank:

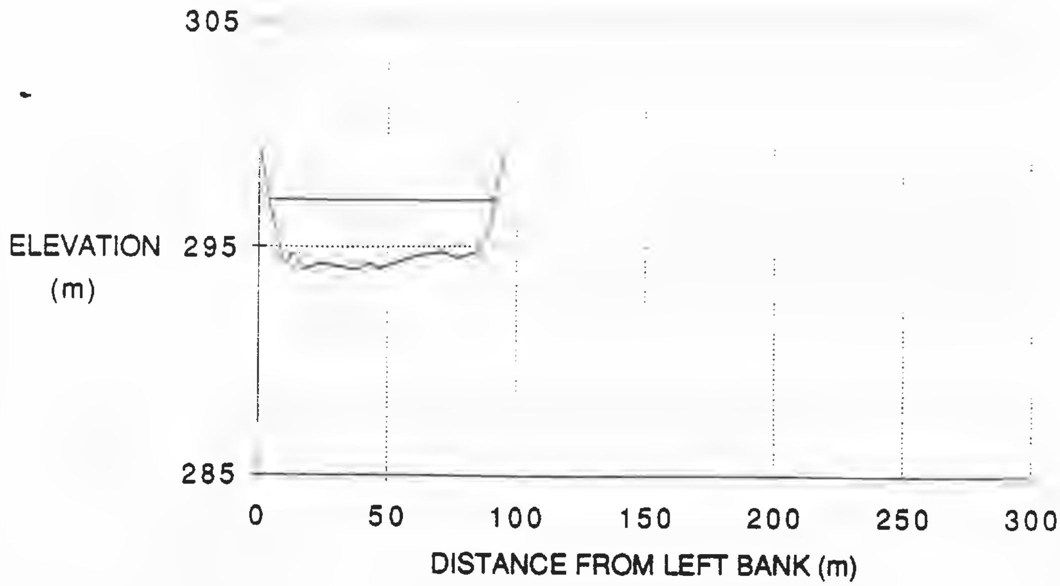
cut bank
mature poplar at top of bank

Water level on July 20,1987: 297.124 m

Q = 98 CMS

Elevations geodetic.

Figure B2 (continued). Hay River near Meander River (f) Cross section km 814.43



CROSS SECTION km 815.040

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	299.57
3.5	297.08
3.5	297.08
9.9	294.18
13.5	294.76
17.5	294.03
22.5	294.33
29.5	294.21
37.5	294.03
43.5	294.33
46.5	294.09
60.5	294.64
71.5	294.79
76.5	294.52
83.5	294.73
90.5	295.92
90.9	297.08
90.9	297.08
95.3	298.29

Location: 0.3 km D/S from Highway 35 bridge crossing

Left bank:

mature poplar at top of bank
willow and brush to waters edge

Right bank:

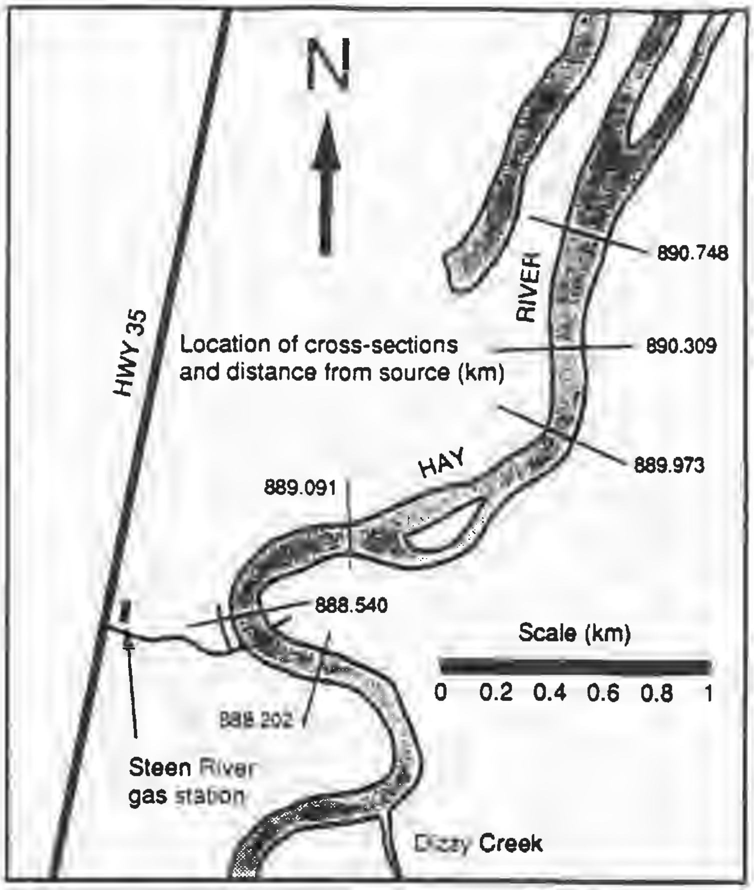
grass
willow
poplar

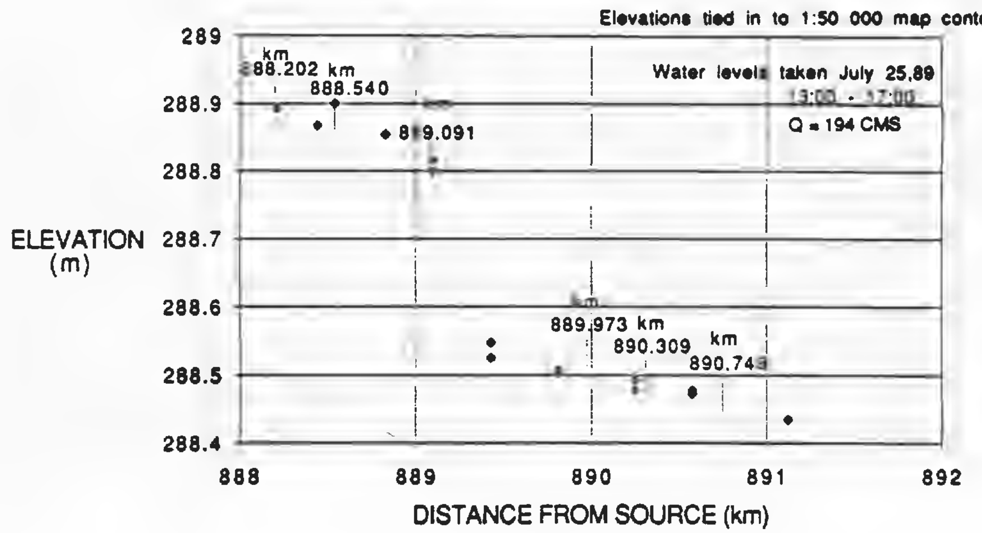
Water level on July 20,1987: 297.078 m

Q = 98 CMS

Elevations geodetic.

Figure B2 (continued). Hay River near Meander River (g) Cross section km 815.04





DISTANCE FROM SOURCE (km)	ELEVATION (m)
888.202	288.893
888.444	288.867
888.54	288.9
888.676	[illegible]
[illegible]	[illegible]
889.091	288.8
889.097	288.815
889.426	288.526
889.426	288.548
889.811	[illegible]
889.811	288.503
890.251	288.493
890.251	288.478
890.576	288.473
[illegible]	[illegible]
[illegible]	[illegible]

Figure B3 Hay River near Steen River (a) Plan (b) Longitudinal profile of water surface on day of survey

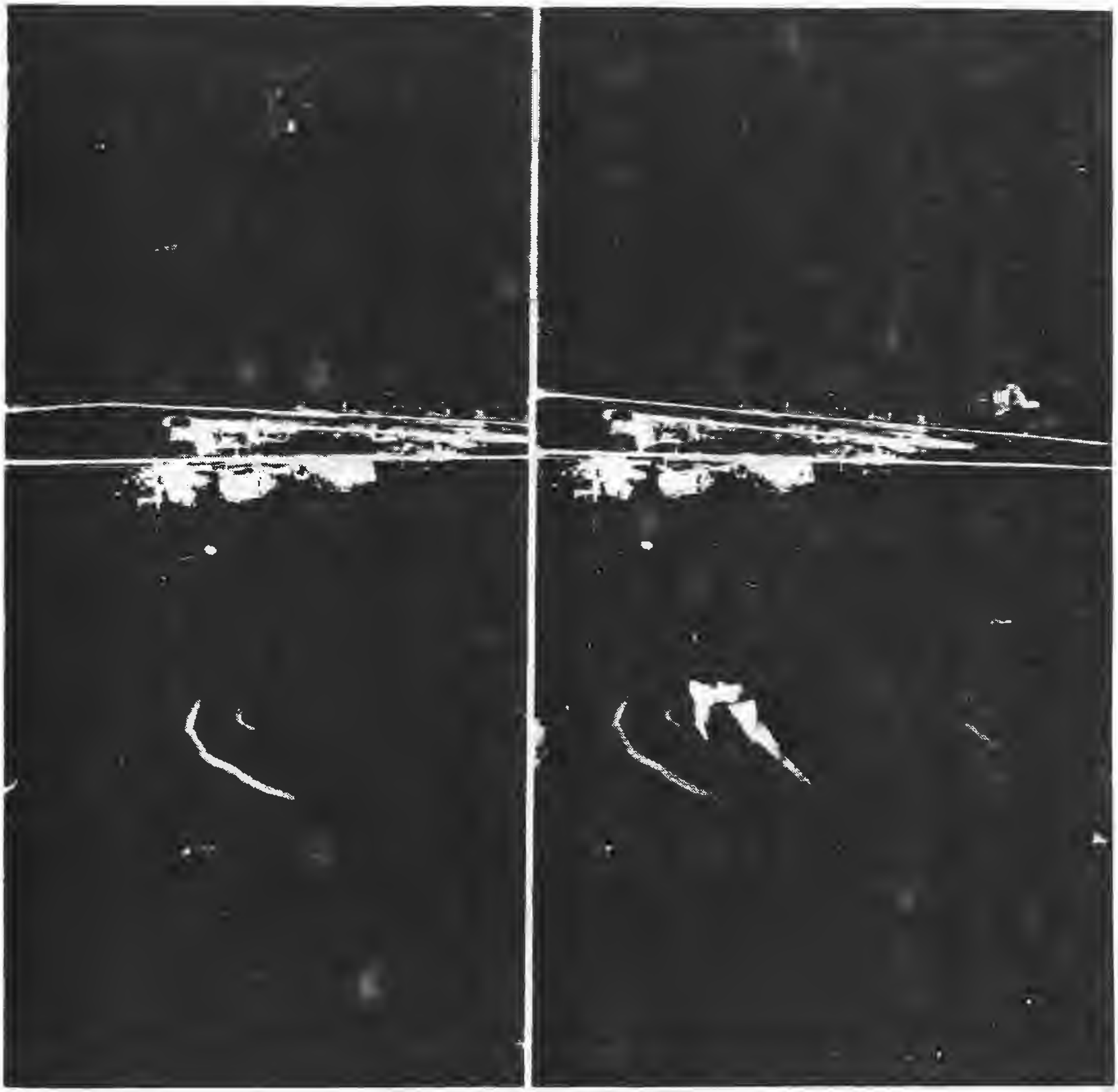


Figure B3 (continued). Hay River near Steen River (c) Airphotos of reach, 17 Aug 84, scale 1:25000

Figure B3 (continued). Hay River near Steen River (d) Reach photos (i) Left bank and (ii) Right bank at section km 888.54.

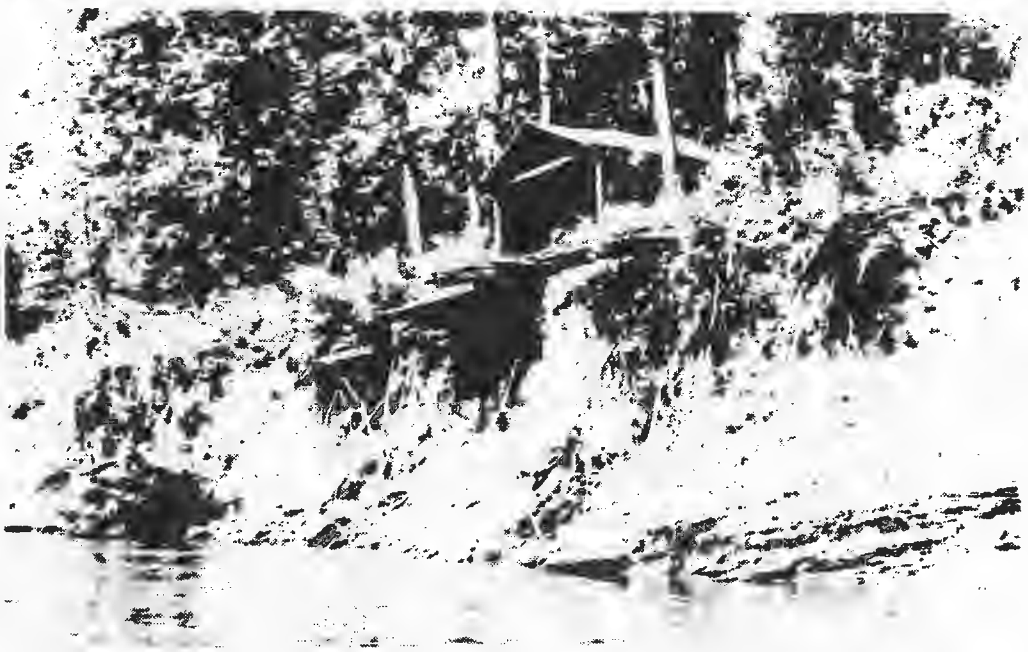
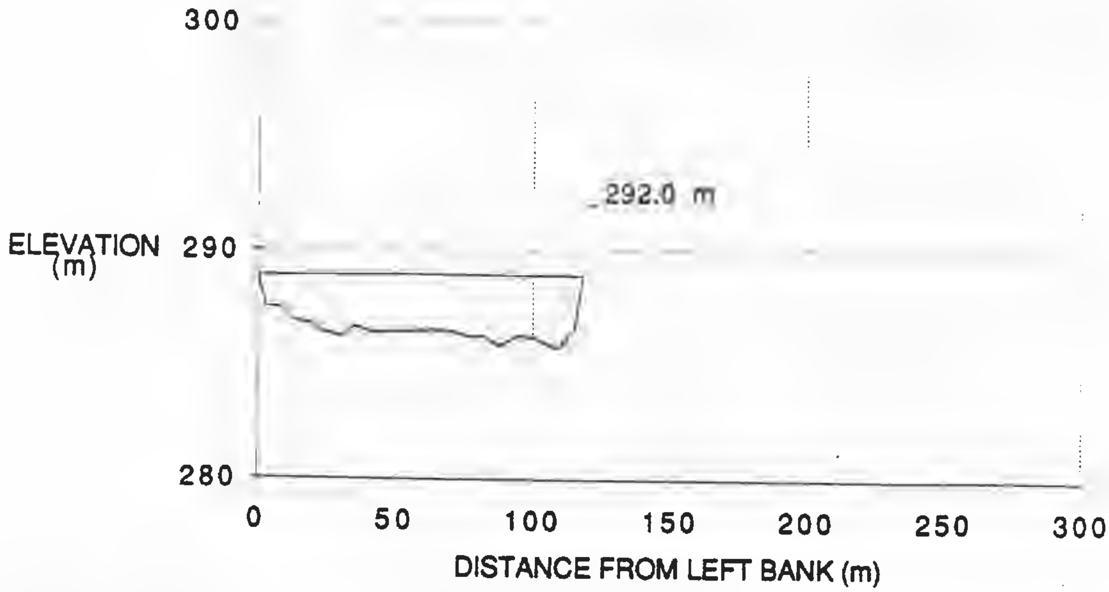


Figure B3 (continued). Hay River near Steen River (d) Reach photos (continued) (iii) Left bank and (iv) Right bank at section km 888.20.



CROSS SECTION km 888.202

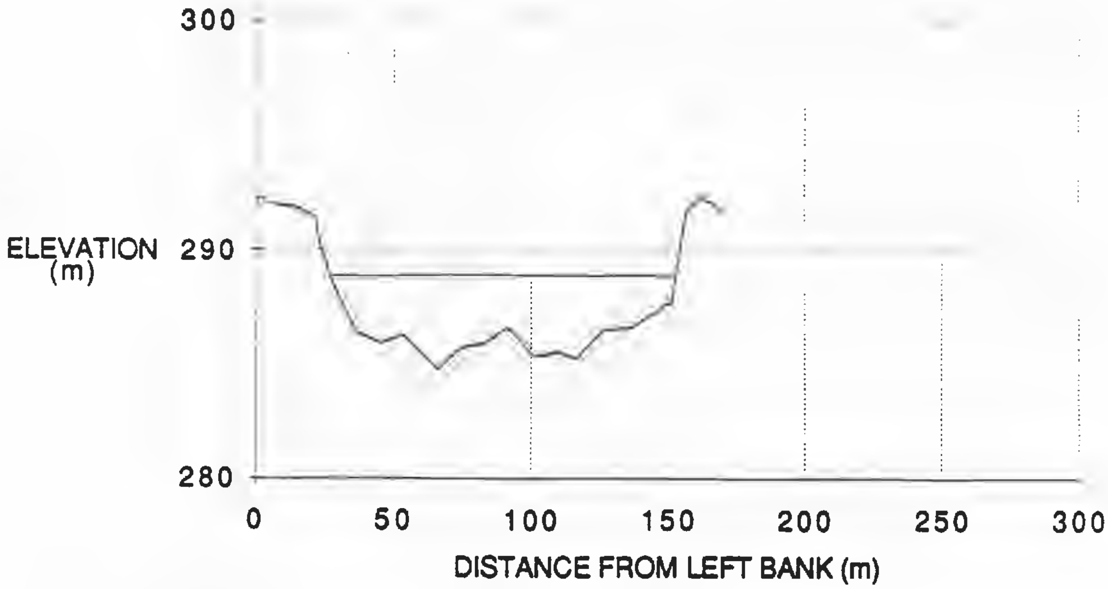
Location: 0.24 km U/S from access road from Steen River gas station.

Q = 194 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	288.891
3.0	287.491
8.0	287.491
13.0	286.891
20.0	286.791
23.0	286.391
31.0	286.191
35.0	286.641
42.0	286.341
50.0	286.391
68.0	286.491
78.0	286.191
81.0	286.291
88.0	285.791
94.0	286.241
100.0	286.191
109.0	285.691
115.0	286.541
118.0	288.891

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B3 (continued). Hay River near Steen River (e) Partial cross-section km 888.20.



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	292.148
1.8	292.378
4.1	292.04
15.0	291.843
21.4	291.414
23.0	290.288
25.5	289.443
26.5	288.896
36.5	286.396
45.5	285.946
53.5	286.296
66.5	284.746
74.5	285.696
83.5	285.896
91.5	286.596
101.5	285.296
109.5	285.546
116.5	285.246
126.5	286.546
138.5	286.596
146.5	287.396
149.5	287.546
151.5	287.696
153.0	288.896
156.6	291.7
161.1	292.33
167.7	291.946
170.2	291.665

CROSS SECTION km 888.540

Location: 0.1 km D/S from access road from Steen River gas station.

Left bank:

0 - 1.8 m	willow
1.8 - 15.0 m	dirt road
15.0 - 21.4 m	willow
21.4 - 26.5 m	silt

Right bank:

153.0 - 156.6 m	silt and small willow
156.6 - 161.1 m	willow
161.1 - 167.7 m	small poplar and willow
167.7 - 170.2 m	mature poplar

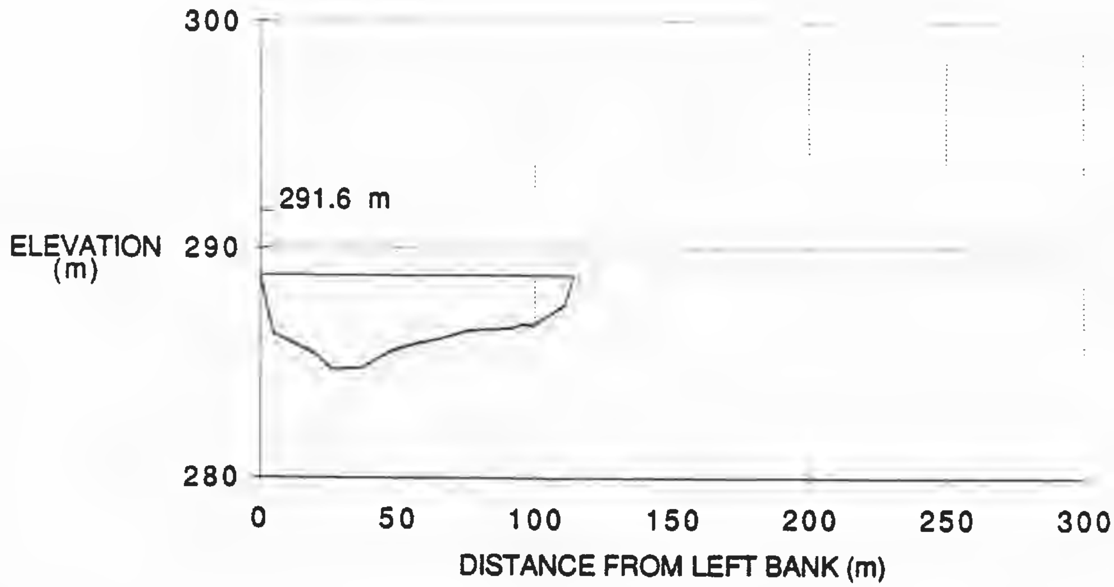
Water level on July 21,1989: 288.896 m @ 13:52
Q = 194 CMS

TBM (L.B.): Elev: 288.906 m @ 15.0 m on cross section.
Spike in 150 mm ϕ slanted poplar.

TBM (R.B.): Elev: 288.434 m @ 167.7 m on cross section.
Spike in 125 mm ϕ poplar.

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B3 (continued). Hay River near Steen River (f) Cross section km 888.54



CROSS SECTION km 889.091

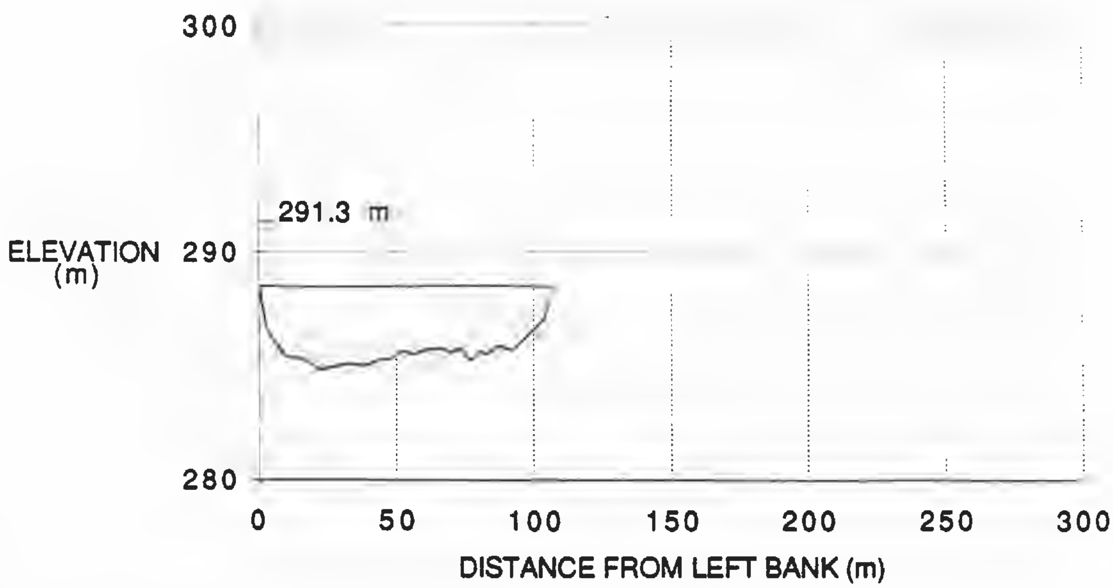
Location: 0.65 km D/S from access road from Steen River gas station.

Q = 194 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	288.807
3.0	287.157
5.0	286.257
20.0	285.357
26.0	284.707
37.0	284.757
47.0	285.507
57.0	285.857
66.0	286.057
75.0	286.407
91.0	286.507
96.0	286.707
99.0	286.607
111.0	287.557
114.0	288.807

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B3 (continued). Hay River near Steen River (g) Partial cross section km 889.09



CROSS SECTION km 889.973

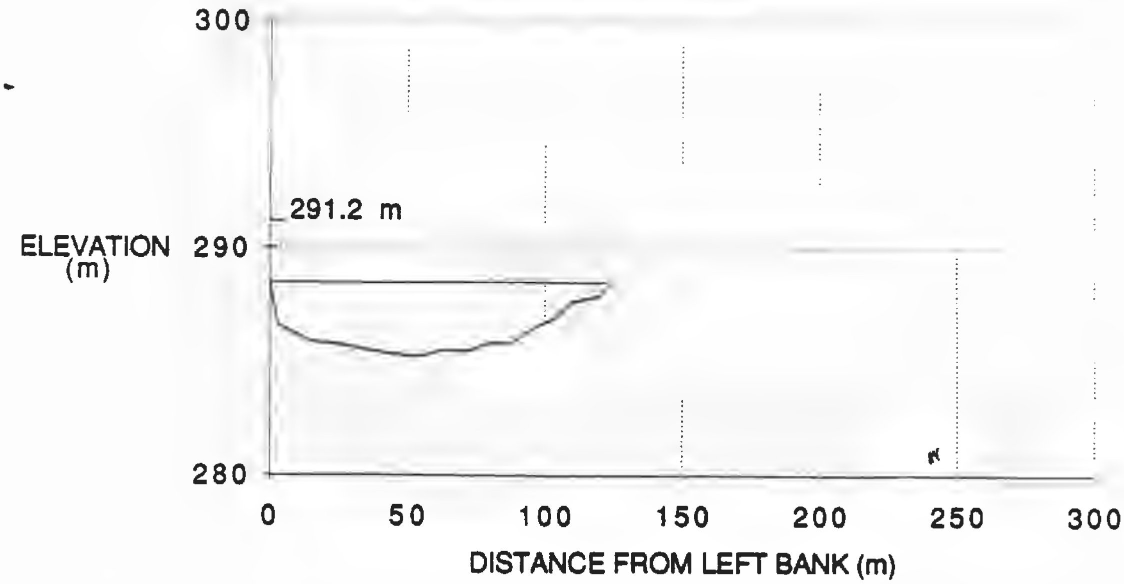
Location: 1.53 km D/S from access road from Steen River gas station.

Q = 194 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	288.498
3.0	286.548
10.0	285.398
16.0	285.348
22.0	284.848
33.0	285.148
38.0	284.998
44.0	285.298
48.0	285.298
52.0	285.698
56.0	285.498
60.0	285.698
66.0	285.798
70.0	285.548
73.0	285.798
77.0	285.198
80.0	285.648
83.0	285.498
87.0	285.898
92.0	285.648
104.0	287.048
107.0	288.498

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B3 (continued). Hay River near Steen River (h) Partial cross-section km 889.97



CROSS SECTION km 890.309

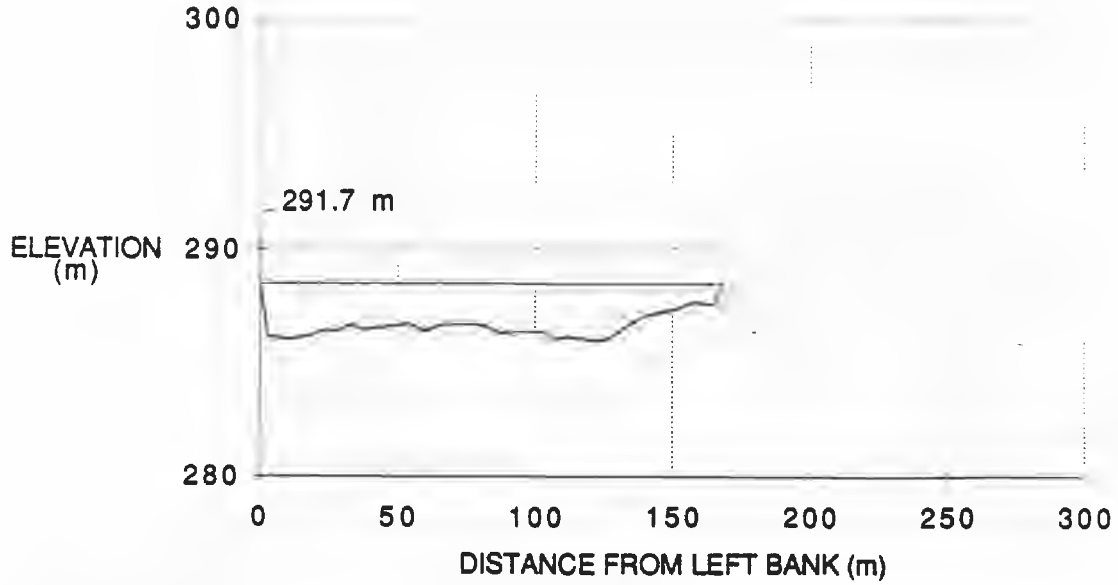
Location: 1.87 km D/S from access road from Steen River gas station.

Q = 194 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	288.482
3.0	286.632
14.7	285.882
24.0	285.782
43.0	285.332
54.0	285.182
63.0	285.482
71.0	285.432
80.0	285.832
87.0	285.782
98.0	286.632
102.0	286.782
109.0	287.582
120.0	287.882
123.0	288.482

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B3 (continued). Hay River near Steen River (i) Partial cross section km 890.31



CROSS SECTION km 890.748

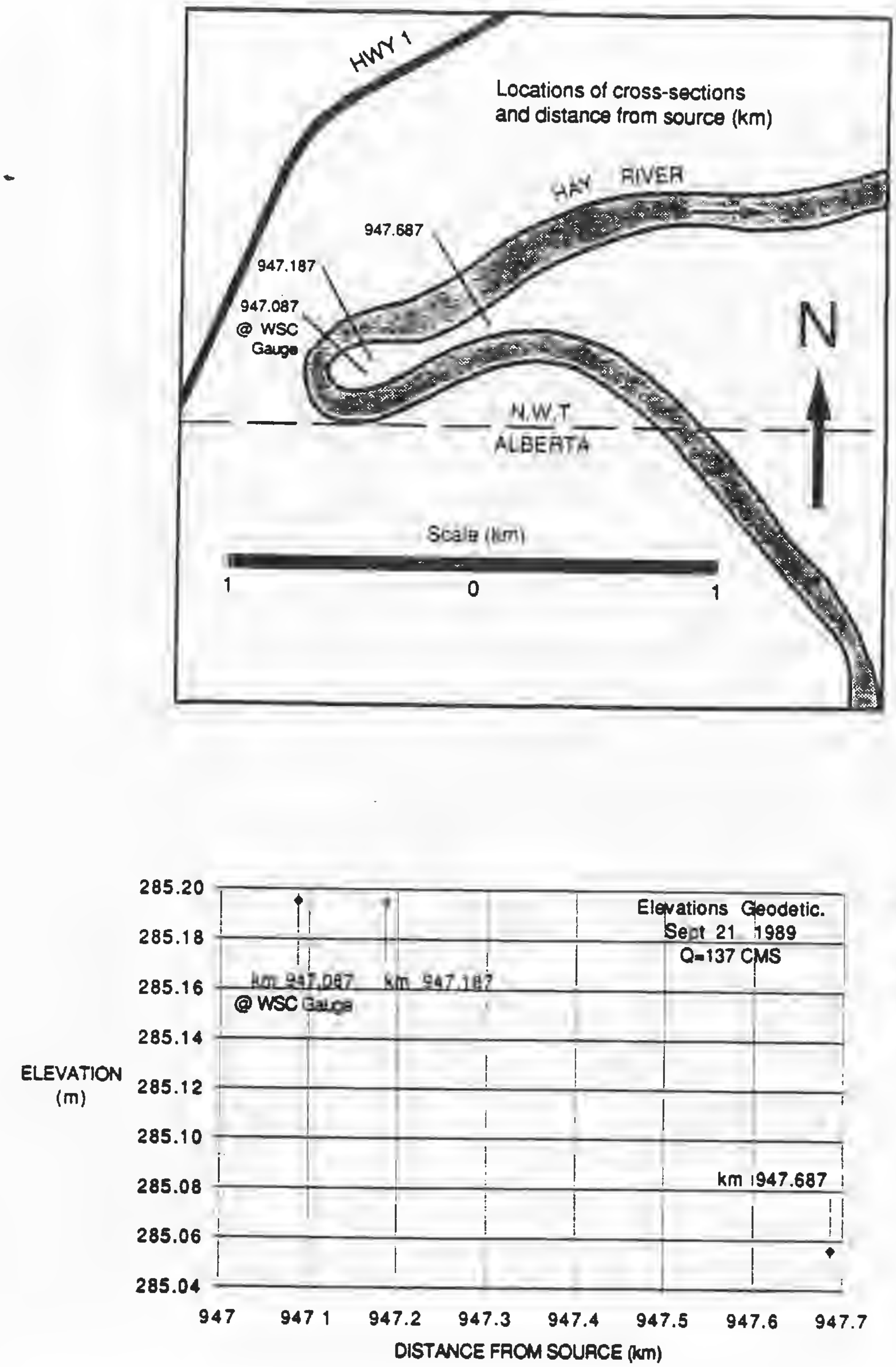
Location: 2.31 km D/S from access road from Steen River gas station.

Q = 194 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	288.461
3.0	286.161
10.0	286.011
18.0	286.161
24.0	286.461
28.0	286.361
33.0	286.711
37.0	286.411
48.0	286.611
54.0	286.711
60.0	286.361
66.0	286.661
79.0	286.661
84.0	286.511
87.0	286.261
90.0	286.361
103.0	286.361
107.0	286.011
111.0	286.161
120.0	286.011
123.0	285.961
127.0	286.061
136.0	286.811
140.0	287.061
152.0	287.361
157.0	287.661
163.0	287.611
165.0	287.661
168.0	288.461

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B3 (continued). Hay River near Steen River (j) Partial cross-section km 890.75



DISTANCE FROM SOURCE (km)	ELEVATION (m)
947.087	285.19
947.187	285.195
947.687	285.056

Figure B4. Hay River at the border (a) Plan (b) Longitudinal profile of water surface on day of survey.

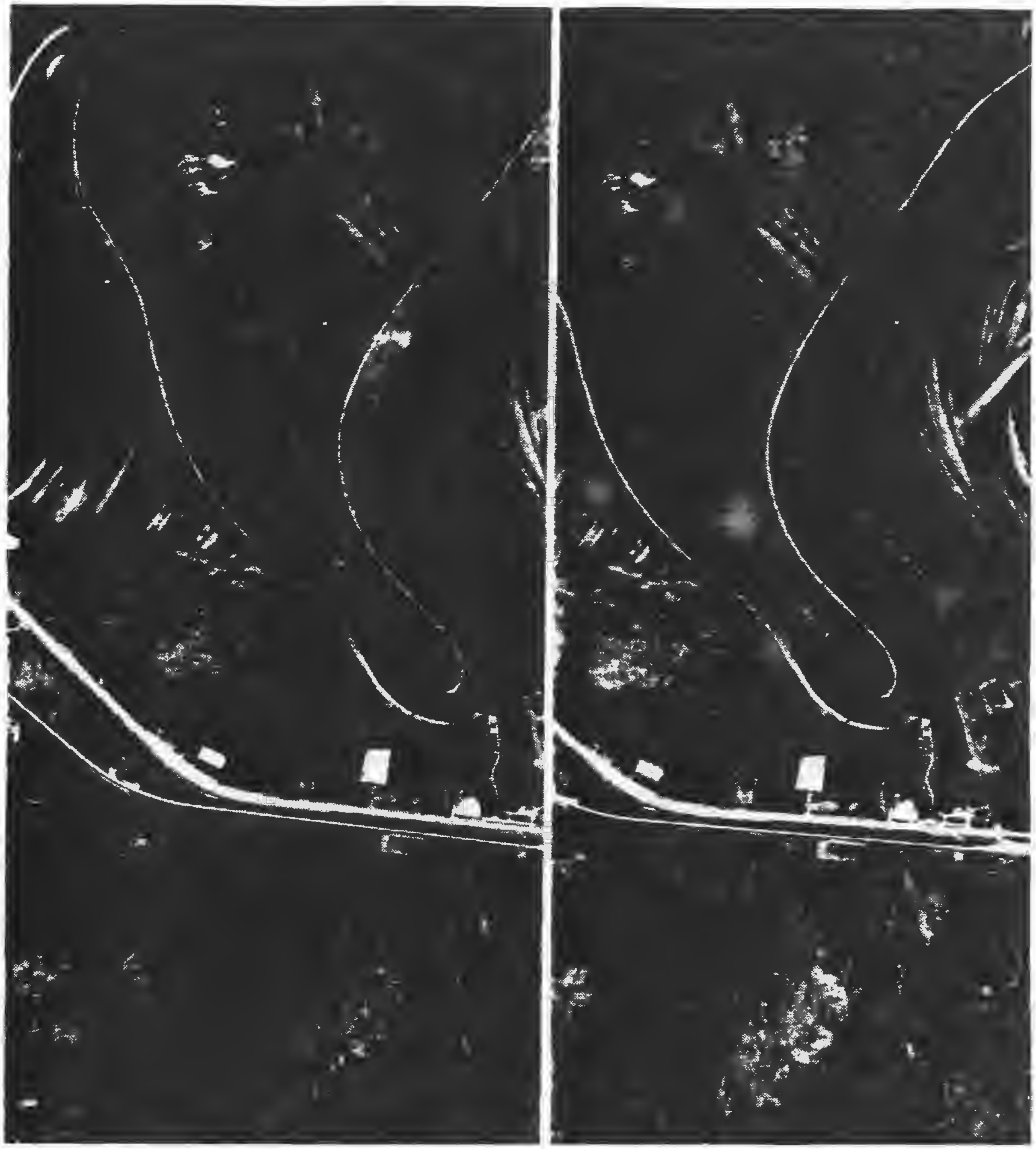
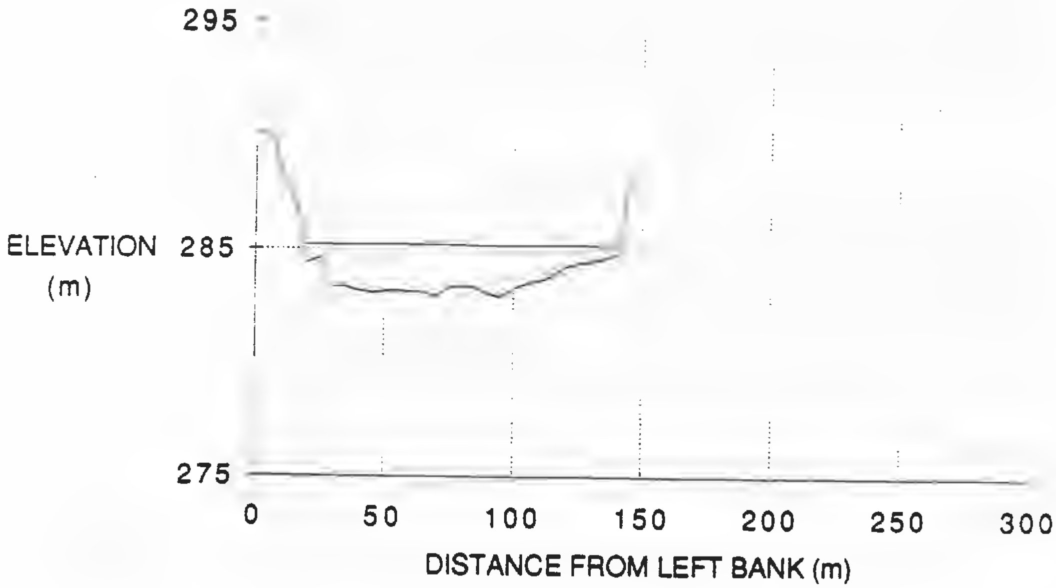


Figure B4 (continued). Hay River at the border (c) Airphotos of the reach. 17 Aug 84, scale 1:25000



CROSS SECTION km 947.087

Location: At Alberta/N.W.T border gauge site.
Left bank:

Right bank:

	140.5 - 142.0 m	grass
	142.0 - 147.0 m	willows
	147.0 - 149.0 m	trees

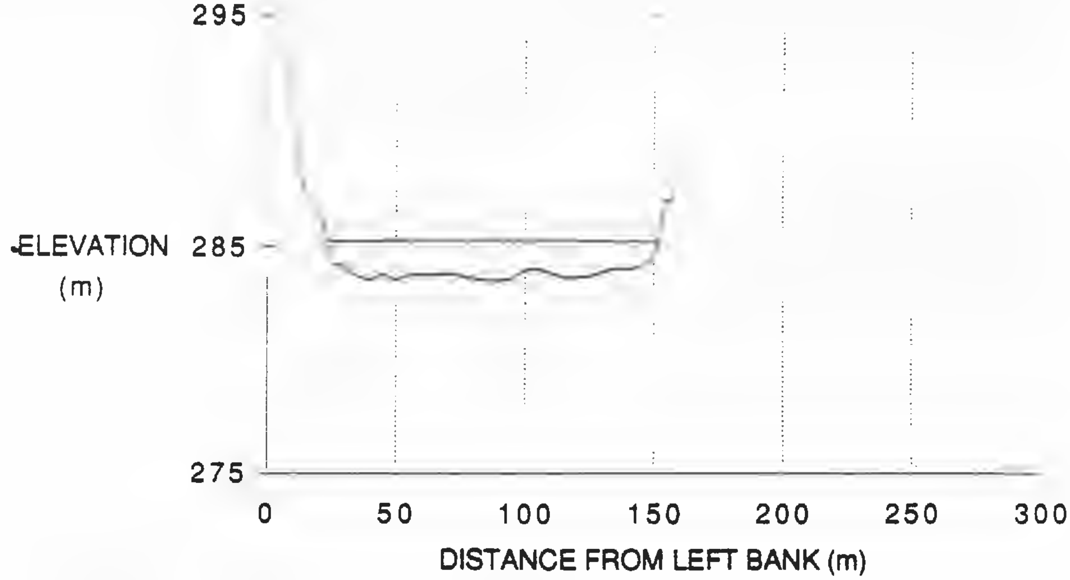
Water level on Sept 21,1989: 285.195 m

Q = 137 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
1.0	290.10
5.0	290.15
10.0	288.52
14.0	287.34
18.0	286.27
19.0	285.20
20.0	284.40
25.0	284.65
30.0	283.37
35.0	283.36
40.0	283.20
45.0	283.10
50.0	283.16
55.0	283.20
60.0	283.12
65.0	283.12
70.0	282.90
75.0	283.34
80.0	283.40
85.0	283.36
90.0	283.10
95.0	282.94
100.0	283.35
105.0	283.56
110.0	283.60
115.0	283.87
120.0	284.27
125.0	284.41
130.0	284.40
135.0	284.62
140.0	284.85
140.5	285.20
142.0	285.47
144.0	287.59
147.0	288.16
149.0	287.86

Elevations geodetic.

Figure B4 (continued). Hay River at the border (d) Cross section km 947.09



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
6.0	292.66
8.0	292.21
10.0	290.91
13.0	288.06
16.0	287.19
21.0	286.45
23.0	285.20
25.0	284.21
30.0	284.10
35.0	283.66
40.0	283.48
45.0	283.76
50.0	283.47
55.0	283.67
60.0	283.72
65.0	283.67
70.0	283.78
75.0	283.66
80.0	283.55
85.0	283.45
90.0	283.43
95.0	283.54
100.0	283.88
105.0	283.95
110.0	283.75
115.0	283.57
120.0	283.59
125.0	283.64
130.0	283.86
135.0	284.00
140.0	284.00
145.0	284.10
150.0	284.62
151.5	285.20
154.0	287.03
156.0	286.90
158.0	287.43

Elevations geodetic.

CROSS SECTION km 947.187

Location: 0.1 km D/S from gauge.
Left bank:

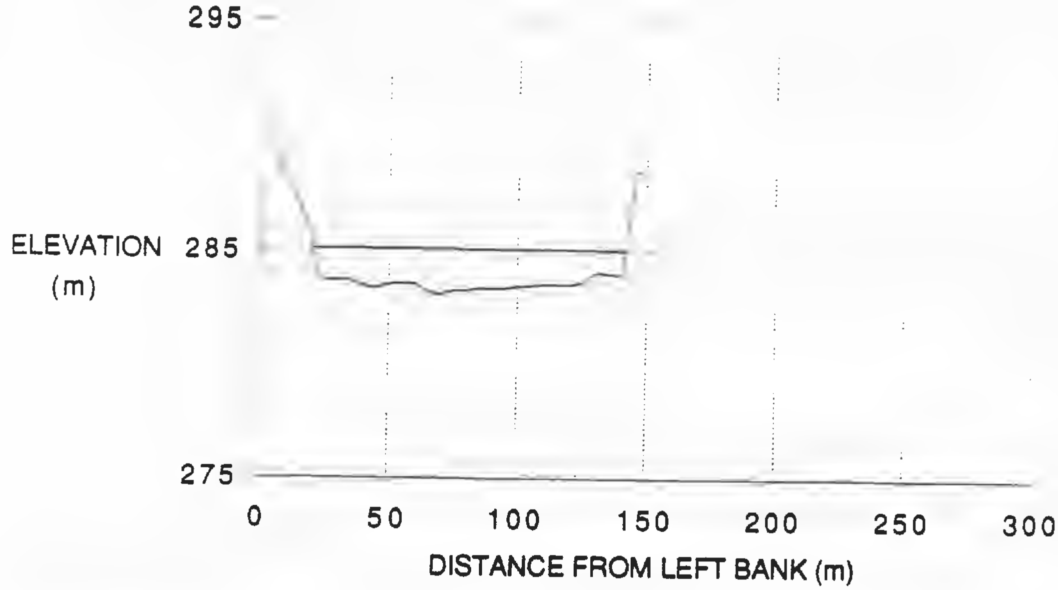
Right bank:

151.5 - 154.0 m	grass
154.0 - 158.0 m	willows
158.0 - .	trees(At distances > 158 m elevations roughly the same.)

Water level on Sept 21,1989: 285.195 m

Q = 137 CMS

Figure B4 (continued). Hay River at the border (e) Cross section km 947.19



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
4.5	290.86
5.0	289.98
8.0	288.44
12.0	287.94
16.0	286.85
20.0	285.70
20.5	285.06
22.0	284.17
25.0	283.62
30.0	283.66
35.0	283.64
40.0	283.39
45.0	283.28
50.0	283.52
55.0	283.52
60.0	283.53
65.0	283.13
70.0	283.08
75.0	283.24
80.0	283.25
85.0	283.32
90.0	283.30
95.0	283.31
100.0	283.44
105.0	283.42
110.0	283.51
115.0	283.48
120.0	283.47
125.0	283.58
130.0	284.02
135.0	284.01
140.0	283.91
142.0	285.06
146.0	288.46
149.0	288.46

CROSS SECTION km 947.687

Location: 0.6 km D/S from gauge.
Left bank:

Right bank:

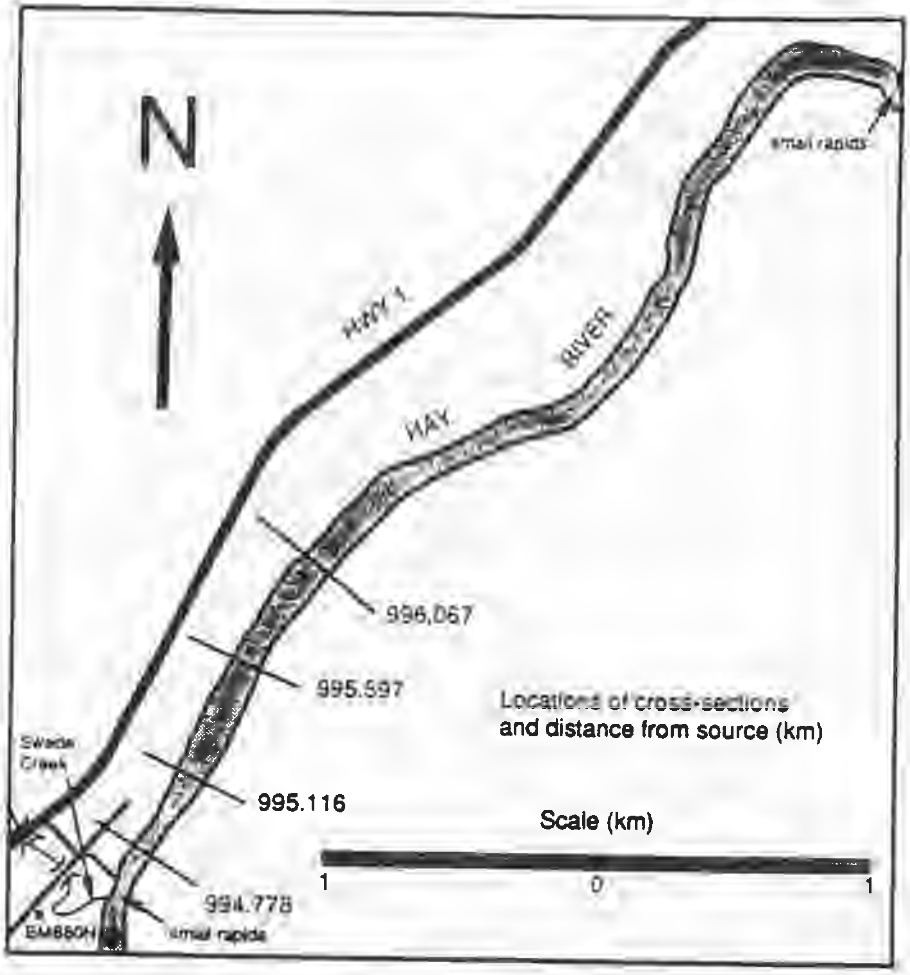
142.0 - 149.0 m willows

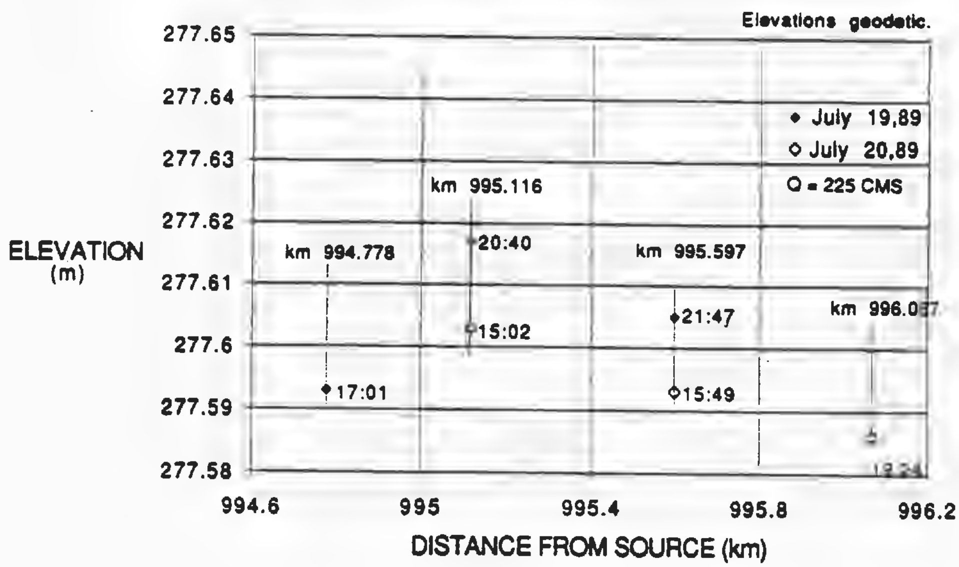
Water level on Sept 21,1989: 285.056 m

Q = 137 CMS

Elevations geodetic.

Figure B4 (continued). Hay River at the border (f) Cross section km 947.69





DISTANCE FROM SOURCE (km)	ELEVATION (m)
994.778	277.59
994.778	277.59
995.116	[illegible]77.61
995.116	[illegible]77.61
995.597	[illegible]77.60
995.597	277.59
995.116	[illegible]77.603
995.597	[illegible]77.59
996.[illegible]	277.5[illegible]
996.[illegible]	[illegible]77.586

Figure B5. Hay River near Swede Creek (a) Plan (b) Longitudinal water surface profile on day of survey

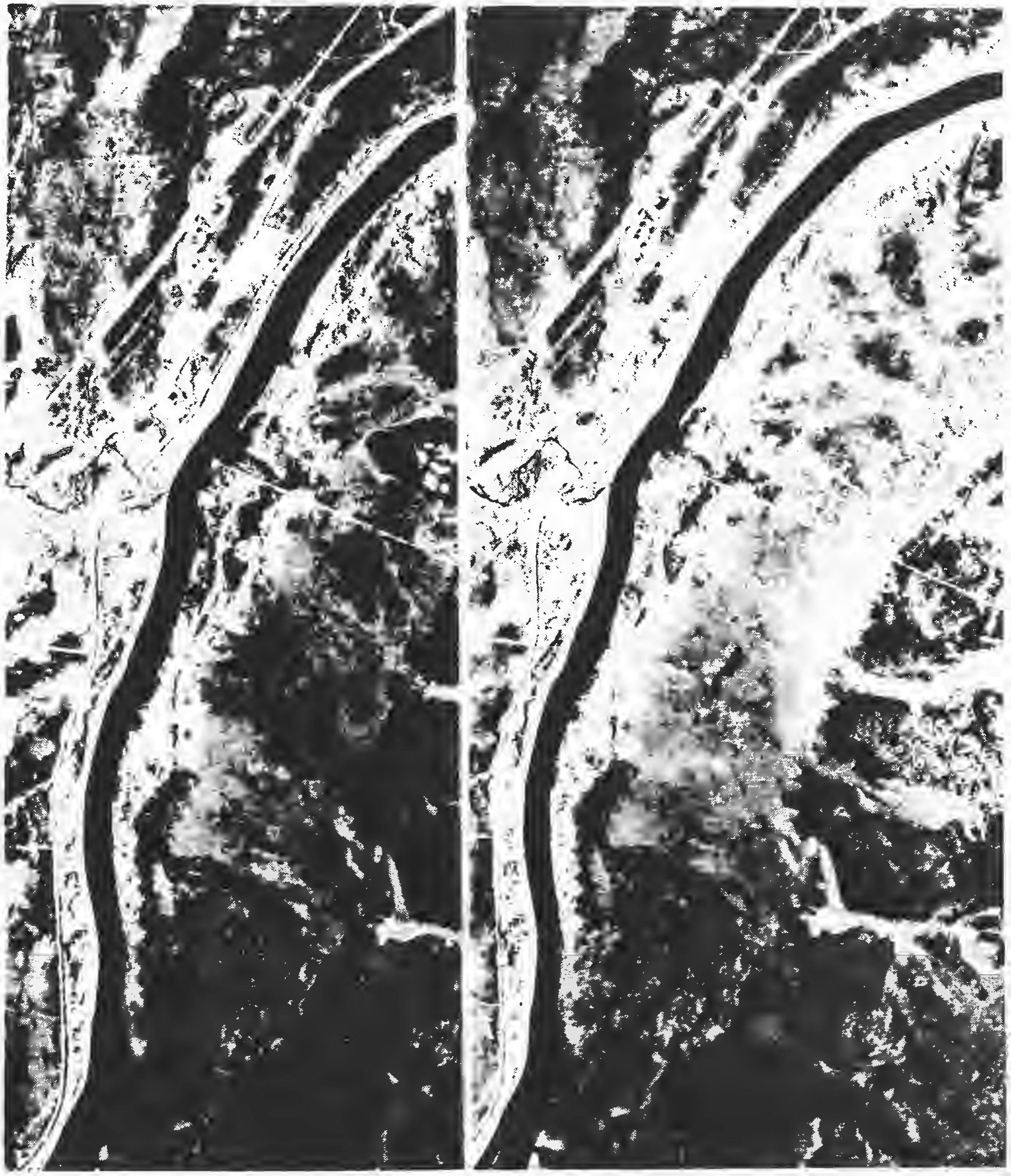


Figure B5 (continued) Hay River near Swede Creek (c) Airphotos of reach, June 1973, Scale 1:26200

Figure B5 (continued). Hay River near Swede Creek (d) Reach photos (i) Looking upstream over section km 995.60.

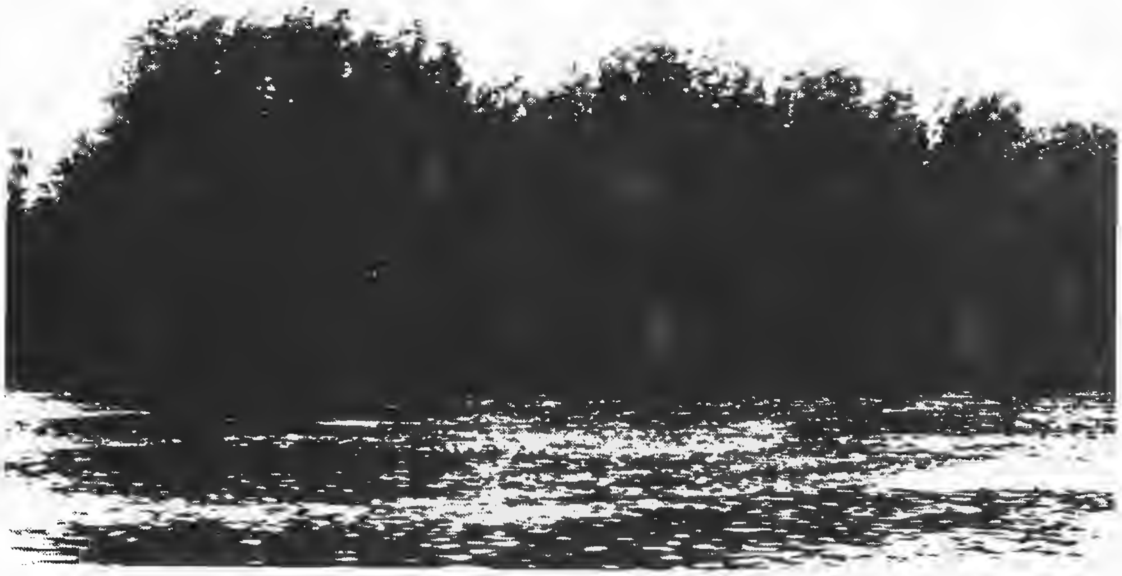
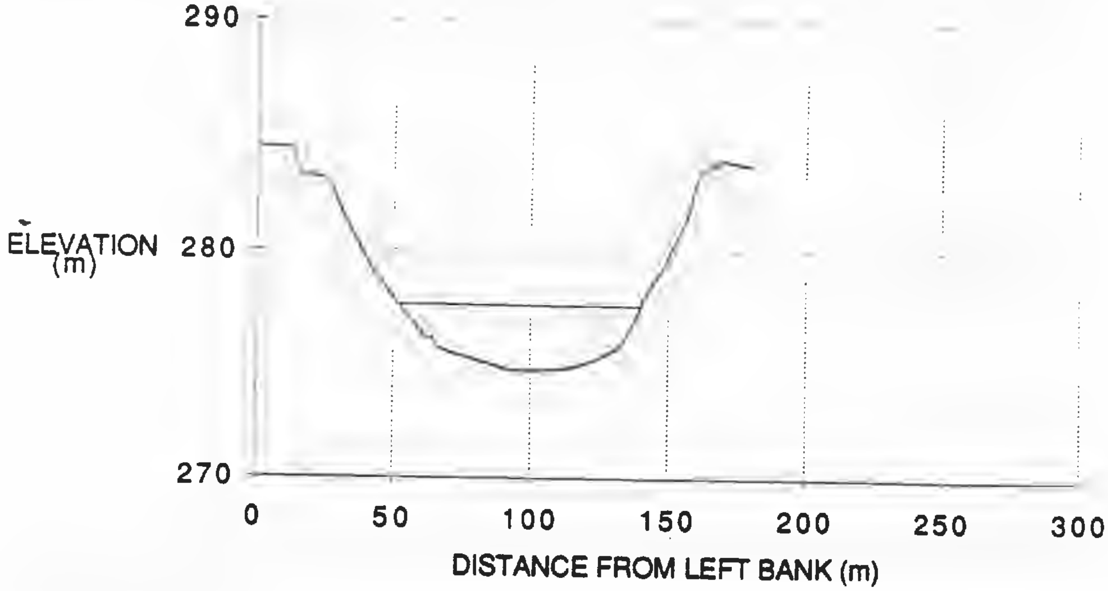


Figure B5 (continued) Hay River near Swede Creek (d) Reach photos (continued) (ii) Left bank and (iii) Right bank at section km 995.60.



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	284.59
13.2	284.541
14.4	283.688
16.8	283.379
24.2	283.31
27.4	282.816
30.1	281.928
32.2	281.353
42.3	279.126
51.9	277.593
61.9	276.143
63.9	276.153
65.9	275.743
70.9	275.493
81.9	275.143
91.9	274.793
100.9	274.743
109.9	274.843
114.9	274.893
122.9	275.193
131.9	275.793
138.1	276.943
140.1	277.593
140.4	277.847
147.9	279.724
155.5	281.176
157.9	281.911
159.7	282.64
160.5	283.151
163.7	283.682
169.0	284.01
180.6	283.789

Elevations geodetic.

CROSS SECTION km 994.778

Location: 0.21 km D/S from Swede Creek.

Left bank:

0 - 30.1 m	small poplar
30.1 - 32.2 m	1 m tall brush
32.2 - 42.3 m	grass
42.3 - 51.9 m	gravel and bedrock , D_{50} = 80mm

Right bank:

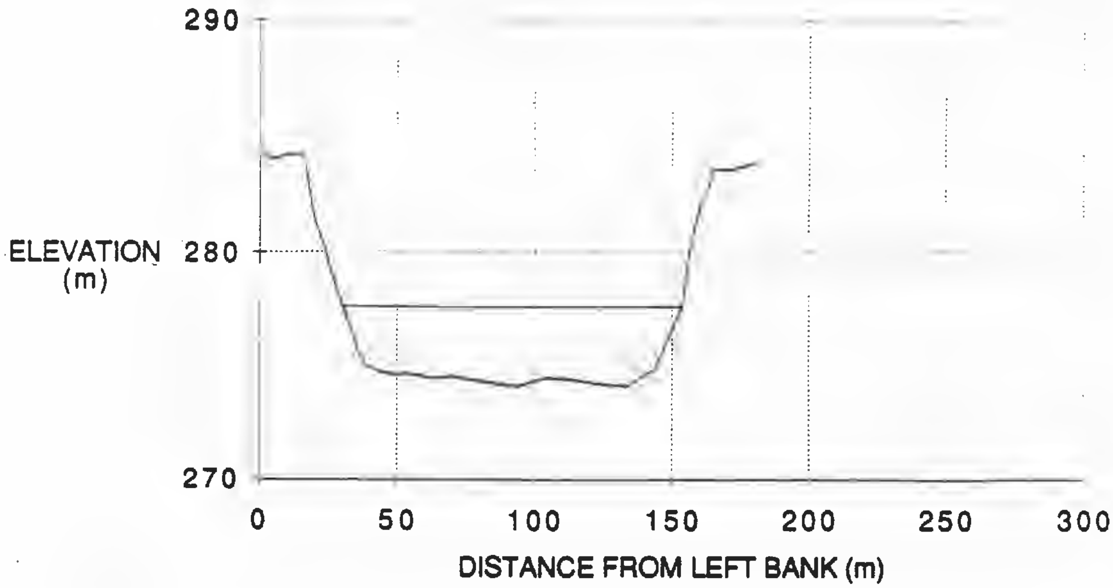
140.1 - 147.9 m	silt with sparse gravel,grass
147.9 - 155.5 m	0.3 m tall brush
155.5 - 160.5 m	1 m tall brush
160.5 - 180.6 m	mature spruce and poplar

Water level on July 19,1989: 277.593 m @ 15:32
Q = 225 CMS

TBM (L.B.): Elev: 283.146 m @ 27.4 m on cross section.
Spike in 180 mm ϕ poplar.

TBM (R.B.): Elev: 283.565 m @ 160.5 m on cross section.
Spike in 450 mm ϕ spruce.

Figure B5 (continued). Hay River near Swede Creek (e) Cross section km 994.78



CROSS SECTION km 995.166

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	284.436
4.3	284.048
10.3	284.305
15.2	284.274
16.6	283.885
19.7	281.542
23.7	280.175
25.3	279.389
29.1	278.014
30.2	277.617
33.8	276.417
35.8	275.617
38.8	275.067
43.8	274.717
48.8	274.617
53.8	274.667
63.8	274.417
69.8	274.567
73.8	274.417
93.8	274.067
103.8	274.467
113.8	274.367
121.8	274.217
135.8	274.007
145.1	274.567
153.8	277.617
156.5	280.050
160.4	282.029
161.6	282.298
163.0	282.889
165.1	283.594
171.7	283.579
181.1	283.909

Elevations geodetic.

Location: 0.55 km D/S from Swede Creek.

Left bank:

0 - 18.6 m	small poplar
18.6 - 25.3 m	0.6 m tall willow
25.3 - 29.1 m	grass
29.1 - 30.2 m	silt with sparse gravel

Right bank:

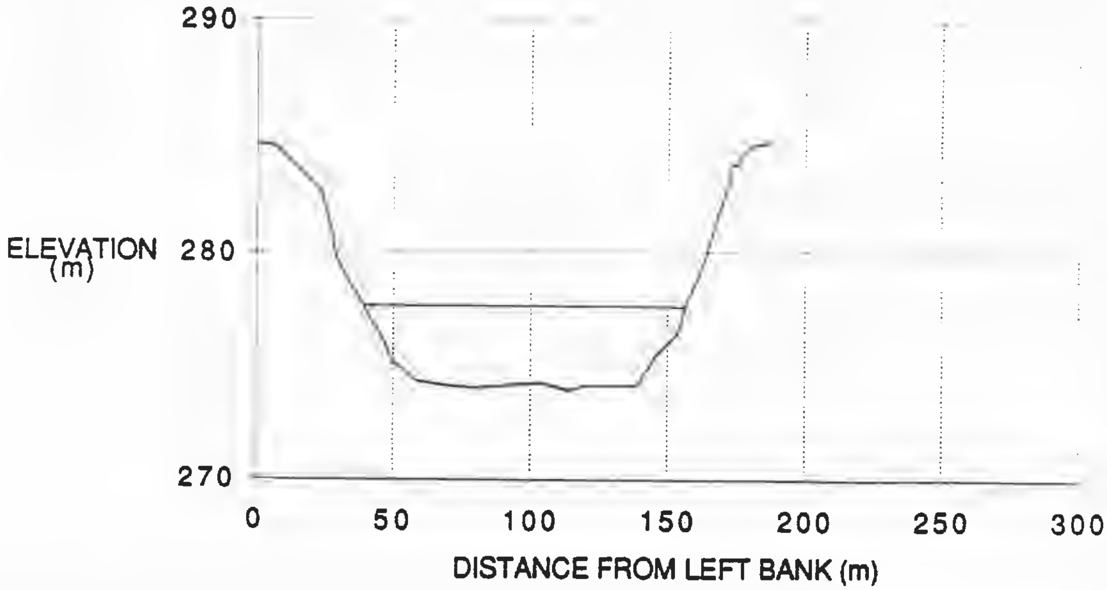
153.8 - 156.5 m	silt with sparse gravel
156.5 - 160.4 m	grass
160.4 - 163.0 m	0.6 m tall brush
163.0 - 181.1 m	small poplar

Water level on July 19,1989: 277.617 m @ 19:40
Q = 225 CMS

TBM (L.B.): Elev: 284.841 m @ 15.2 m on cross section.
Spike in 100 mm ϕ poplar behind 180 mm ϕ poplar

TBM (R.B.): Elev: 283.522 m @ 163.0 m on cross section.
Spike in 300 mm ϕ spruce.

Figure B5 (continued). Hay River near Swede Creek (f) Cross section km 995.17



CROSS SECTION km 995.597

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	284.748
6.3	284.648
13.1	283.935
19.7	283.194
23.6	282.595
24.3	282.356
25.1	281.872
27.9	280.71
28.2	280.215
30.3	279.367
36.7	278.112
39.1	277.593
44.1	276.543
46.6	276.093
49.1	275.293
58.1	274.343
69.1	274.143
79.1	274.043
104.1	274.293
114.1	273.943
120.1	274.143
139.1	274.143
145.1	275.343
154.1	276.543
156.4	277.593
163.3	279.583
167.9	281.628
172.4	283.075
172.7	283.918
175.6	283.837
177.0	284.207
180.1	284.587
182.6	284.677
187.6	284.767

Elevations geodetic.

Location: 1.0 km D/S from Swede Creek.

Left bank:

0 - 25.1 m	small poplar
25.1 - 28.2 m	2 m tall brush
28.2 - 39.1 m	silt with grass

Right bank:

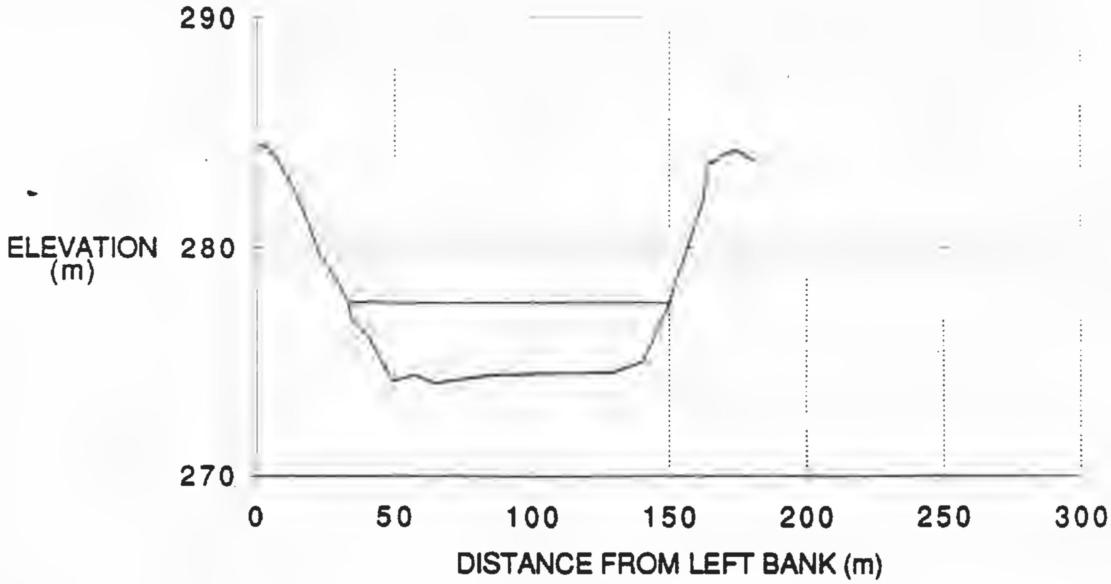
156.4 - 167.9 m	silt and gravel D_{50} = 40 mm
167.9 - 172.7 m	3 m tall brush
172.7 - 187.6 m	small poplar

Water level on July 20,1989: 277.593 m @ 15:02
Q = 225 CMS

TBM (L.B.): Elev: 282.838 m @ 23.6 m on cross section.
Spike in 100 mm ϕ poplar.

TBM (R.B.): Elev: 284.223 m @ 172.7 m on cross section.
Spike in 400 mm ϕ poplar

Figure B5 (continued). Hay River near Swede Creek (g) Cross section km 995.60



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	284.529
4.3	284.411
7.8	283.787
12.3	282.805
16.8	281.611
18.0	281.268
22.5	279.764
29.1	278.477
33.0	277.586
34.0	276.936
37.1	276.536
40.3	276.136
49.3	274.136
57.3	274.436
64.3	274.036
84.3	274.386
109.3	274.486
120.3	274.486
130.3	274.536
140.3	274.986
150.3	277.586
153.3	278.872
155.1	279.350
160.4	281.434
162.4	282.066
164.2	283.704
166.7	283.79
169.2	283.990
174.0	284.276
182.0	283.723

Elevations geodetic.

CROSS SECTION km 996.067

Location: 1.5 km D/S from Swede Creek.

Left bank:

0 - 16.8 m	small poplar
16.8 - 18.0 m	0.7 m tall brush
18.0 - 33.0 m	gravel D_{50} = 80 mm

Right bank:

150.3- 155.1 m	silt and grass
155.1 - 160.4 m	0.3 m tall brush
160.4 - 162.4 m	3 m tall brush
162.4 - 182.0 m	small poplar

Water level on July 20,1989: 277.586 m @ 19:24
Q = 225 CMS

TBM (L.B.): Elev: 284.060 m @ 7.8 m on cross section.
Spike in 100 mm ϕ poplar.

TBM (R.B.): Elev: 284.211 m @ 166.7 m on cross section.
Spike in 375 mm ϕ spruce.

Figure B5 (continued). Hay River near Swede Creek (h) Cross section km 996.07

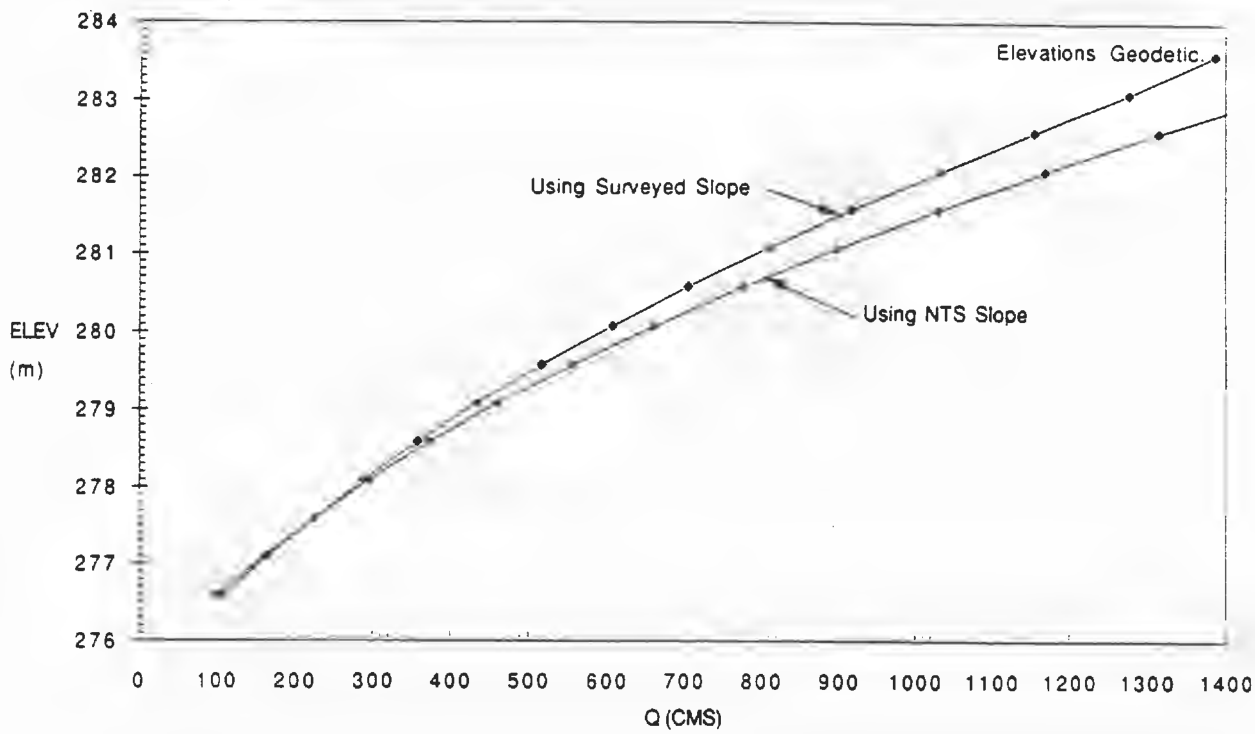
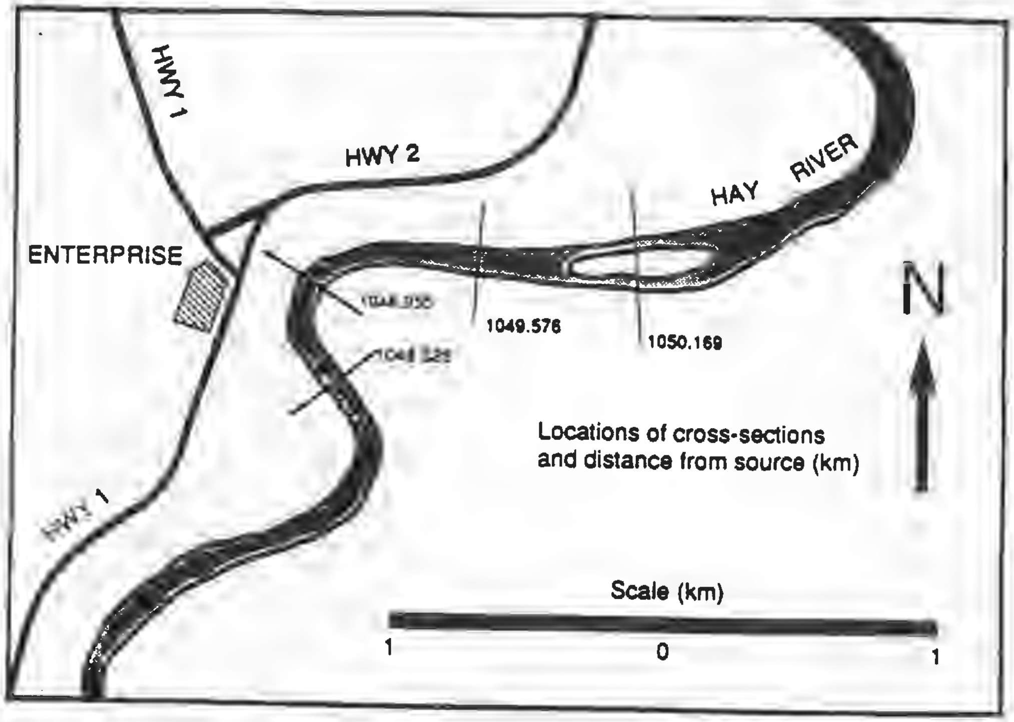
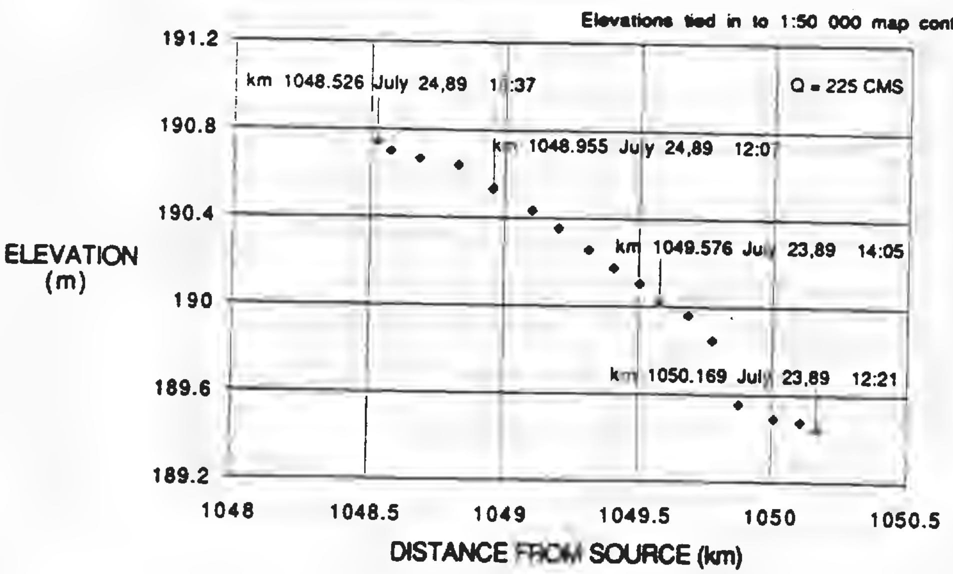


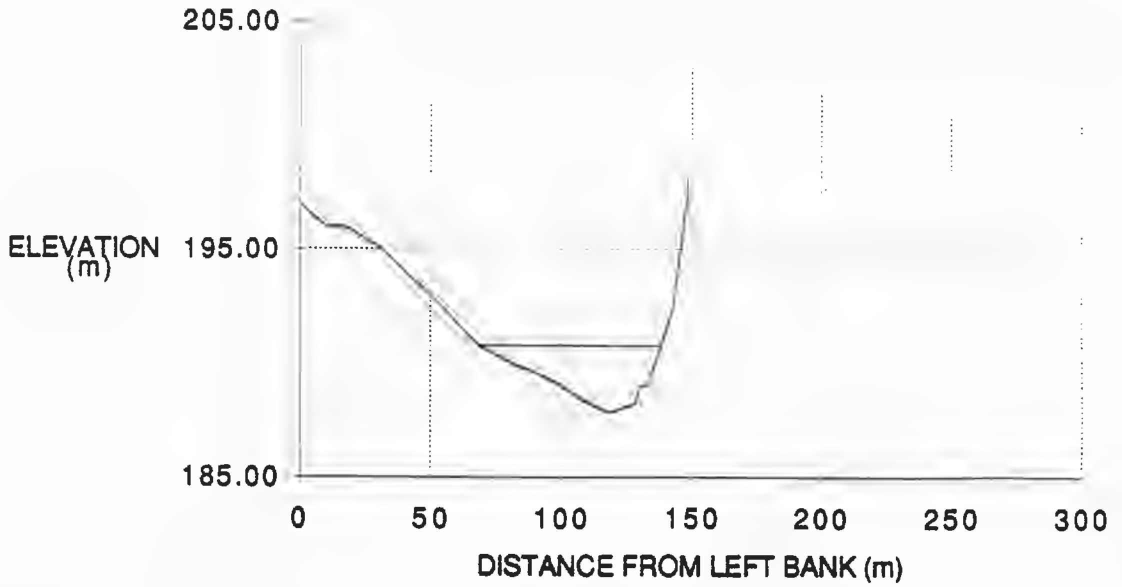
Figure B5 (continued). Hay River near Swede Creek (i) Calculated rating curves using surveyed water slope and slope estimated from NTS maps





DISTANCE FROMSOURCE (km)	ELEVATION (m)
1050.169	189.44[illegible]
1050.106	189.47[illegible]
1050.007	189.[illegible]
1049.877	189.[illegible]
1049.774	189.[illegible]
1049.684	189.[illegible]
1049.575	190.[illegible]
1049.575	190.[illegible]
1049.575	190.[illegible]
1049.575	190.[illegible]
1049.503	190.1[illegible]
1049.409	190.[illegible]
1049.316	190.[illegible]
1049.205	190.[illegible]
1049.104	190.[illegible]
1048.955	190.[illegible]
1048.855	190.[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
1048.629	190.[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
1048.526	190.749

Figure B6 Hay River near Enterprise (a) Plan (b) Longitudinal profile of water surface on day of survey



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	196.966
3.0	196.631
10.0	195.981
17.5	195.936
29.9	195.132
51.8	192.758
68.3	190.740
68.3	190.740
83.3	189.840
88.3	189.640
98.3	189.090
108.3	188.340
118.3	187.790
128.3	188.190
130.3	188.990
133.3	188.990
138.3	190.740
138.3	190.740
143.0	192.433
148.3	197.023
148.6	198.073

CROSS SECTION km 1048.526

Location: 0.4 km U/S from Enterprise.
Left bank:

0 m	mature spruce
0 - 17.5 m	small poplar
17.5 - 56.0 m	1m shrubs and grass
56.0 - 68.3 m	gravel , D_{50} =150 mm

Right bank:

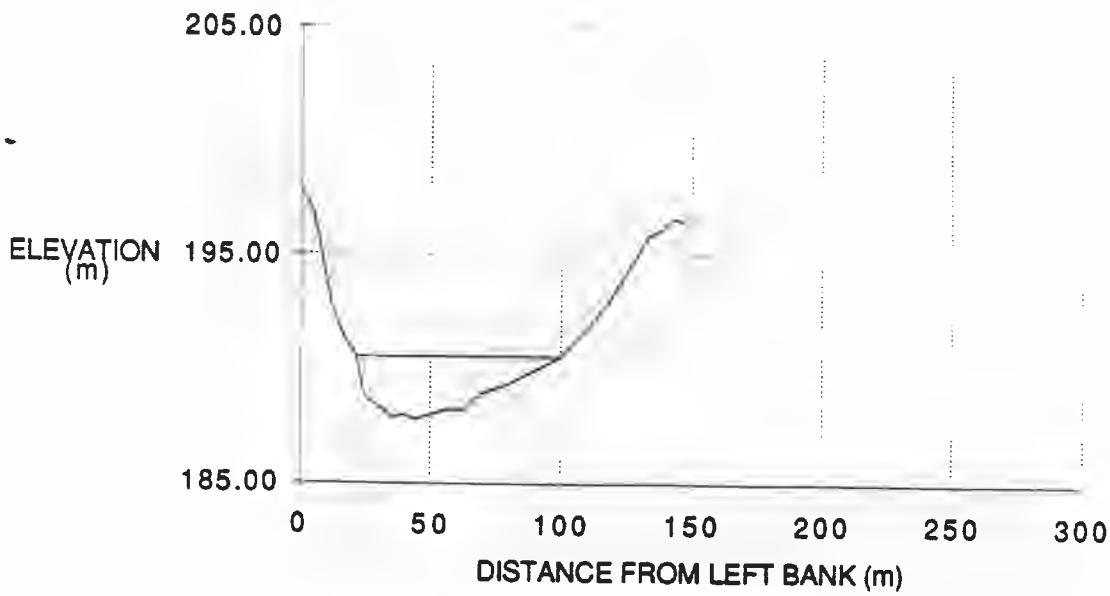
138.3 -143.0 m	large rock , D_{50} = 400 mm
143.0- 148.6 m	cut bank , till
148.6 m	high water mark Elev =198.07 m

Water level on July 24,1989: 190.740 m @ 16:37
Q = 225 CMS

TBM (L.B.): Elev: 197.669 m @ 0.0 m on cross section.
Spike in large spruce.

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B6 (continued). Hay River near Enterprise (c) Cross section km 1048.53



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	197.829
4.9	196.767
7.5	195.275
11.8	192.785
16.6	191.408
21.6	190.544
23.1	189.644
24.6	188.793
35.6	187.841
39.6	187.990
43.6	187.789
47.6	187.888
56.6	188.236
62.6	188.185
69.6	188.933
79.6	189.331
99.6	190.526
117.1	192.765
133.4	195.798
138.3	196.09[illegible]
142.8	[illegible]
147.8	196.44[illegible]

CROSS SECTION km 1048.955

Location: Enterprise.
Left bank:

0 - 4.9 m	mature spruce
4.9 - 21.6 m	cut bank , till , D_{50} = 300 mm

Right bank:

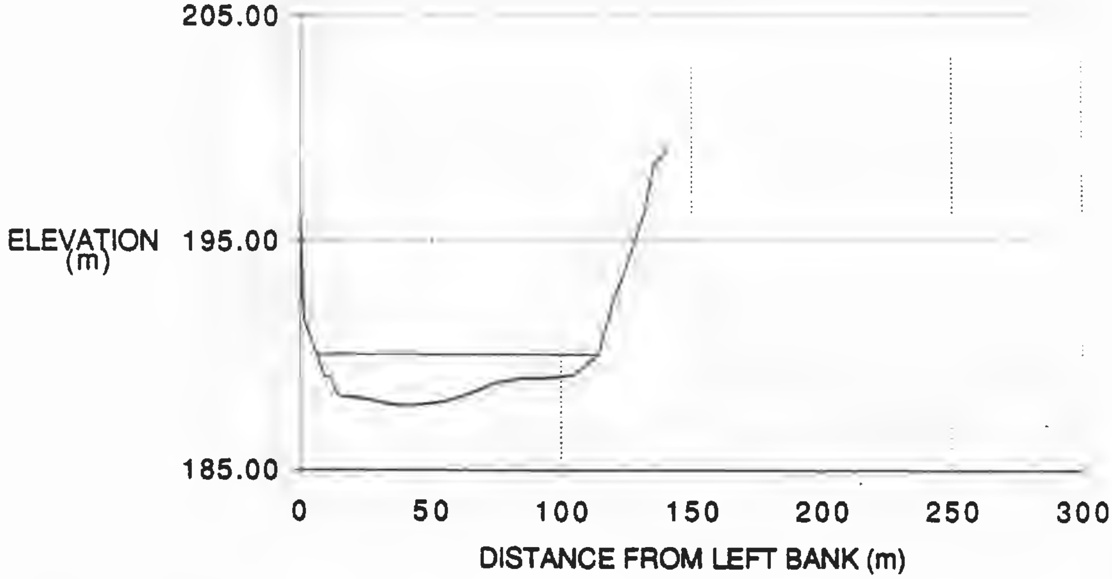
99.6 -117.1 m	gravel , D_{50} =150 mm
117.1- 133.4 m	1 m tall brush
133.4- 142.8 m	small poplar
142.8- 147.8 m	mature poplar and spruce

Water level on July 24,1989: 190.535 m @ 14:05
Q = 225 CMS

TBM (R.B.): Elev: 197.322 m @ 142.8 m on cross section.
Spike in large spruce.

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B6 (continued). Hay River near Enterprise (d) Cross section km 1048.96



DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	196.216
1.7	191.516
6.5	190.031
9.5	189.081
11.5	189.031
14.5	188.231
24.5	188.081
34.5	187.831
41.5	187.781
54.5	187.981
64.5	188.331
74.5	188.781
84.5	188.981
94.5	189.031
104.5	189.131
114.5	190.031
119.5	192.225
123.9	193.502
132.0	198.215
135.4	198.251
141.0	199.091

CROSS SECTION km 1049.576

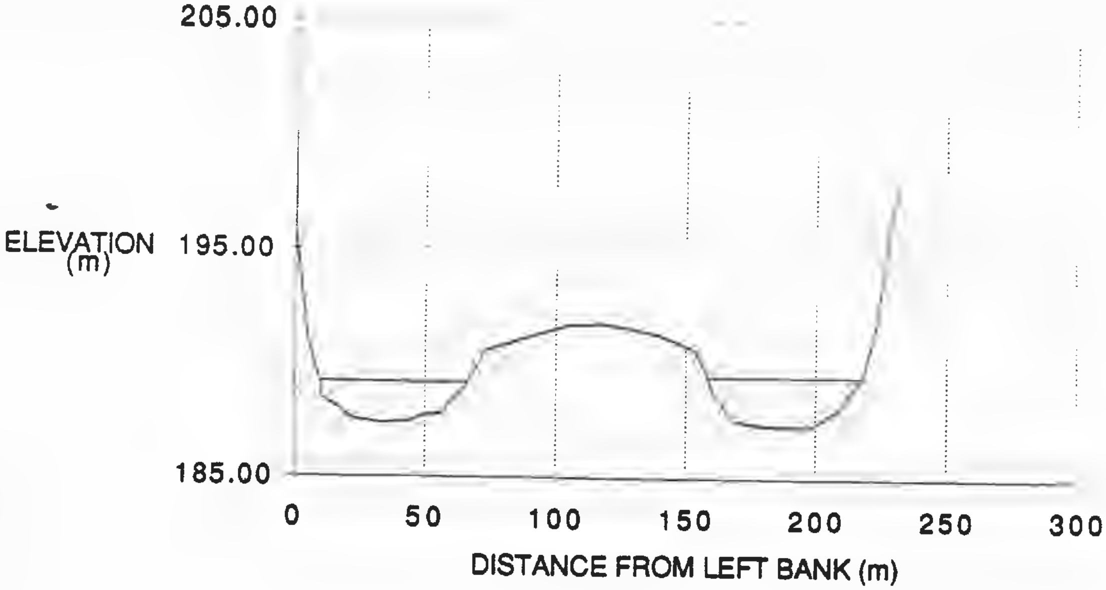
Location: 0.6 km D/S from Enterprise.
Left bank:
0 - 6.5 m cut bank , till , D_{50} = 200 mm

Right bank:
114.5 -132.0 m till , D_{50} =100 mm , 0.4 m tall brush
132.0- 141.0 m mature spruce and poplar

Water level on July 23,1989: 190.031 m @ 14:05
Q = 225 CMS
TBM (R.B.): Elev: 198.265 m @ 135.4 m on cross section.
Spike in 450 mm ϕ spruce.

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B6 (continued). Hay River near Enterprise (e) Cross section km 1049.58



CROSS SECTION km 1050.169

Location: 1.2 km D/S from Enterprise.
Left bank:

0 m	high water mark , Elev =195.82 m
0 - 9.7 m	cut bank , till , D_{50} = 200 mm

Island:

66.2- 158.7 m	gravel and silt, D_{50} =100 mm ,0.2 m tall brush

Right bank:

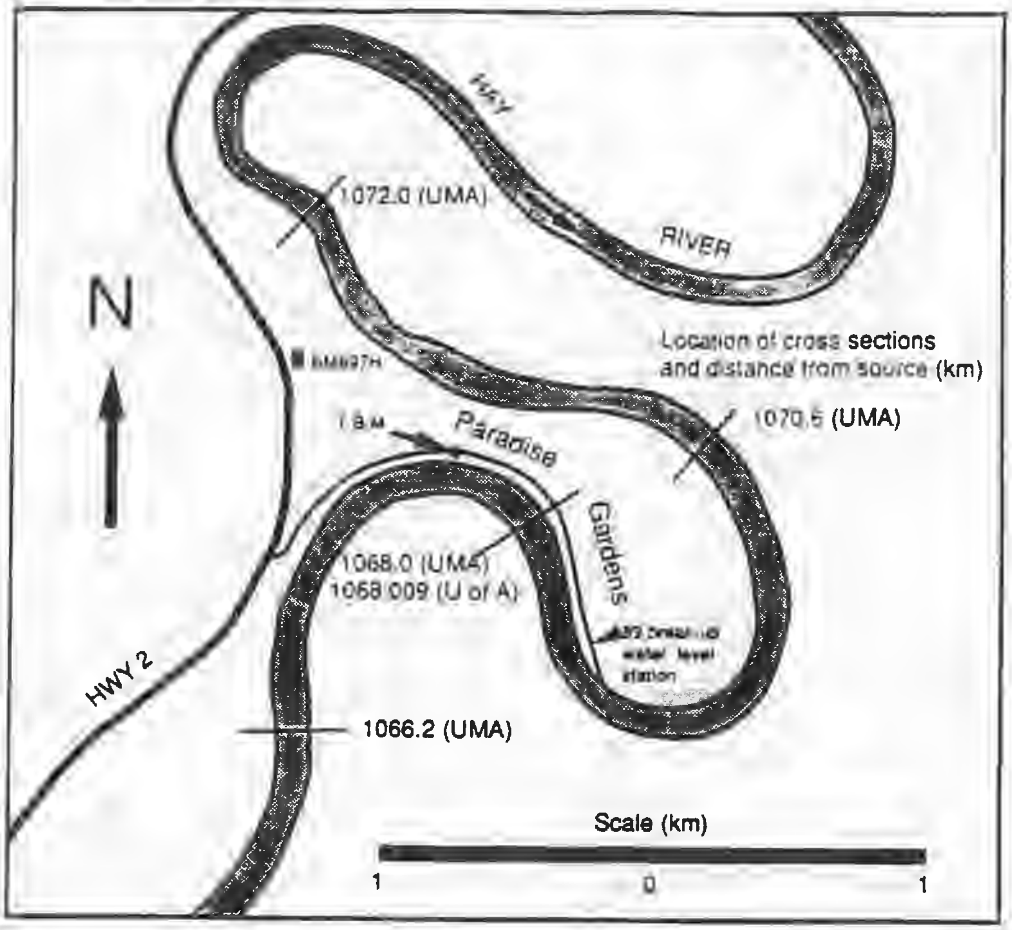
217.9- 229.3 m	till , D_{50} =100 mm , 1 m tall brush
229.3- 232.1 m	mature spruce

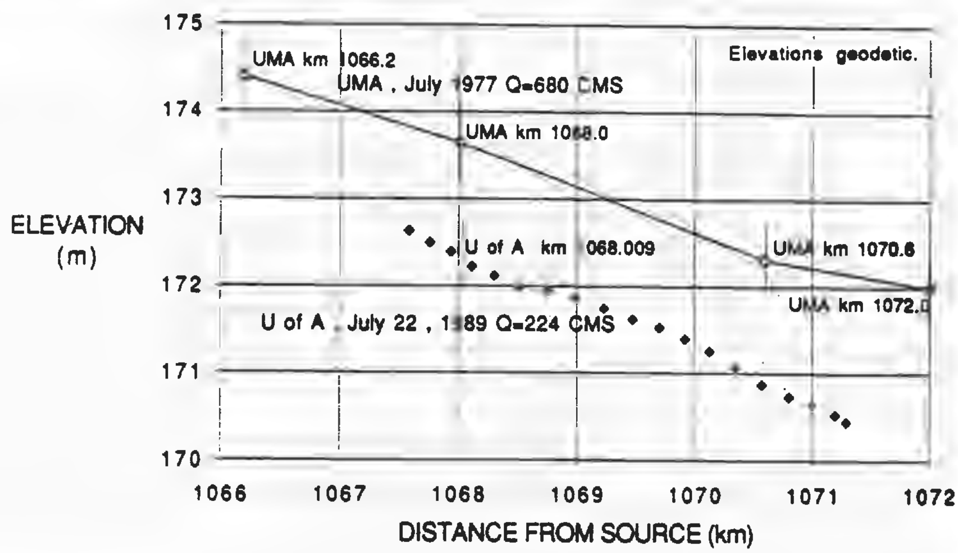
Water level (L.Channel) on July 23,1989: 189.167 m @ 13:25
Water level (R.Channel) on July 23,1989: 189.467 m @ 12:21
Q = 225 CMS
TBM (R.B.): Elev: 197.030 m @ 229.3 m on cross section.
Spike in 300 mm ϕ twin spruce.

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	195.819
5.0	190.919
9.7	189.159
10.2	188.459
16.2	188.061
22.2	187.563
33.2	187.366
36.2	187.417
44.2	187.469
49.2	187.720
56.2	187.772
66.2	189.175
73.3	190.582
103.8	191.637
118.8	191.727
138.8	191.299
153.8	190.602
158.7	189.467
160.2	188.817
162.9	188.517
167.9	187.567
177.9	187.367
187.9	187.317
197.9	187.317
207.9	188.317
217.9	189.467
222.9	191.727
226.4	194.830
229.3	197.030
232.1	197.957

Elevations tied in to 1:50 000 map contour; approximately geodetic.

Figure B6 (continued). Hay River near Enterprise (f) Cross section km 1050.17





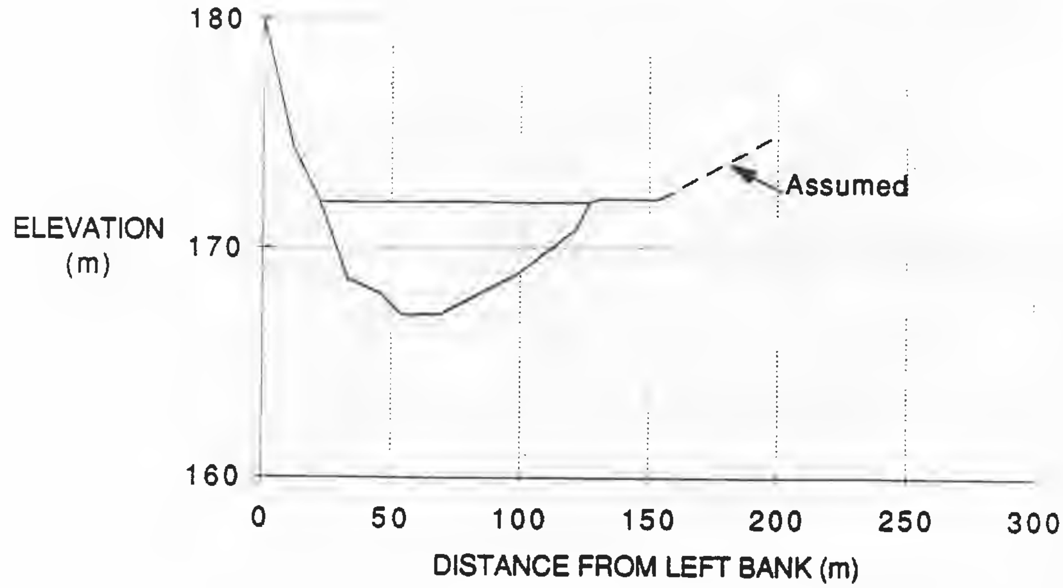
[illegible] SLOPE	
DISTANCE FROM [illegible] (km)	ELEVATION (m)
1066.2	174.40
1068.0	173.65
1070.6	172.31
1072.0	172.00

[illegible]	
DISTANCE FROM SOURCE (km)	ELEVATION (m)
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	172.118
[illegible]	171.981
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]
[illegible]	[illegible]

Figure B7. Hay River at Paradise Gardens (a) Plan (b) Longitudinal water surface profile surveyed in field and as determined from UMA (1977).

Figure B7 (continued). Hay River at Paradise Gardens (c) Airphoto of reach, 18 June 1979, scale 1:20000

Figure B7 (continued). Hay River at Paradise Gardens (d) Reach photos (i) Looking left to right bank and (ii) Left bank at section km 1068.01.



UMA CROSS SECTION km 1072.0

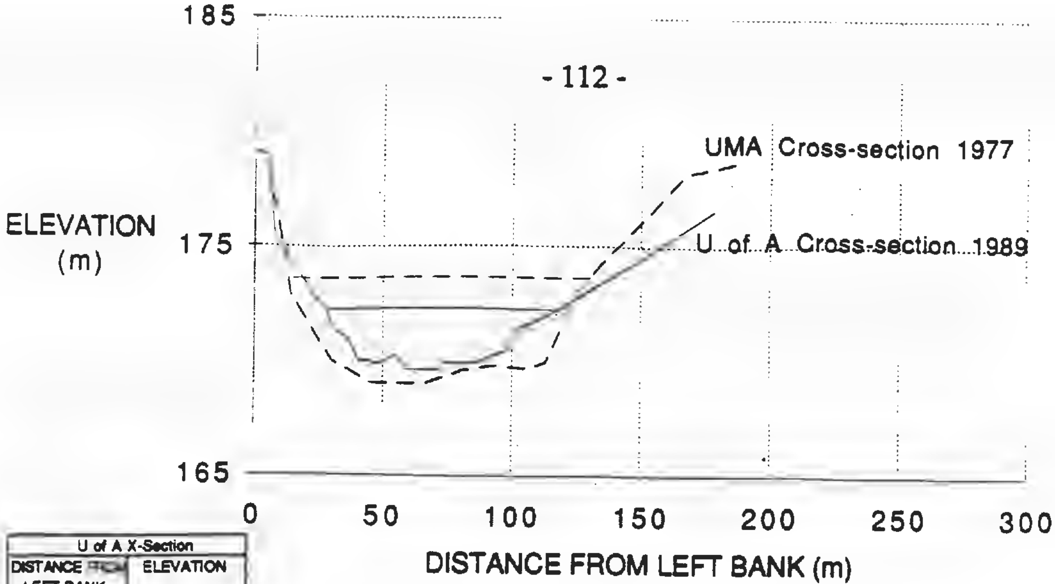
Location: 4 km D/S from Paradise Gardens.

Water level July ,1977: 172.00 m
Q = 680 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0	180.00
12	174.30
22	172.00
33	168.65
46	167.98
54	167.05
69	167.12
98	168.85
121	170.80
127	172.00
130	172.15
153	172.15
200	*175*

Elevations geodetic.

Figure B7 (continued). Hay River at Paradise Gardens (e) Cross section km 1072.0 taken from UMA (1977).



U of A X-Section	
DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0.0	179.099
5.0	179.024
7.7	175.4
12.6	174.68
17.5	173.779
27.4	172.311
30.4	171.411
35.7	171.061
40.7	169.981
43.7	170.061
47.7	169.811
54.7	170.261
58.7	169.611
71.7	169.611
73.7	170.011
77.7	169.911
86.7	170.011
97.7	170.561
101.7	171.311
102.7	171.461
117.7	172.311
148.5	174.349
154.0	174.829
178.4	176.601

UMA X-Section	
DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
5	178.72
13	173.65
14	172.95
30	170.00
45	169.03
54	168.98
66	169.01
81	169.62
93	169.33
106	169.70
118	170.01
123	173.01
130	173.65
134	174.25
157	178.12
186	178.65

CROSS SECTION km 1068.009

Location: 0.4 km D/S from TBM at Paradise Gardens.
Left bank:

	0 m	centerline of road
0 -	3.0 m	road
3.0 -	5.0 m	poplar and willow
5.0 -	7.7 m	silt cut bank
7.7 -	27.4 m	gravel , D_{50} = 150 mm

Right bank:

117.7 -140.0 m	large rock , D_{50} = 400 mm
140.0- 178.4 m	cut bank , till
178.4 m	large mature poplar

Water level on July 24,1989: 172.311 m @ 20:10
Q = 224 CMS
TBM (L.B.): Elev: 179.854 m 0.4 km U/S along road.
Spike in power pole.

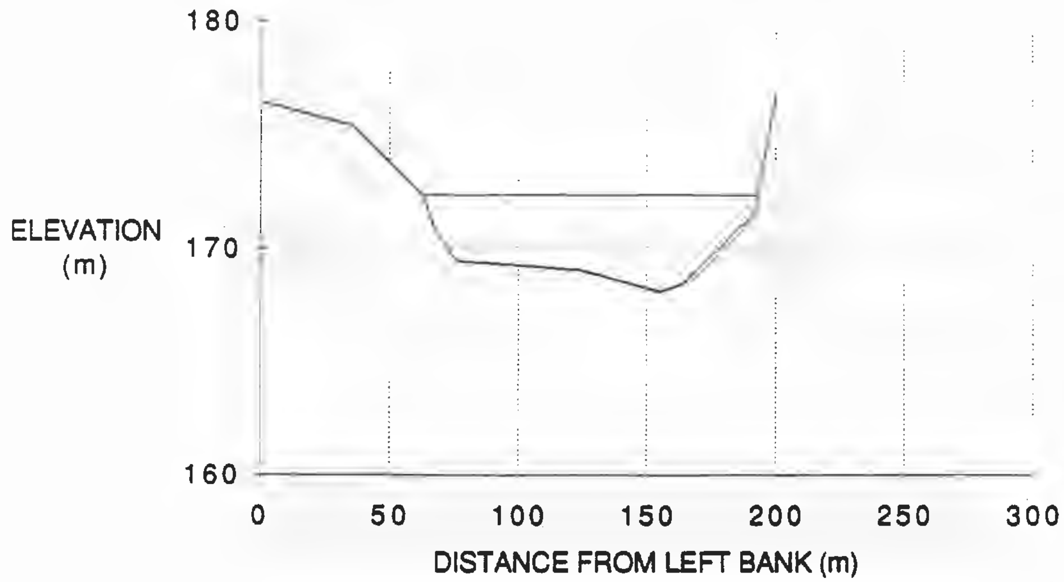
UMA CROSS SECTION km 1068.0

Location: Approximaitly same as cross-section km 1068.009
0.4 km D/S from TBM at Paradise Gardens.

Water level July ,1977: 173.65 m
Q = 680 CMS

Elevations geodetic.

Figure B7 (continued). Hay River at Paradise Gardens (f) Cross section km 1068.01 surveyed in 1989 and as taken from UMA (1977) (Note: UMA cross-section reversed from that shown in report).



UMA CROSS SECTION km 1070.6

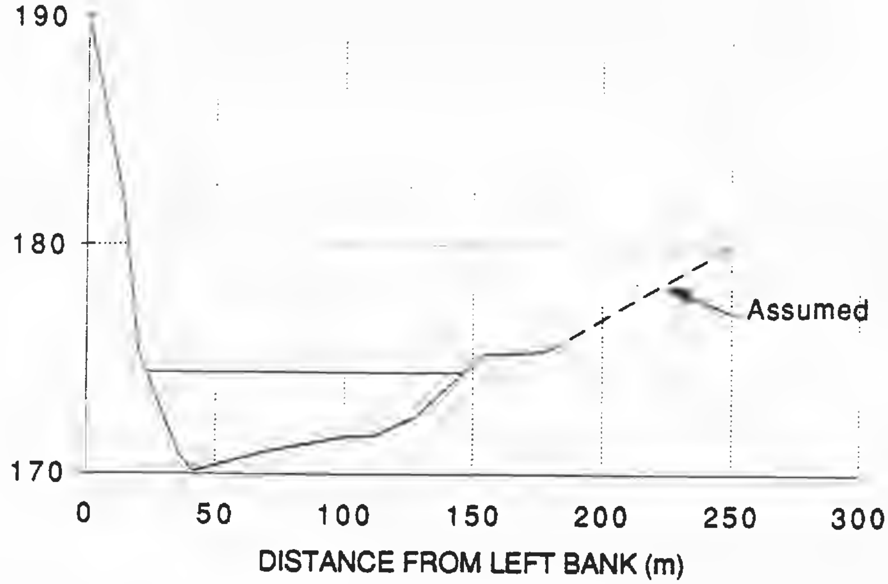
Location: 2.6 km D/S from Paradise Gardens.

Water level July ,1977: 172.31 m
Q = 680 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0	176.40
36	175.35
63	172.31
67	170.97
76	169.40
124	169.03
155	168.03
164	168.40
192	171.60
193	172.31
200	176.80

Elevations geodetic.

Figure B7 (continued). Hay River at Paradise Gardens (g) Cross section km 1070.6 taken from UMA (1977)



UMA CROSS SECTION km 1066.2

Location: 1.8 km U/S from Paradise Gardens.

Water level July ,1977: 174.40 m
Q = 680 CMS

DISTANCE FROM LEFT BANK (m)	ELEVATION (m)
0	190.00
14	182.00
20	175.38
23	174.40
36	170.80
41	170.10
71	171.00
100	171.60
113	171.70
128	172.50
146	174.40
153	175.20
176	175.35
250	180

Elevations geodetic.

Figure B7 (continued). Hay River at Paradise Gardens (h) Cross section km 1066.2 taken from UMA (1977).

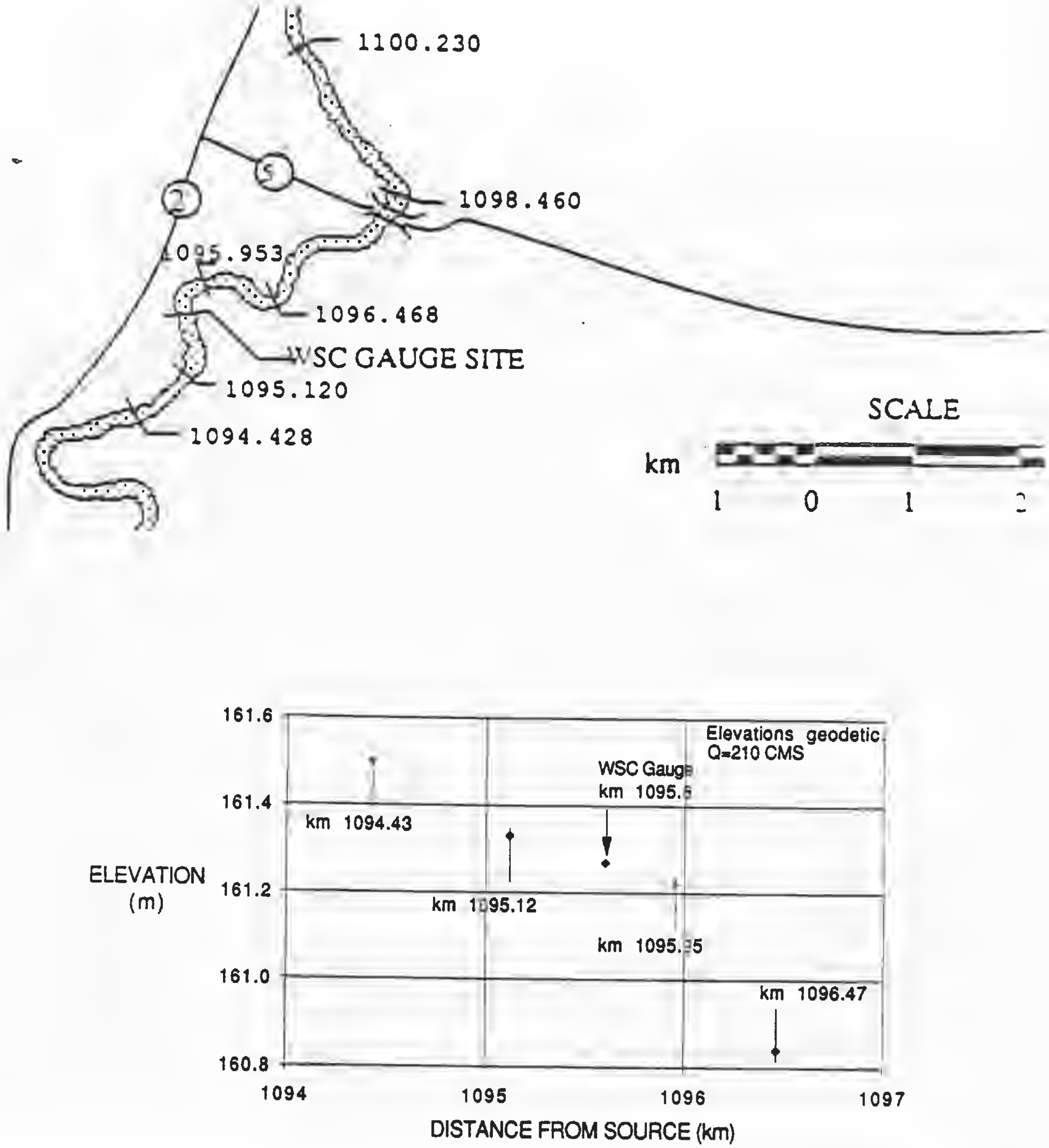


Figure B8 Hay River near Hay River WSC gauge (a) Plan and (b) Longitudinal profile of water surface on day of survey

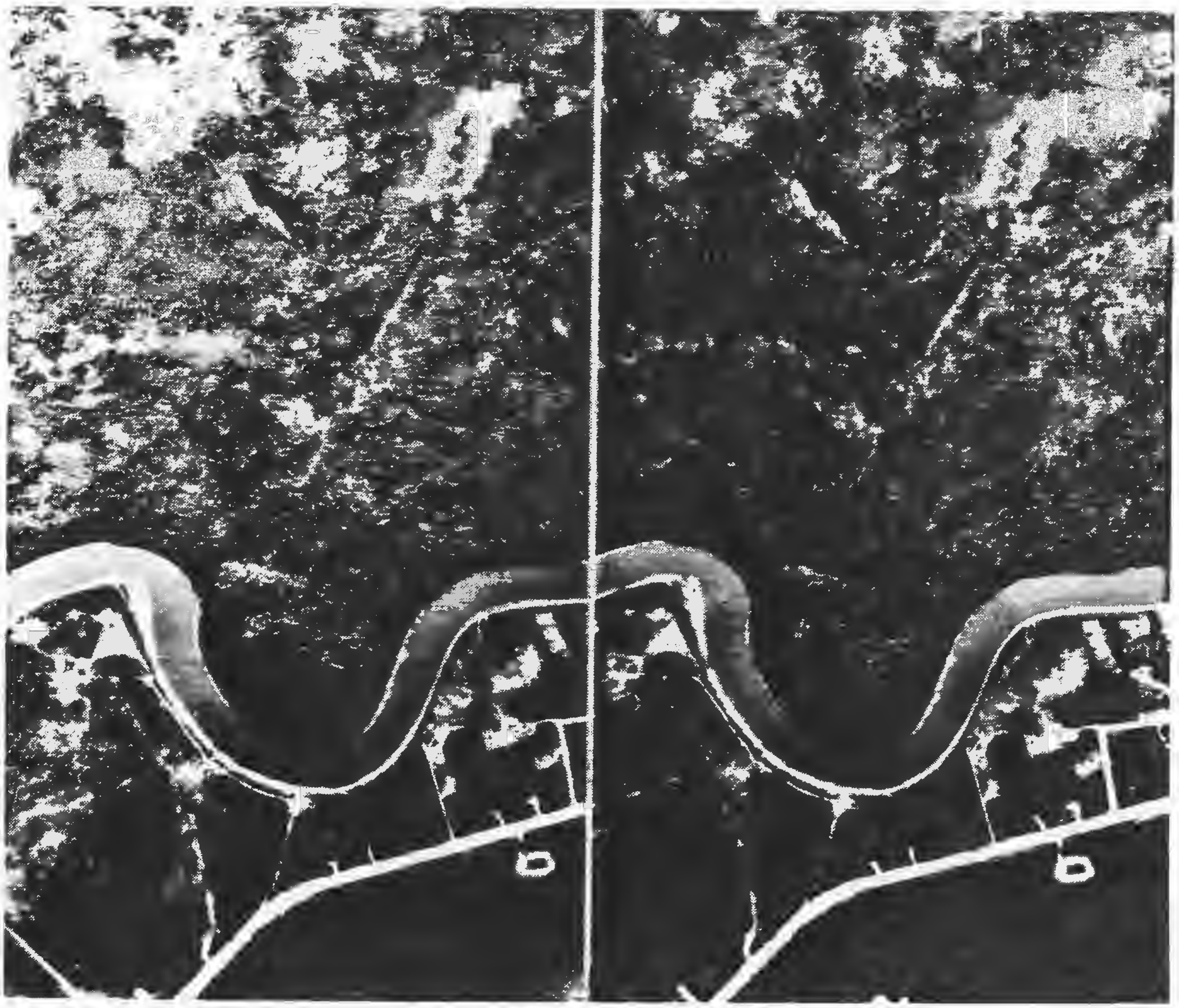


Figure B8 (continued) Hay River near Hay River WSC gauge (c) Airphotos of reach, 18 June 1979, scale 1:20000

Figure B8 (continued) Hay River near Hay River WSC gauge (d) Reach photos (i) Looking from right bank to left bank and (ii) Bed material at section km 1095.95.

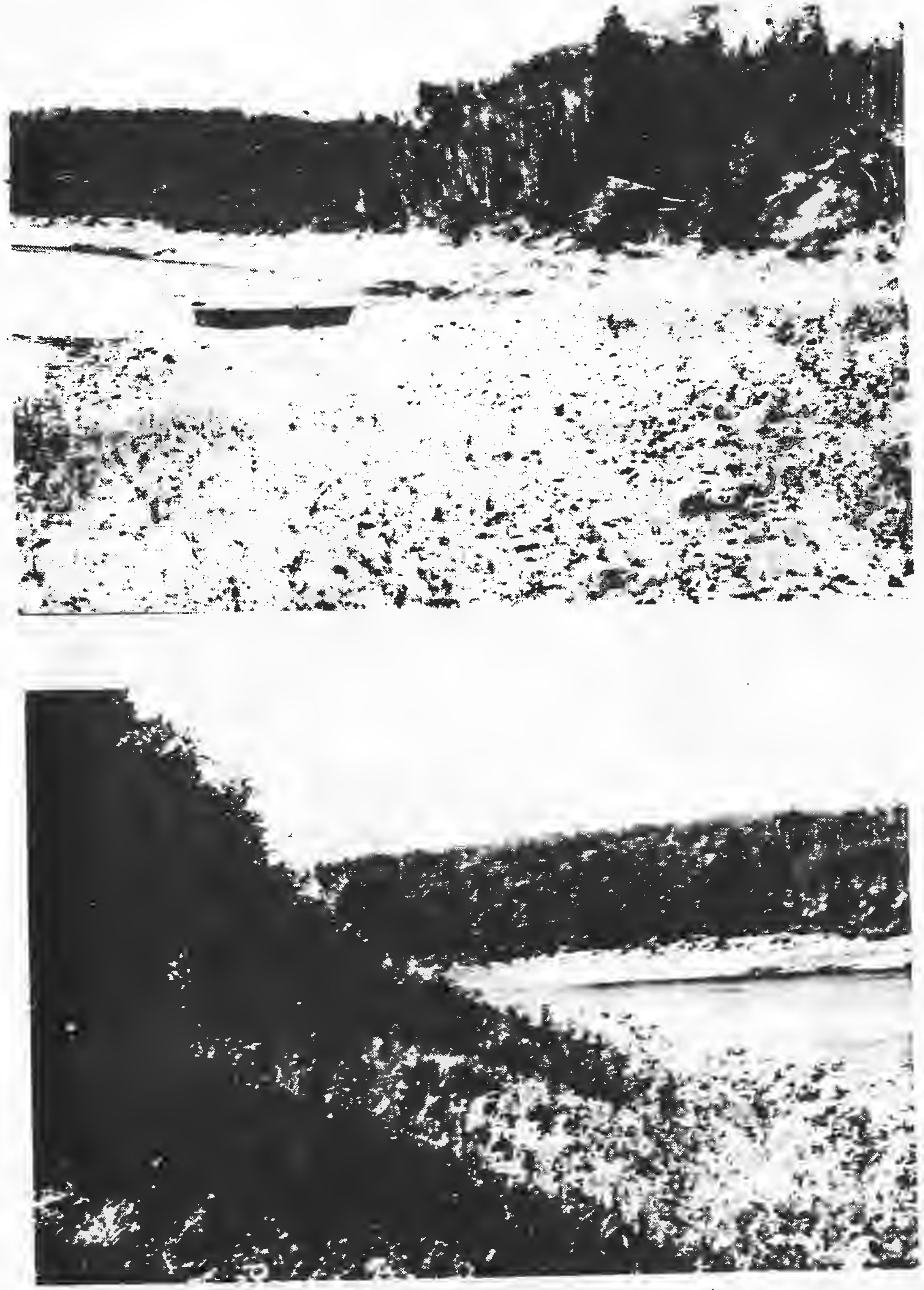
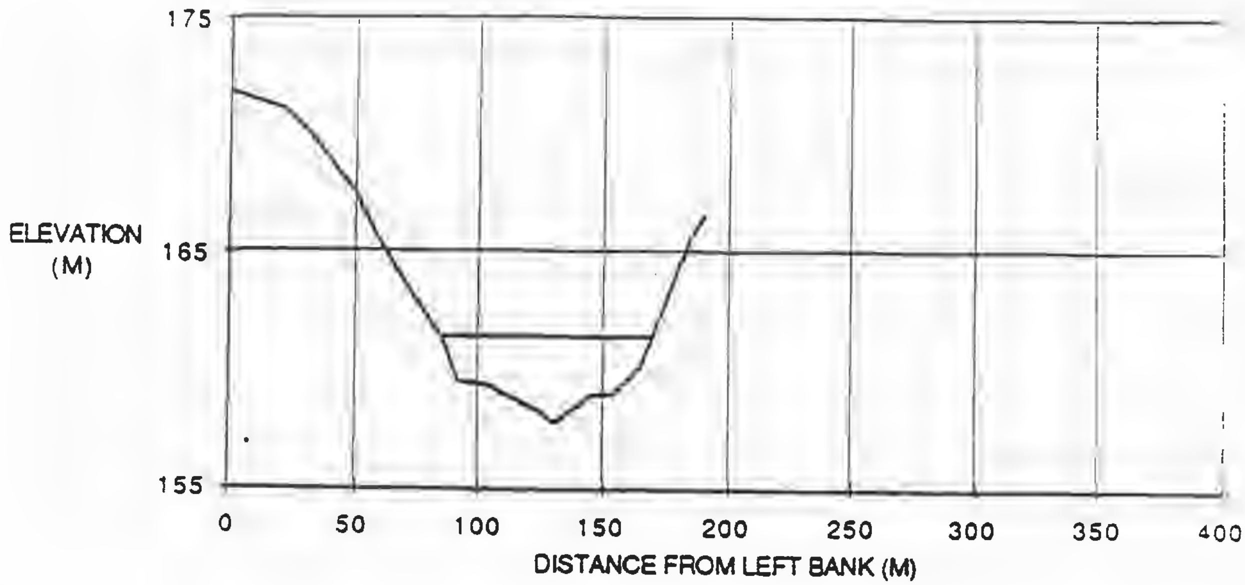


Figure B8 (continued) Hay River near Hay River WSC gauge (d) Reach photos (continued) (iii) Left bank and (iv) Right bank at section km 1095.95.



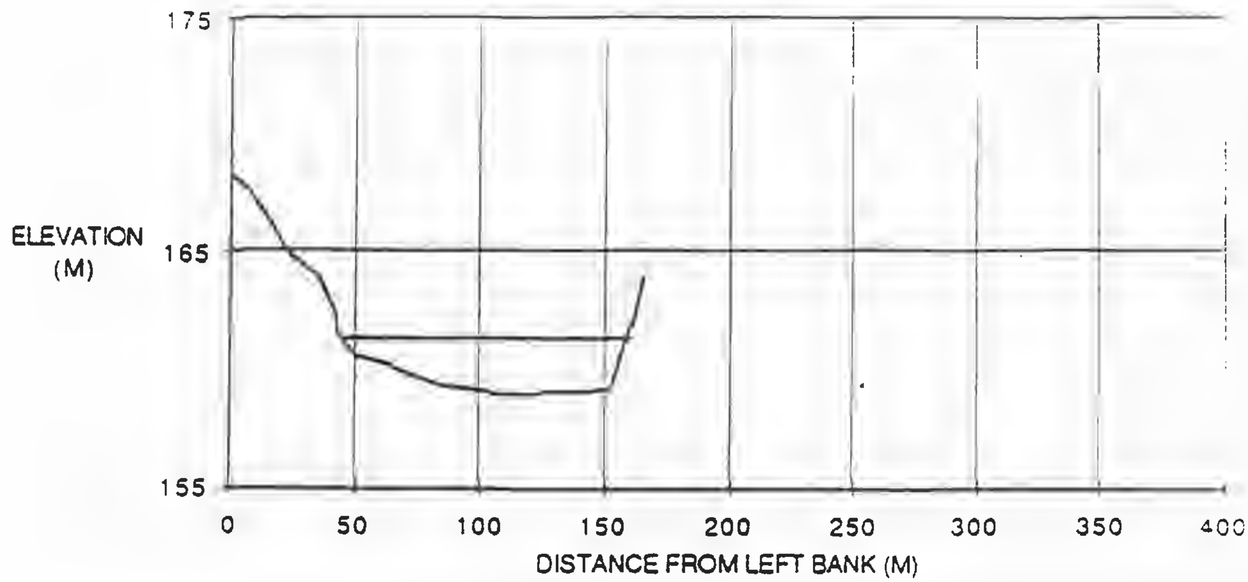
DISTANCE FROM LEFT BANK (M)	ELEVATION (M)
0	171.9
20	171 2
30.5	170.12
51	167 29
53	166.71
58.3	165.65
69	163.9
84 5	161.5
91	159.54
105	159 24
125	158 13
130	157 78
145	158.94
153	159.04
165	160.36
169 5	161 [illegible]
172 5	[illegible].39
179	[illegible]
182.6	[illegible]
185	[illegible]
190	[illegible].69

CROSS SECTION KM 1094.43

DESCRIPTION: This cross section is 1.1 km U/S of the Water Survey gauge site. The left bank is covered by poplar up to 51 m, after which the cover is willows to the water level. The right bank is grassed from the water level to 185 m, beyond which the trees start. The bed material had a D_{50} of 130 mm. Water level on the day of survey, July 18, 1987, was 161.50 m.

TBM: Spike in tree, top of left bank, 0.0 m on the cross section. Elevation 172.50 m.

Figure B8 (continued) Hay River near Hay River WSC gauge (e) Cross section km 1094.43.



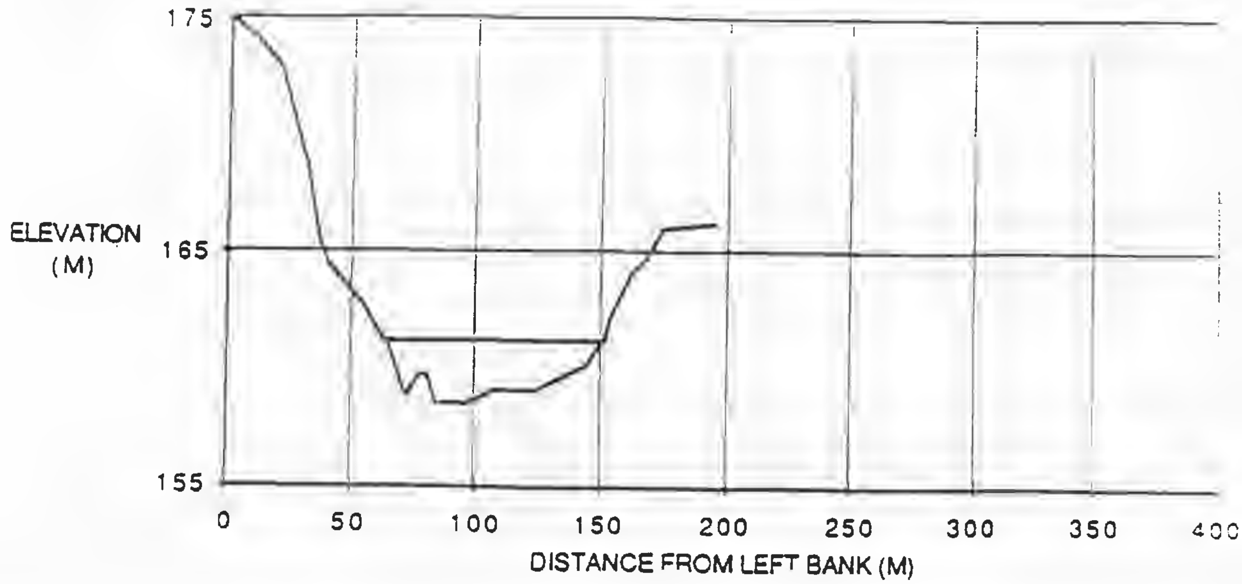
DISTANCE FROM LEFT BANK (M)	ELEVATION (M)
0	168.25
7	167.63
17	166.01
23	164.9[illegible]
35	163.8[illegible]
42	162.41
44	161.33
51	160.5[illegible]
64	160.11
84	159.2[illegible]
104	158.9[illegible]
124	159.0[illegible]
144	159.1
152	159.3
160	161.[illegible]
160.5	161.[illegible]
[illegible]	164.[illegible]

CROSS SECTION KM 1095.12

DESCRIPTION: This cross section is just U/S from the WSC gauge site. The left bank is treed to 23 m and from there willows extend to 37 m, with grass beyond to the water level. The right bank is grassed from the water level to 167 m, beyond which it is treed. The bed material has a D_{50} of 130 mm and the water level on the day of survey, July 18, 1987, was 161.33 m.

TBM: Spike in tree top of left bank, at 5 m on the cross section. Elevation 168.588 m.

Figure B8 (continued) Hay River near Hay River WSC gauge (f) Cross section km 1095.12.



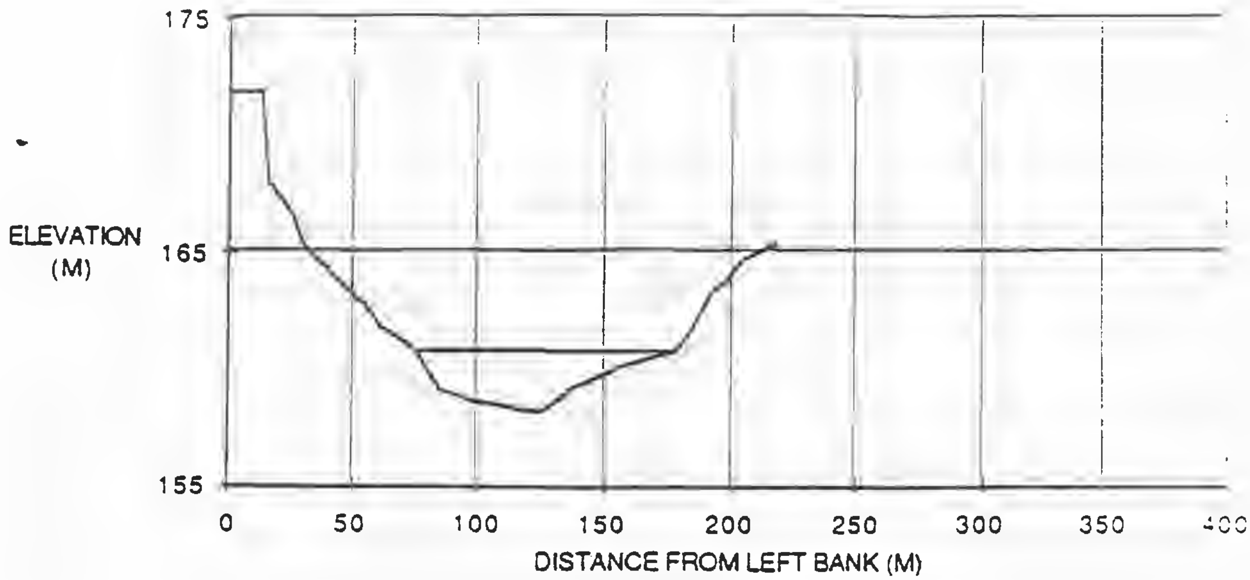
DISTANCE FROM LEFT BANK (M)	ELEVATION (M)
0	175
10	174.14
20	172.96
32	168 41
38	165.19
40 5	164 45
57	162.38
64 2	161 22
72	158.78
78	159 79
80	159.8
84	158.58
94	158 47
105	159 08
125	159.19
145	160.3
150.8	161 22
153	16[illegible].27
161	164 04
167	164 71
174	166 01
[illegible]	166 [illegible]

CROSS SECTION KM 1095.95

DESCRIPTION: This cross section is 450 m D/S of the Water Survey gauge. The left bank is treed to 33 m after which it is gravel to the water level. The right bank is grassed from the water level to 160 m, at which point the trees begin. The bed material has a D_{50} of 125 mm with some stones having a diameter up to 400 mm. Water level on the day of survey, July 18, 1987, was 161.22 m.

TBM: Spike in tree on the left bank at 35 m on cross section. Elevation - 168.564 m.

Figure B8 (continued) Hay River near Hay River WSC gauge (g) Cross section km 1095.95.



DISTANCE FROM LEFT BANK (M)	ELEVATION (M)
0	171.83
13	171.86
17	167 81
25.5	166.61
30	165.33
34	164 78
41	164 18
53	162.9
60	161.95
75	160.84
85	159 06
95	158.66
115	158.25
124	158 15
135	159.06
145	159 57
155	160 07
178	160 84
186	161 935
193	[illegible]
200	164.05
205	164.7
218	[illegible]

CROSS SECTION KM 1096.47

DESCRIPTION: This cross section is 1 km D/S of the Water Survey gauge. The left bank is covered with poplar up to 35 m. From 35 m to the water level the bank is gravel with no vegetation. The right bank consists of a gravel surface from the water to 190 m where willows start, with the trees starting at 200 m. The bed material had a D_{50} of 125 - 175 mm. The water level on the day of survey, July 18, 1987, 160.84.

TBM: Hub in ground on left bank at 60 m on cross section. Elevation - 162.006 m.

Figure B8 (continued) Hay River near Hay River WSC gauge (h) Cross section km 1096.47.

APPENDIX C

ICEJAM user's manual

The *FALCONeditor™* combines the power of scientific programming with the user friendliness of the personal computer by providing a menu-driven interface between the user and scientific/engineering programs.

Hardware Requirements

The *FALCONeditor™* for *ICEJAM* will run on any IBM® PC/XT®, PC/AT® or compatible computers with 256K of memory. A single floppy disk drive is also required, though we recommend you run the program from a hard disk if you have one. The *FALCONeditor™* is compatible with most monitor types such as CGA, EGA and Monochrome. It will also support both dot matrix and laser printers, though the program does assume that the printer is attached to LPT1. If this is not the case, please contact us for a modified version of the program.

Installing the ICEJAM package

We strongly suggest that you make a backup copy of the ICEJAM disk before using the program. Refer to your DOS manual if you don't already know how to do this.

If you decide to run the ICEJAM program from your hard disk, be sure to copy the entire contents of the "Product Disk" to your hard disk.

Running the ICEJAM program

After you have successfully installed the ICEJAM package you can run the test data to get familiar with the program and the editor. To run the *FALCONeditor™*, simply type ICEJAM and press the ENTER key.

Note that the drive containing the ICEJAM package must be the current, or "default", drive. For example, if you are running the program from the floppy drive "A" then this must be the default drive.

The first screen, Figure C1, contains information about the program authors. Refer to this screen if you need to contact us about a problem or if you would like to consider updating or expanding your program. Press any key to continue after viewing this screen.

You should now be looking at the main menu, Figure C2, where the FKeys offer the following choices:

F1 First Level Flood Watch

F2 Second Level Flood Watch

F3 Set Date

F4 Set Screen Colors

F5 Quit

Notice that the functions performed by each of the FKeys are shown in an abbreviated form at the bottom of the screen as well.

- The **F1** key allows you to conduct First Level Flood Watch calculations.

- The **F2** key allows you to conduct Second Level Flood Watch calculations. There is a sub-option depending upon whether information is available about an ice jam that may release a surge or not.

- The **F3** key allows you to change the date when the one shown below the window is incorrect. In that situation, you would press **F3** and then enter the correct date.

- The **F4** key allows you to set the screen colors to suit your own preferences. Try experimenting if you have a color monitor.

- The **F5** key causes you to quit the program and return to DOS. This key performs the same function from every menu in the *FALCONeditor™*, allowing you to exit the program at any time.

First Level Flood Watch

Press the **F1** key. You should see a screen entitled "First Level Flood Watch". (If you don't, return to the Main Menu by pressing **F4** and try again.) This is a *data entry* screen for the ICEJAM program. A data entry screen has "fields" for entering the data required by the ICEJAM program; you access these fields by using the cursor (arrow) keys. You can tell when you are in a particular field when it is displayed in reverse video. To enter or change the value in a field, type in the appropriate numbers and then press the **ENTER** key or the "down" cursor key.

NOTE: you must press the ***ENTER*** *key or a cursor key after typing the numbers or your new value will not be accepted.*

Try a test calculation to see how it works. Enter the following three values for snowfall in cm (centimetres):

1) Hay River	144
2) Fort Nelson	128
3) High Level	109

If you have entered the values correctly, the calculated numbers should be the same as those shown in Figure C3.

You may notice that some text appears in the space below the data entry window from time to time. This is extra information, to assist you as you use the ICEJAM program. For example, when the maximum likely discharge is less than 200 cms (cubic metres per second), no values can be calculated. To see an example of this, type in new values (you will have to use the cursor keys to move around):

1) Hay River	52
2) Fort Nelson	81
3) High Level	44

You should also notice the FKeys at the bottom of the screen. Even though this is a data entry screen, the FKeys are still ready to perform useful functions. For example, if you would like to have a printout of your results you can press the F3 key (**F3 Print**). This key should only be used if you have a printer attached to your computer.

*Note: on this and any other data entry screen you may quit the program at any time by pressing the **F5** key or return to the Main Menu by pressing **F4**.*

Press **F4** to return to the main menu.

Second Level Flood Watch

The Second Level Flood Watch calculation offers two alternatives. As illustrated in Figure C2, when you press **F2** to select the Second Level Flood Watch you will be asked at the bottom of the screen if you know the location and length of an ice jam on the Hay River. If you do not know this information just type the letter "n" (without the quotes) and then press the <u>ENTER</u> key. Try doing this.

You should now see the screen entitled *Second Level Flood Watch / 1 Day Forecast.* , Figure C4. This is used whenever you are WITHOUT ice jam details. You use this screen in exactly the same way as you did the First Level Flood Watch screen. Try entering these values:

Current discharge at the Border	=	680 cms
Bright sunshine with mean daily temperature above -5 deg. C	=	200 hours
Accumulated snow at Hay River	=	144 cm

The results should agree with the numbers shown in Figure C4. Note that the determination of the entry *"Bright sunshine with mean daily temperature above -5 deg. C"* is based on data from Ft. Smith. It should not be initiated for isolated occurrences of temperatures above -5 °C (wait for, say, 5 consecutive days of temperatures above -5 °C).

Next, press the **F2** key to see results for other sites in the delta. This new screen, Figure C5, entitled "Discharge/Water Level Estimates at Other Sites", shows results calculated at the Fishing Village, East Channel Docks and at Fill C, as seen in Figure C5. As described in the body of the report, the values predicted here are based on a number of assumptions. For example, the calculations include the effects of variability from year to year. In addition, the levels calculated at the East Channel Docks assume that the jam in the East Channel is located at the mouth. For each of the four sites, a convenient reference level is provided, and the predicted water level is related to this reference level (in feet). Therefore, a negative number indicates that the anticipated water level is below the reference feature.

Again, if you would like to have a printout of your results you can press the F3 key (**F3 Print**). If you would like to see the previous screen (that shown in Figure C4), press the **F2** key to go back (**F2 Last Page**).

Note: you may select the F3 ***Print*** *key from either of these two screens used in the Second Level Flood Watch - all results will be sent to the printer for all sites.*

Now press **F4** to return to the main menu. This will enable you to try the other Second Level Flood Watch alternative. This time, when you press **F2** to select the Second Level Flood Watch, type the letter "y" and then press the <u>ENTER</u> key. You should now see the screen entitled *Second Level Flood Watch / WITH Ice Jam Details.* This screen is illustrated in Figure C6. Here you are asked to enter the estimated discharge at the town of Hay River, the distance between the town and the ice jam, and the length of the jam.

To test this portion of the program enter:

Estimated discharge at Hay River	=	680 cms
Distance of jam upstream of Hay River	=	230 km
Length of ice jam	=	25 km

The results should agree with the numbers shown in Figure C6.

Based on this input and the unsteady flow analysis conducted for the Hay River, two surge discharges (at the town) are predicted. These values encompass the range expected for the two extreme cases of open water downstream of the ice jam and an ice covered channel. An estimate of the time of arrival of this peak discharge at the town, after the ice jam release, is also provided. Based on these surge discharges, water levels at the West Channel Bridge are predicted. If you would like information at other sites, press **F2**.

Note: this is the same screen used in the other Second Level Flood Watch alternative - however, results are based on the highest surge discharge predicted from the unsteady flow analysis. All the same assumptions apply.

Do you have a question? Contact:

Dr. Faye Hicks (403) 492-7170

or

Dr. Robert Gerard (403) 492-2066
Department of Civil Engineering
University of Alberta
Edmonton, Alberta, T6G 2G7

FAX (403) 492-0249

Remember, HYprocessing Ltd. specializes in converting your problem (or program) into user friendly computer software! For more information, contact:

Mr. Doug Yeomans (403) 463-6974
HYprocessing Ltd.
6808-10 Avenue, Edmonton, Alberta, T6K 2T7

FALCONeditor™

Flood Watch Program for the Town of Hay River

ICEJAM

prepared for
Environment Canada and Indian and Northern Affairs Canada

by
R. Gerard and S. Stanley
Civil Engineering Department, University of Alberta
Edmonton, Alberta, T6G 2G7

Interactive Programming by
HYprocessing Ltd.
6808-10 Avenue, Edmonton, Alberta, T6K 2T7

Figure C1 Program Information Screen

FIRST LEVEL FLOOD WATCH

LOCATION	SNOW DEPTH (cm)
1) Hay River	144
2) Fort Nelson	128
3) High Level	109

- Basin Average Accumulated Snowfall = 118 cm
- Maximum likely discharge at Hay River (based on accumulated snow only) = 397 to 960 cms
- Maximum likely water level upstream of the West Channel Bridge at breakup (assuming no ice deterioration)
  = 161.1 to 163.4 m (geodetic)
  = 11.6 to 19.1 ft (on the pier of W.C.B.)

F1	F2 Level 2	F3 Print	F4 Main Menu	F5 Quit

Figure C2. First Level Flood Watch Calculations

MAIN MENU

F1 First Level Flood Watch
F2 Second Level Flood Watch
F3 Set Screen Colors
F4 Set Date
F5 Quit

Do you know the location and length of an ice jam ?
(enter n or y and then press RETURN)

F1 Level 1	F2 Level 2	F3 Set Color	F4 Set Date	F5 Quit

Figure C3. Main menu with "F2 Second level flood watch" selected.

SECOND LEVEL FLOOD WATCH / 1-DAY FORECAST

WITHOUT ICE JAM DETAILS

Current discharge at the Border	=	680	cms
Bright sunshine with mean daily temperature above -5°C	=	200	hours
Accumulated snow at Hay River	=	144	cm

West Channel	Datum	Possible Range of Levels		
Open Water	Geodetic (m)	**160.7**	to	**161.4**
	On pier of W.C.B. (ft)	**10.3**	to	**12.8**
Jam at W.C.B.	Geodetic (m)	**161.4**	to	**162.3**
	On pier of W.C.B. (ft)	**12.6**	to	**15.5**

F1 Level 1	F2 Oth. Sites	F3 Print	F4 Main Menu	F5 Quit

Figure C4. Second level flood watch calculations without ice jam information.

DISCHARGE / WATER LEVEL ESTIMATES AT OTHER SITES

WEST CHANNEL CONDITIONS	ESTIMATE	LOCATION W.C.B.	F.V.	Fill C	E.C.*
Open Water	Discharge (cms)	369	369	310	310
	Water Level (m)	161.4	160.2	160.7	158.7
	Above Reference (ft)	12.8	5.1	-7.4	1.4
Ice Jam	Discharge (cms)	240	240	438	438
	Water Level (m)	162.3	159.5	161.6	159.4
	Above Reference (ft)	15.5	3.0	-4.7	3.4

NOTE - All levels geodetic and are upper value of range.
- Calculations assume ice jam in East Channel.

*** assumes jam at East Channel mouth**

W.C.B. - West Channel Bridge Pier (157.55m)
F.V. - Fishing Village Docks (158.6m)
Fill C - Strang's corner south edge of pavement (spike in pole 31 - 163.0m)
E.C. - East Channel Docks (158.3m)

F1 Level 1	F2 Last Page	F3 Print	F4 Main Menu	F5 Quit

Figure C5. Discharge/Water Level Estimates at Other Sites

SECOND LEVEL FLOOD WATCH

WITH ICE JAM DETAILS

Estimated discharge at Hay River = 680 cms
Distance of ice jam upstream of Hay River = 230 km
Length of ice jam = 25 km

Estimated surge discharge at Hay River = **801.2** to **1050.6** cms
(occurring approximately **26.7** hours after ice jam release)

West Channel	Datum	No Surge	Ice Cover	Open Water
Open Water	Geodetic (m)	**160.7**	**161.0**	**161.6**
	Pier of WCB (ft)	**10.3**	**11.4**	**13.2**
Jam at WCB	Geodetic (m)	**161.4**	**161.8**	**162.5**
	Pier of WCB (ft)	**12.6**	**13.9**	**16.2**

F1 Level 1	F2 Oth. Sites	F3 Print	F4 Main Menu	F5 Quit

Figure C6. Second level flood watch calculations given the location and length of an ice jam.

Table C1

Distance from Source (km)	Bed Elevation (m)	Top Width (m)
615.00	350.52	107.95
616.00	349.93	108.75
617.00	349.35	109.42
618.00	348.76	109.94
619.00	348.18	110.26
620.00	347.59	110.36
621.00	347.01	110.25
622.00	346.43	109.94
623.00	345.86	109.43
624.00	345.30	108.78
625.00	344.77	108.02
626.00	344.26	107.21
627.00	343.77	106.39
628.00	343.30	105.60
629.00	342.86	104.87
630.00	342.43	104.24
631.00	342.03	103.74
632.00	341.63	103.39
633.00	341.25	103.23
634.00	340.88	103.29
635.00	340.52	103.62
636.00	340.16	104.22
637.00	339.80	105.11
638.00	339.45	106.28
639.00	339.10	107.69
640.00	338.75	109.29
641.00	338.40	111.02
642.00	338.05	112.77
643.00	337.71	114.47
644.00	337.37	116.00
645.00	337.04	117.27
646.00	336.72	118.19
647.00	336.42	118.72
648.00	336.14	118.83
649.00	335.88	118.54
650.00	335.64	117.90
651.00	335.43	116.98
652.00	335.23	115.86
653.00	335.04	114.62
654.00	334.87	113.33
655.00	334.71	112.01
656.00	334.56	110.71
657.00	334.41	109.42
658.00	334.27	108.13
659.00	334.13	106.84
660.00	334.00	105.55
661.00	333.87	104.24
662.00	333.74	102.92
663.00	333.60	101.59

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
664.00	333.47	100.23
665.00	333.34	98.85
666.00	333.21	97.44
667.00	333.08	96.03
668.00	332.96	94.63
669.00	332.83	93.25
670.00	332.70	91.92
671.00	332.57	90.65
672.00	332.44	89.45
673.00	332.31	88.31
674.00	332.18	87.22
675.00	332.05	86.17
676.00	331.92	85.15
677.00	331.79	84.15
678.00	331.66	83.18
679.00	331.53	82.23
680.00	331.41	81.33
681.00	331.28	80.49
682.00	331.15	79.72
683.00	331.02	79.05
684.00	330.89	78.47
685.00	330.76	77.99
686.00	330.63	77.62
687.00	330.50	77.36
688.00	330.37	77.21
689.00	330.24	77.15
690.00	330.11	77.20
691.00	329.98	77.34
692.00	329.86	77.58
693.00	329.73	77.92
694.00	329.60	78.35
695.00	329.47	78.88
696.00	329.34	79.53
697.00	329.21	80.31
698.00	329.08	81.25
699.00	328.95	82.40
700.00	328.82	83.80
701.00	328.69	85.47
702.00	328.56	87.43
703.00	328.43	89.65
704.00	328.28	92.09
705.00	328.13	94.68
706.00	327.97	97.34
707.00	327.78	99.96
708.00	327.58	102.45
709.00	327.36	104.75
710.00	327.13	106.79
711.00	326.88	108.55
712.00	326.62	110.03

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
713.00	326.35	111.25
714.00	326.06	112.26
715.00	325.78	113.09
716.00	325.48	113.79
717.00	325.19	114.42
718.00	324.89	114.99
719.00	324.59	115.54
720.00	324.28	116.09
721.00	323.98	116.66
722.00	323.67	117.29
723.00	323.37	118.00
724.00	323.06	118.81
725.00	322.75	119.74
726.00	322.45	120.79
727.00	322.14	121.93
728.00	321.84	123.14
729.00	321.53	124.39
730.00	321.22	125.62
731.00	320.92	126.81
732.00	320.61	127.93
733.00	320.30	128.97
734.00	320.00	129.93
735.00	319.69	130.83
736.00	319.38	131.67
737.00	319.08	132.44
738.00	318.77	133.15
739.00	318.46	133.77
740.00	318.16	134.28
741.00	317.85	134.68
742.00	317.54	134.97
743.00	317.24	135.16
744.00	316.93	135.28
745.00	316.62	135.35
746.00	316.32	135.41
747.00	316.01	135.46
748.00	315.70	135.50
749.00	315.40	135.52
750.00	315.09	135.50
751.00	314.78	135.43
752.00	314.48	135.30
753.00	314.17	135.10
754.00	313.86	134.83
755.00	313.56	134.51
756.00	313.25	134.16
757.00	312.95	133.79
758.00	312.64	133.40
759.00	312.33	132.98
760.00	312.03	132.51
761.00	311.72	131.98

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
762.00	311.41	131.39
763.00	311.11	130.73
764.00	310.80	130.03
765.00	310.49	129.33
766.00	310.19	128.64
767.00	309.88	128.01
768.00	309.57	127.42
769.00	309.27	126.87
770.00	308.96	126.32
771.00	308.65	125.72
772.00	308.35	125.04
773.00	308.04	124.22
774.00	307.73	123.26
775.00	307.43	122.17
776.00	307.12	120.95
777.00	306.81	119.66
778.00	306.51	118.33
779.00	306.20	117.01
780.00	305.89	115.71
781.00	305.59	114.49
782.00	305.28	113.36
783.00	304.97	112.36
784.00	304.67	111.51
785.00	304.36	110.86
786.00	304.06	110.44
787.00	303.75	110.32
788.00	303.44	110.55
789.00	303.14	111.19
790.00	302.83	112.30
791.00	302.52	113.90
792.00	302.22	115.99
793.00	301.91	118.53
794.00	301.60	121.47
795.00	301.30	124.70
796.00	300.99	128.12
797.00	300.68	131.61
798.00	300.38	135.05
799.00	300.07	138.33
800.00	299.76	141.32
801.00	299.46	143.93
802.00	299.15	146.04
803.00	298.84	147.60
804.00	298.54	148.55
805.00	298.23	148.90
806.00	297.92	148.68
807.00	297.62	147.99
808.00	297.31	146.93
809.00	297.01	145.66
810.00	296.72	144.30

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
811.00	296.45	143.00
812.00	296.19	141.88
813.00	295.95	141.04
814.00	295.73	140.54
815.00	295.53	140.40
816.00	295.36	140.63
817.00	295.20	141.20
818.00	295.06	142.03
819.00	294.93	143.06
820.00	294.82	144.19
821.00	294.71	145.34
822.00	294.61	146.42
823.00	294.51	147.33
824.00	294.42	147.98
825.00	294.33	148.28
826.00	294.24	148.18
827.00	294.15	147.64
828.00	294.06	146.65
829.00	293.98	145.27
830.00	293.89	143.55
831.00	293.80	141.60
832.00	293.72	139.52
833.00	293.63	137.43
834.00	293.55	135.41
835.00	293.46	133.54
836.00	293.38	131.86
837.00	293.29	130.40
838.00	293.21	129.15
839.00	293.12	128.09
840.00	293.04	127.18
841.00	292.95	126.39
842.00	292.87	125.65
843.00	292.78	124.93
844.00	292.70	124.18
845.00	292.61	123.36
846.00	292.53	122.45
847.00	292.44	121.42
848.00	292.36	120.27
849.00	292.27	119.01
850.00	292.19	117.64
851.00	292.10	116.20
852.00	292.02	114.71
853.00	291.93	113.23
854.00	291.85	111.81
855.00	291.76	110.50
856.00	291.68	109.36
857.00	291.59	108.43
858.00	291.51	107.76
859.00	291.42	107.37

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
860.00	291.33	107.27
861.00	291.25	107.45
862.00	291.16	107.89
863.00	291.08	108.56
864.00	290.99	109.44
865.00	290.91	110.52
866.00	290.82	111.78
867.00	290.74	113.22
868.00	290.65	114.84
869.00	290.57	116.65
870.00	290.48	118.65
871.00	290.40	120.84
872.00	290.31	123.21
873.00	290.23	125.72
874.00	290.14	128.32
875.00	290.06	130.93
876.00	289.97	133.49
877.00	289.89	135.95
878.00	289.80	138.29
879.00	289.72	140.55
880.00	289.63	142.79
881.00	289.55	145.09
882.00	289.46	147.53
883.00	289.38	150.18
884.00	289.29	153.06
885.00	289.21	156.17
886.00	289.12	159.48
887.00	289.04	162.93
888.00	288.95	166.46
889.00	288.87	170.04
890.00	288.78	173.61
891.00	288.70	177.16
892.00	288.61	180.65
893.00	288.53	184.04
894.00	288.44	187.29
895.00	288.36	190.31
896.00	288.27	193.02
897.00	288.19	195.36
898.00	288.10	197.27
899.00	288.02	198.74
900.00	287.93	199.77
901.00	287.85	200.43
902.00	287.76	200.75
903.00	287.68	200.81
904.00	287.59	200.66
905.00	287.50	200.34
906.00	287.42	199.87
907.00	287.33	199.25
908.00	287.25	198.49

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
909.00	287.16	197.55
910.00	287.08	196.43
911.00	286.99	195.09
912.00	286.91	193.53
913.00	286.82	191.76
914.00	286.74	189.77
915.00	286.65	187.58
916.00	286.57	185.22
917.00	286.48	182.74
918.00	286.40	180.18
919.00	286.31	177.63
920.00	286.23	175.15
921.00	286.14	172.81
922.00	286.06	170.68
923.00	285.97	168.81
924.00	285.89	167.20
925.00	285.80	165.84
926.00	285.72	164.69
927.00	285.63	163.70
928.00	285.55	162.80
929.00	285.46	161.94
930.00	285.38	161.10
931.00	285.29	160.28
932.00	285.21	159.49
933.00	285.12	158.81
934.00	285.04	158.28
935.00	284.95	157.99
936.00	284.87	158.00
937.00	284.78	158.38
938.00	284.70	159.17
939.00	284.61	160.38
940.00	284.52	162.02
941.00	284.44	164.06
942.00	284.35	166.45
943.00	284.25	169.12
944.00	284.14	172.02
945.00	284.03	175.05
946.00	283.91	178.16
947.00	283.78	181.26
948.00	283.64	184.27
949.00	283.50	187.15
950.00	283.35	189.86
951.00	283.19	192.38
952.00	283.03	194.74
953.00	282.86	196.99
954.00	282.70	199.15
955.00	282.53	201.25
956.00	282.36	203.27
957.00	282.19	205.15

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
958.00	282.01	206.77
959.00	281.84	208.03
960.00	281.67	208.81
961.00	281.49	209.02
962.00	281.32	208.61
963.00	281.15	207.58
964.00	280.97	205.96
965.00	280.80	203.80
966.00	280.63	201.20
967.00	280.45	198.24
968.00	280.28	195.00
969.00	280.10	191.57
970.00	279.93	188.03
971.00	279.76	184.42
972.00	279.58	180.77
973.00	279.41	177.13
974.00	279.24	173.53
975.00	279.06	170.00
976.00	278.89	166.59
977.00	278.71	163.40
978.00	278.54	160.49
979.00	278.37	157.95
980.00	278.19	155.83
981.00	278.02	154.15
982.00	277.84	152.87
983.00	277.67	151.93
984.00	277.50	151.24
985.00	277.32	150.69
986.00	277.15	150.15
987.00	276.97	149.55
988.00	276.80	148.80
989.00	276.63	147.86
990.00	276.45	146.73
991.00	276.28	145.43
992.00	276.11	144.00
993.00	275.93	142.51
994.00	275.76	141.04
995.00	275.59	139.65
996.00	275.41	138.43
997.00	275.24	137.43
998.00	275.07	136.74
999.00	274.90	136.42
1000.00	274.73	136.54
1001.00	274.56	137.16
1002.00	274.39	138.31
1003.00	274.21	139.97
1004.00	274.04	142.09
1005.00	273.87	144.55
1006.00	273.70	147.22

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
1007.00	273.53	149.90
1008.00	273.36	152.43
1009.00	273.19	154.61
1010.00	273.02	156.33
1011.00	272.85	157.50
1012.00	272.68	158.12
1013.00	272.50	158.24
1014.00	272.33	157.97
1015.00	272.16	157.42
1016.00	271.99	156.69
1017.00	271.82	155.86
1018.00	271.65	154.96
1019.00	271.48	153.98
1020.00	271.31	152.88
1021.00	271.14	151.62
1022.00	270.97	150.14
1023.00	270.81	148.44
1024.00	270.65	146.49
1025.00	270.49	144.33
1026.00	270.34	142.01
1027.00	270.19	139.60
1027.90	270.06	137.17
1028.70	269.93	134.81
1029.40	269.81	132.62
1030.01	269.70	130.67
1030.54	269.60	129.02
1031.00	269.51	127.70
1031.40	269.42	126.72
1031.75	269.35	126.05
1032.05	269.28	125.68
1032.32	269.22	125.55
1032.55	269.17	125.63
1032.75	269.11	125.89
1032.95	269.07	126.31
1033.15	269.01	126.86
1033.35	268.88	127.53
1033.55	268.52	128.28
1033.75	267.72	129.04
1033.95	266.33	129.75
1034.15	264.28	130.35
1034.35	261.65	130.78
1034.55	258.60	131.03
1034.75	255.34	131.13
1034.95	252.06	131.16
1035.15	248.96	131.26
1035.35	246.15	131.59
1035.55	243.66	132.31
1035.75	241.46	133.55
1035.95	239.43	135.34

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
1036.15	237.48	137.66
1036.35	235.53	140.37
1036.55	233.56	143.24
1036.75	231.59	146.00
1036.95	229.66	148.38
1037.15	227.85	150.12
1037.35	226.20	151.04
1037.55	224.73	151.03
1037.75	223.45	150.07
1037.95	222.34	148.22
1038.15	221.36	145.61
1038.35	220.48	142.38
1038.55	219.65	138.70
1038.75	218.84	134.71
1038.95	218.04	130.53
1039.15	217.22	126.23
1039.35	216.37	121.87
1039.55	215.48	117.47
1039.75	214.56	113.04
1039.95	213.59	108.61
1040.15	212.56	104.23
1040.35	211.48	99.98
1040.55	210.33	95.99
1040.75	209.12	92.42
1040.95	207.87	89.45
1041.15	206.61	87.26
1041.35	205.36	85.99
1041.55	204.14	85.73
1041.75	202.98	86.48
1041.95	201.89	88.19
1042.18	200.87	90.70
1042.44	199.91	93.83
1042.74	199.00	97.37
1043.09	198.13	101.12
1043.49	197.30	104.93
1044.02	196.50	108.70
1044.48	195.72	112.39
1045.00	194.95	115.99
1045.50	194.19	119.53
1046.00	193.44	123.03
1046.50	192.69	126.49
1047.00	191.96	129.88
1047.50	191.23	133.14
1048.00	190.52	136.22
1048.50	189.82	139.04
1049.00	189.13	141.51
1049.50	188.45	143.60
1050.00	187.78	145.26
1050.50	187.13	146.49

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
1051.00	186.48	147.33
1051.50	185.84	147.80
1052.00	185.21	147.97
1052.50	184.58	147.91
1053.00	183.95	147.69
1053.50	183.33	147.39
1054.00	182.72	147.07
1054.50	182.13	146.81
1055.00	181.55	146.65
1055.50	181.00	146.63
1056.00	180.48	146.74
1056.50	179.99	146.97
1057.00	179.54	147.28
1057.50	179.11	147.63
1058.00	178.71	148.01
1058.50	178.34	148.41
1059.00	177.98	148.83
1059.50	177.64	149.31
1060.00	177.31	149.87
1060.50	177.00	150.52
1061.00	176.68	151.24
1061.50	176.38	151.99
1062.00	176.08	152.74
1062.50	175.78	153.41
1063.00	175.48	153.95
1063.50	175.18	154.31
1064.00	174.89	154.45
1064.50	174.59	154.34
1065.00	174.30	153.98
1065.50	174.00	153.36
1066.00	173.71	152.50
1066.50	173.41	151.41
1067.00	173.12	150.11
1067.50	172.82	148.63
1068.00	172.53	147.00
1068.50	172.24	145.27
1069.00	171.94	143.48
1069.50	171.65	141.67
1070.00	171.35	139.89
1070.50	171.06	138.16
1071.00	170.77	136.53
1071.50	170.48	135.01
1072.00	170.20	133.62
1072.50	169.92	132.37
1073.00	169.65	131.26
1073.50	169.39	130.31
1074.00	169.14	129.54
1074.50	168.89	128.95
1075.00	168.65	128.57

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
1075.50	168.42	128.40
1076.00	168.19	128.45
1076.50	167.97	128.71
1077.00	167.75	129.17
1077.50	167.53	129.78
1078.00	167.32	130.50
1078.50	167.10	131.28
1079.00	166.89	132.07
1079.50	166.68	132.81
1080.00	166.46	133.47
1080.50	166.25	134.02
1081.00	166.04	134.46
1081.50	165.83	134.77
1082.00	165.62	134.96
1082.50	165.40	135.05
1083.00	165.19	135.04
1083.50	164.98	134.96
1084.00	164.77	134.80
1084.50	164.56	134.57
1085.00	164.35	134.30
1085.50	164.13	133.96
1086.00	163.92	133.56
1086.50	163.71	133.08
1087.00	163.50	132.50
1087.50	163.29	131.80
1088.00	163.08	130.97
1088.50	162.86	129.99
1089.00	162.65	128.87
1089.50	162.44	127.63
1090.00	162.23	126.28
1090.50	162.02	124.86
1091.00	161.81	123.42
1091.50	161.60	121.97
1092.00	161.39	120.56
1092.50	161.18	119.21
1093.00	160.98	117.92
1093.50	160.80	116.70
1094.00	160.62	115.58
1094.50	160.45	114.55
1095.00	160.30	113.65
1095.30	160.15	112.89
1095.60	160.01	112.31
1095.80	159.88	111.93
1096.00	159.76	111.79
1096.50	159.63	111.90
1097.00	159.51	112.29
1097.50	159.39	112.99
1098.00	159.27	114.02
1098.50	159.15	115.42

Table C1

Distance from Source (m)	Bed Elevation (m)	Top Width (m)
1099.00	159.03	117.22
1099.50	158.91	119.44
1100.00	158.79	122.09
1100.50	158.67	125.15
1101.00	158.55	128.63
1101.50	158.43	132.50
1102.00	158.31	136.74
1102.50	158.19	141.34
1103.00	158.07	146.31
1103.50	157.95	151.70
1104.00	157.83	157.60
1104.50	157.71	164.15
1105.00	157.58	171.50
1105.50	157.45	179.82
1106.00	157.32	189.22
1106.50	157.17	199.76
1107.00	157.01	211.39
1107.50	156.84	223.97
1108.00	156.66	237.26
1108.50	156.46	250.98
1109.00	156.25	264.84
1109.50	156.04	278.58
1110.00	155.81	292.01
1110.50	155.58	305.01
1111.00	155.35	317.57
1111.50	155.11	329.73
1112.00	154.87	341.62
1112.50	154.62	353.38
1113.00	154.38	365.13

Table C2 Summary of Data for ICEJAM Configuration Runs

Run #	Qet	Δt	L	X(Jam)	Δx(Jam)/L	Qo	Qet-Qo/Qo	Δt Qo^.33/L	d
run24	959.5	36.6	5	800	61.6	900	0.06615889	70.65779973	8.97
run23	989.0	36.5	10	800	30.8	900	0.09893778	35.23237282	9.34
run22	1192.3	36.2	50	800	6.16	900	0.32477778	6.988558334	8.50
run6	536.0	46.9	5	800	61.6	500	0.071952	74.35433492	6.56
run5	554.6	46.6	10	800	30.8	500	0.109216	36.95233222	6.81
run4	697.3	45.4	50	800	6.16	500	0.394662	7.200017775	6.24
run18	217.0	68.6	5	800	61.6	200	0.08517	80.20138762	4.10
run17	227.0	67.8	10	800	30.8	200	0.134945	39.63293359	4.23
run16	310.2	63.9	50	800	6.16	200	0.551135	7.476367507	3.94
run21	1172.1	21.3	5	920	37.6	900	0.30232222	41.18487416	9.15
run20	1193.6	21.3	10	920	18.8	900	0.32625556	20.51199787	8.97
run19	1550.2	21.3	50	920	3.76	900	0.7224	4.112052279	11.81
run3	655.4	27.0	5	920	37.6	500	0.310892	42.90385316	6.59
run2	676.2	26.9	10	920	18.8	500	0.352318	21.31967056	6.47
run1	899.0	26.4	50	920	3.76	500	0.79806	4.18987074	8.43
run12	266.0	38.5	5	920	37.6	200	0.33015	45.06090114	3.99
run11	282.3	38.1	10	920	18.8	200	0.411365	22.28682545	3.93
run10	399.2	36.8	50	920	3.76	200	0.996115	4.301445018	5.03
run9	804.8	3.9	5	1070	7.6	500	0.60952	6.136679367	5.72
run8	977.5	3.8	10	1070	3.8	500	0.954954	3.015437275	5.78
run7	1776.6	3.6	30	1070	1.26666667	500	2.55322	0.95224335	6.55
run15	377.2	5.4	5	1070	7.6	200	0.8858	6.314762962	3.88
run14	488.0	5.2	10	1070	3.8	200	1.439875	3.040441426	3.89
run13	1052.6	4.7	30	1070	1.26666667	200	4.26285	0.912782095	8.31
rune	1502.1	1.0	5	1095.6	2.48	900	0.669	1.86618961	8.34
rund	1753.8	1.1	10	1095.6	1.24	900	0.9487	1.013534012	8.12
run27	2156.5	1.3	20	1095.6	0.62	900	1.39613333	0.603294055	8.32
runc	3070.5	2.2	50	1095.6	0.248	900	2.41165556	0.424719015	8.13
run26	1334.9	1.6	20	1095.6	0.62	500	1.66986	0.6348289	6.17
runb	2032.2	2.8	50	1095.6	0.248	500	3.06442	0.439089989	6.50
run25	658.5	2.3	20	1095.6	0.62	200	2.292445	0.667532813	3.95
runa	1135.6	3.8	50	1095.6	0.248	200	4.67805	0.446321209	4.74

Runs Done With Ice Cover Intact

Run #	Qet	Δt	L	X(Jam)	Δx(Jam)/L	Qo	Qet-Qo/Qo	Δt Qo^.33/L	d
run36	514.1	60.1	5	800	61.6	500	0.0281	95.43594464	?
run28	538.6	60.3	20	800	15.4	500	0.077162	23.92511417	6.64
run35	590.2	61.0	50	800	6.16	500	0.180464	9.681140726	6.24
run34	543.3	35.9	5	920	37.6	500	0.086576	56.92299137	6.59
run29	574.8	36.1	20	920	9.4	500	0.149546	14.31010146	6.31
run33	662.1	37.0	50	920	3.76	500	0.324286	5.869522205	8.44
run32	657.5	1.6	5	1095.6	2.48	500	0.315046	2.565766804	6.18
run30	[illegible]	2.3	20	1095.6	0.62	500	0.79761	0.899340942	6.16
run31	[illegible]	4.2	50	1095.6	0.248	500	1.60566	0.671860586	6.47

Runs Done With Open Water Downstream

Run #	Qet	Δt	L	X(Jam)	Δx(Jam)/L	Qo	Qet-Qo/Qo	Δt Qo^.33/L	d
run6	536.0	46.9	5	800	61.6	500	0.071952	74.35433492	6.56
run5	554.6	46.6	10	800	30.8	500	0.109216	36.95233222	6.81
run4	697.3	45.4	50	800	6.16	500	0.394662	7.200017775	6.24
run3	655.4	27.0	5	920	37.6	500	0.310892	42.90385316	6.59
run2	676.2	26.9	10	920	18.8	500	0.352318	21.31967056	6.47
run1	899.0	26.4	50	920	3.76	500	0.79806	4.18987074	8.43
run26	1334.9	1.6	20	1095.6	0.62	500	1.66986	0.6348289	6.17
runb	2032.2	2.8	50	1095.6	0.248	500	3.06442	0.439089989	6.50

APPENDIX D

ICEJAM program listing

What follows is the program listing for ICEJAM written in TURBObasic™. It is noted that the various formulae can be changed by the user but that the input/output components are portion of the FALCONeditor™ which is proprietary. However the source code for both components is provided to allow changes to the ICEJAM component to be recompiled, but it is to be understood that the FALCONeditor™ portion of the program is protected by copyright as described in the licence agreement which follows. If changes are required to the FALCONeditor™ component, please contact HYprocessing at the address given in Appendix C.

```
'       ***************************************************************
'       *                   FIRST LEVEL FLOOD WATCH                   *
'       ***************************************************************
'       *                  VER 1.02 - Dec 03, 1990                    *
'       ***************************************************************

SUB FLFW
SHARED V$(),FORE%,BACK%,FOR1%,A$,H1,H2,H3,H4

      SHR = VAL(V$(2,1))              '     SNOWFALL AT HAY RIVER
      SFN = VAL(V$(2,2))              '     SNOWFALL AT FORT NELSON
      SHL = VAL(V$(2,3))              '     SNOWFALL AT HIGH LEVEL
      H1=0:H2=0:H3=0:H4=0             '     HELP NOTES

'       ***************************************************************
'       *              BASIN AVERAGE ACCUMULATED SNOWFALL             *
'       ***************************************************************

      SAV = (0.10*SHR) + (0.34*SFN) + (0.56*SHL)
      V$(2,10)=STR$(INT(SAV))
      CALL QPRINT(FOR1%,BACK%,10,47,4,V$(2,10))
      IF SAV>180 THEN H1=4:H2=5
      IF SAV< 55 THEN H1=1:H2=2:H3=2:GOTO 30110

'       ***************************************************************
'       *                 MAXIMUM LIKELY DISCHARGES                   *
'       ***************************************************************

      Q1 = -290 + (5.78 * SAV)
      IF SAV =>97 THEN 30105
      Q2 = -800 + (16.7 * SAV)
      GOTO 30120
30105 Q2 = 21.0 * (SAV^0.80)
      GOTO 30120

30110 Q1=10:Q2=10
30120 V$(2,11)=STR$(INT(Q1)):V$(2,12)=STR$(INT(Q2))
      CALL QPRINT(FOR1%,BACK%,12,23,6,V$(2,11))
      CALL QPRINT(FOR1%,BACK%,12,33,6,V$(2,12))
      IF Q1<200 OR Q2<200 THEN H3=8
      IF Q1<200 AND Q2<200 THEN 30200

'       ***************************************************************
'       *                   MAXIMUM LIKELY STAGES                     *
'       ***************************************************************

30140 IF Q1<200 THEN 30150
      ST1 = 0.222 * (Q1^0.4713)
30150 ST2 = 0.222 * (Q2^0.4713)

      IF ST1<0 OR ST2<0 THEN H3=3:H4=9

'       ***************************************************************
'       *              CALCULATE GEODETIC WATER LEVELS                *
'       ***************************************************************

      IF Q1<200 THEN 30160
      WL1 = 156.6 + (1.2*ST1)
```

```
       CALL FixDec(WL1)
       V$(2,13)=A$
       CALL QPRINT(FOR1%,BACK%,15,23,6,V$(2,13))

30160  WL2 = 156.6 + (1.2*ST2)
       CALL FixDec(WL2)
       V$(2,14)=A$
       CALL QPRINT(FOR1%,BACK%,15,33,6,V$(2,14))

'      ****************************************************************
'      *               CALCULATE WATER LEVELS ON THE PIERS            *
'      ****************************************************************

       IF Q1<200 THEN 30170
       WL3 = 3.2808 * (WL1-157.55)
       CALL FixDec(WL3)
       V$(2,15)=A$
       CALL QPRINT(FOR1%,BACK%,16,23,6,V$(2,15))

30170  WL4 = 3.2808 * (WL2-157.55)
       CALL FixDec(WL4)
       V$(2,16)=A$
       CALL QPRINT(FOR1%,BACK%,16,33,6,V$(2,16))
       IF Q1>1600 OR Q2>1600 THEN H3=6:H4=7
       IF Q1<200 THEN 30210
       CALL HLP(H1,H2,H3,H4)
       EXIT SUB

30200  V$(2,14)="   ?  ":V$(2,16)="   ?  "
       CALL QPRINT(FOR1%,BACK%,15,33,6,"   ?  ")
       CALL QPRINT(FOR1%,BACK%,16,33,6,"   ?  ")
30210  V$(2,13)="   ?  ":V$(2,15)="   ?  "
       CALL QPRINT(FOR1%,BACK%,15,23,6,"   ?  ")
       CALL QPRINT(FOR1%,BACK%,16,23,6,"   ?  ")
       CALL HLP(H1,H2,H3,H4)
       EXIT SUB

END SUB


'      ****************************************************************
'      *                    SECOND LEVEL FLOOD WATCH                  *
'      ****************************************************************

SUB SLFWNJ
SHARED V$(),WL(),FORE%,BACK%,FOR1%,A$,WL1,WL2,DH1,DH2,STFLG,H1,H2,H3,H4

       Q  = VAL(V$(3,1))         '      HAY RIVER DISCHARGE
       B  = VAL(V$(3,3))         '      BRIGHT SUNSHINE
       Sn = VAL(V$(3,4))         '      SNOW AT HAY RIVER
       H1=0:H2=0:H3=0:H4=0       '      HELP NOTES

'      ****************************************************************
'      *                 STAGE AND SURGE CORRECTION       NO JAM      *
'      ****************************************************************

       E  = B - (1.2*Sn)
       IF E<0 THEN E=0
```

```
        R  = 1.2 - (0.000024*E*E)
        IF R<1.0 THEN R=1.0:H3=14
        ST1 = 0.2193 * (Q^0.4489)      '     OPEN W.C.
        ST2 = 0.2220 * (Q^0.4713)      '     JAMMED W.C.
        IF Q<200 THEN H1=10:H2=11:GOTO 30300
        IF Q>1600 THEN H1=12:H2=13
        IF ST1<0 OR ST2<0 THEN H3=3:H4=9
'
'       **************************************************************
'       *                 WATER LEVEL UPSTREAM OF W.C.B.             *
'       **************************************************************

'       WEST CHANNEL OPEN

        WLA1 = 156.6 + ST1             '     GEODETIC - NO SURGE
        WLB1 = 3.2808 * (WLA1-157.55)  '     ON PIER  - NO SURGE
        WLC1 = 156.6 + (R * ST1)       '     GEODETIC - WITH SURGE
        WLD1 = 3.2808 * (WLC1-157.55)  '     ON PIER  - WITH SURGE

'       WEST CHANNEL JAMMED

        WLA2 = 156.6 + ST2             '     GEODETIC - NO SURGE
        WLB2 = 3.2808 * (WLA2-157.55)  '     ON PIER  - NO SURGE
        WLC2 = 156.6 + (R * ST2)       '     GEODETIC - WITH SURGE
        WLD2 = 3.2808 * (WLC2-157.55)  '     ON PIER  - WITH SURGE

        WL1  = WLA1
        WL2  = WLA2
        DH1  = WLC1-WLA1
        DH2  = WLC2-WLA2

        WL(1)=WLA1:WL(2)=WLB1:WL(3)=WLC1:WL(4)=WLD1
        WL(5)=WLA2:WL(6)=WLB2:WL(7)=WLC2:WL(8)=WLD2

        FOR I=1 TO 8
        CALL FixDec(WL(I))
        V$(3,10+I)=A$
        NEXT
        FOR I=1 TO 2
        CALL QPRINT(FOR1%,BACK%,11+I,55,6,V$(3,I+10))
        CALL QPRINT(FOR1%,BACK%,11+I,66,6,V$(3,I+12))
        NEXT
        FOR I=5 TO 6
        CALL QPRINT(FOR1%,BACK%,9+I,55,6,V$(3,I+10))
        CALL QPRINT(FOR1%,BACK%,9+I,66,6,V$(3,I+12))
        NEXT
        CALL HLP(H1,H2,H3,H4)
        STFLG=0
        EXIT SUB
30300   STFLG=1
        FOR I=1 TO 4
        V$(3,I+10)="    ?   ":V$(3,I+14)="    ?   "
```

```
        CALL QPRINT(FOR1%,BACK%,I+11,55,6,"   ?  ")
        CALL QPRINT(FOR1%,BACK%,I+11,66,6,"   ?  ")
        NEXT
        CALL HLP(H1,H2,H3,H4)
        EXIT SUB

END SUB

'       ****************************************************************
'       *          SECOND LEVEL FLOOD WATCH WITH ICE JAM INFO           *
'       ****************************************************************

SUB SLFWWJ
SHARED V$(),WL(),FORE%,BACK%,FOR1%,A$,WL1,WL2,DH1,DH2,STFLG,H1,H2,H3,H4

        Q  = VAL(V$(5,1))          '    HAY RIVER DISCHARGE
        XX = VAL(V$(5,2))          '    DISTANCE TO ICE JAM
        LL = VAL(V$(5,3))          '    LENGTH OF ICE JAM
        H1=0:H2=0:H3=0:H4=0             '    HELP NOTES

'       ****************************************************************
'       *                   STAGE AND SURGE CALCULATION                 *
'       ****************************************************************

        IF LL>0 AND XX>0 THEN 30310
        DT=0:QO=0:QC=0
        EXIT SUB
        GOTO 30350

30310 IF (XX/LL)<1.0 THEN DT = 0.771 * ((XX/LL)^-0.5918) ELSE DT = 0.796
              * ((XX/LL)^0.1123)
        DT = DT * XX/(Q^0.3333)
        IF XX<290 THEN 30320
        QO = Q + Q * (1.609 * ((XX/LL)^-0.7570))
        QC = Q + Q * (0.7417 * ((XX/LL)^-0.8013))
        GOTO 30350
30320 IF XX<=70 THEN 30330
        QO = Q + Q * (1.444 * ((XX/LL)^-0.4391))
        QC = Q + Q * (0.6163 * ((XX/LL)^-0.5590))
        GOTO 30350
30330 QO = Q + Q * (1.219 * ((XX/LL)^-0.7074))
        QC = Q + Q * (0.5893 * ((XX/LL)^-0.7043))
30350 ST1  = 0.2193 * (Q^0.4489)     '    OPEN W.C.
        ST1O = 0.2193 * (QO^0.4489)    '    OPEN W.C. - D/S OPEN
        ST1C = 0.2193 * (QC^0.4489)    '    OPEN W.C. - D/S COVERED
        ST2  = 0.2220 * (Q^0.4713)     '    JAMMED W.C.
        ST2O = 0.2220 * (QO^0.4713)    '    JAMMED W.C. - D/S OPEN
        ST2C = 0.2220 * (QC^0.4713)    '    JAMMED W.C. - D/S COVERED
        IF Q<200 THEN H1=10:H2=11:GOTO 30400
        IF Q>1600 THEN H1=12:H2=13
        IF ST1<0 OR ST2<0 THEN H3=3:H4=9
        IF ST1O<0 OR ST2O<0 THEN H3=3:H4=9
        IF ST1C<0 OR ST2C<0 THEN H3=3:H4=9
```

```
'       ****************************************************************
'       *                WATER LEVEL UPSTREAM OF W.C.B.                *
'       ****************************************************************

'       WEST CHANNEL OPEN

        WLA1  = 156.6 + ST1              'GEODETIC - NO SURGE
        WLB1  = 3.2808 * (WLA1-157.55)   'ON PIER  - NO SURGE
        WLC1O = 156.6 + ST1O             'GEODETIC - WITH SURGE - D/S OPEN
        WLC1C = 156.6 + ST1C             'GEODETIC - WITH SURGE - D/S COVERED
        WLD1O = 3.2808 * (WLC1O-157.55)'ON PIER  - WITH SURGE - D/S OPEN
        WLD1C = 3.2808 * (WLC1C-157.55)'ON PIER  - WITH SURGE - D/S COVERED

'       WEST CHANNEL JAMMED

        WLA2  = 156.6 + ST2              'GEODETIC - NO SURGE
        WLB2  = 3.2808 * (WLA2-157.55)   'ON PIER  - NO SURGE
        WLC2O = 156.6 + ST2O             'GEODETIC - WITH SURGE - D/S OPEN
        WLC2C = 156.6 + ST2C             'GEODETIC - WITH SURGE - D/S COVERED
        WLD2O = 3.2808 * (WLC2O-157.55)'ON PIER  - WITH SURGE - D/S OPEN
        WLD2C = 3.2808 * (WLC2C-157.55)'ON PIER  - WITH SURGE - D/S COVERED

        WL1   = WLC1O
        WL2   = WLC2O
        DH1   = 0.0
        DH2   = 0.0

        WL(1)=WLA1:WL(2)=WLB1:WL(3)=WLC1C:WL(4)=WLD1C:WL(5)=WLC1O:WL(6)=WLD1O
        WL(7)=WLA2:WL(8)=WLB2:WL(9)=WLC2C:WL(10)=WLD2C:WL(11)=WLC2O:WL(12)
              =WLD2O

          CALL FixDec(QC)
          V$(5,6)=A$
          CALL QPRINT(FOR1%,BACK%,9,50,6,V$(5,6))
          CALL FixDec(QO)
          V$(5,7)=A$
          CALL QPRINT(FOR1%,BACK%,9,60,6,V$(5,7))
          CALL FixDec(DT)
          V$(5,8)=A$
          CALL QPRINT(FOR1%,BACK%,10,34,6,V$(5,8))

        FOR I=1 TO 12
        CALL FixDec(WL(I))
        V$(5,10+I)=A$
        NEXT
        FOR I=1 TO 2
        CALL QPRINT(FOR1%,BACK%,13+I,46,6,V$(5,I+10))
        CALL QPRINT(FOR1%,BACK%,13+I,56,6,V$(5,I+12))
        CALL QPRINT(FOR1%,BACK%,13+I,67,6,V$(5,I+14))
        NEXT
        FOR I=7 TO 8
        CALL QPRINT(FOR1%,BACK%,9+I,46,6,V$(5,I+10))
        CALL QPRINT(FOR1%,BACK%,9+I,56,6,V$(5,I+12))
        CALL QPRINT(FOR1%,BACK%,9+I,67,6,V$(5,I+14))
        NEXT
        CALL HLP(H1,H2,H3,H4)
        STFLG=0
```

```
      EXIT SUB
30400 STFLG=1
      FOR I=1 TO 4
      V$(5,I+10)="    ?   ":V$(5,I+14)="    ?   ":V$(5,I+18)="    ?   "
      CALL QPRINT(FOR1%,BACK%,I+13,46,6,"   ?  ")
      CALL QPRINT(FOR1%,BACK%,I+13,56,6,"   ?  ")
      CALL QPRINT(FOR1%,BACK%,I+13,67,6,"   ?  ")
      NEXT
      CALL HLP(H1,H2,H3,H4)
      EXIT SUB

END SUB


'      ************************************************************
'      *                SECOND LEVEL FLOOD WATCH (2)              *
'      ************************************************************

SUB SLFW2
SHARED V(),V$(),WLL(),FORE%,BACK%,FOR1%,A$,WL1,WL2,DH1,DH2,H1,H2,H3,H4

'      ************************************************************
'      *                    SURGE DISCHARGES                      *
'      ************************************************************

       Q1 = 29.373 * ((WL1-156.6)^2.2277)
       Q2 = 24.125 * ((WL2-156.6)^2.1213)
```

Code for Q1 and Q2 superceded by subsequent model work. Left in for possible future use.

```
'      ************************************************************
'      *                 WEST CHANNEL DISCHARGES                  *
'      ************************************************************

       QWOW = 15.05 * ((WL1-156.6)^2.2693)
       QWIJ = 10.06 * ((WL2-156.6)^2.022)

'      ************************************************************
'      *               FISHING VILLAGE CALCULATIONS               *
'      ************************************************************

       IF QWOW<440 THEN FVOW = (156.6 + (0.1121 * (QWOW^0.5841)) - 0.72 +
         DH1) ELSE FVOW = (156.6 + (0.6682 * (QWOW^0.2911)) - 0.72 + DH1)
       IF QWIJ<440 THEN FVIJ = (156.6 + (0.1121 * (QWIJ^0.5841)) - 0.72 +
         DH2) ELSE  FVIJ = (156.6 + (0.6682 * (QWIJ^0.2911)) - 0.72 + DH2)
       DFVOW = 3.2808 * (FVOW-158.6)
       DFVIJ = 3.2808 * (FVIJ-158.6)

'      ************************************************************
'      *            EAST CHANNEL (DOCKS) CALCULATIONS             *
'      ************************************************************

       QEOW  = 14.418 * ((WL1-156.6)^2.1764)
       QEIJ  = 14.418 * ((WL2-156.6)^2.1764)
       ECOW  = 156.1 + (0.0409 * (QEOW^0.6678)) + DH1
       ECIJ  = 156.1 + (0.0409 * (QEIJ^0.6678)) + DH2
       DECOW = 3.2808 * (ECOW-158.3)
       DECIJ = 3.2808 * (ECIJ-158.3)
```

```
'       *************************************************************
'       *               EAST CHANNEL (FILL C) CALCULATIONS          *
'       *************************************************************

        FCOW  = WL1 - 0.71 + DH1
        FCIJ  = WL2 - 0.71 + DH2
        DFCOW = 3.2808 * (FCOW-163)
        DFCIJ = 3.2808 * (FCIJ-163)

'       *************************************************************
'       *               SCREEN DISPLAY AND PRINTER PASS             *
'       *************************************************************

        H1=0:H2=0:H3=0:H4=0             '     HELP NOTES

        FOR I=1 TO 24:WLL(I)=0:NEXT
        WLL(8)=FVOW:WLL(9)=DFVOW:WLL(11)=FVIJ:WLL(12)=DFVIJ
        WLL(14)=FCOW:WLL(15)=DFCOW:WLL(17)=FCIJ:WLL(18)=DFCIJ
        WLL(20)=ECOW:WLL(21)=DECOW:WLL(23)=ECIJ:WLL(24)=DECIJ
        FOR I=1 TO 24
        CALL FixDec(WLL(I))
        V$(4,I)=A$
        NEXT
        V$(4,1)=STR$(INT(QWOW)):V$(4,7)=V$(4,1):V$(4,4)=STR$(INT(QWIJ))
              :V$(4,10)=V$(4,4)
        V$(4,13)=STR$(INT(QEOW)):V$(4,19)=V$(4,13):V$(4,16)=STR$(INT(QEIJ))
              :V$(4,22)=V$(4,16)
        IF V(1)=5 THEN 30500
        V$(4,2)=V$(3,13):V$(4,3)=V$(3,14):V$(4,5)=V$(3,17):V$(4,6)=V$(3,18)
        GOTO 30510
30500 V$(4,2)=V$(5,11):V$(4,3)=V$(5,12):V$(4,5)=V$(5,17):V$(4,6)=V$(5,18)
30510 FOR I=1 TO 6
        IF I>3 THEN RW=I+7 ELSE RW=I+6
        FOR J=1 TO 4
        CALL QPRINT(FORE%,BACK%,RW,32+(J*9),6,V$(4,(I-6)+(J*6)))
        NEXT
        NEXT
        EXIT SUB

END SUB
```

APPENDIX E

Surge Parameter Analysis

The objective is to find some simple functional relationships which will allow the presentation of the surge routing results in terms of variables readily determined in the field and for the easy incorporation of the results in the interactive program ICEJAM. Dimensional analysis can be of assistance in this.

To first consider the increase in discharge, ΔQ, at some distance Δx below an ice jam of length L for a carrier discharge of Q_o in a channel of slope S. For a kinematic-type river wave, which the surge approaches within a short distance downstream of the jam, it is likely a functional relation for ΔQ can be written as:

$$\Delta Q = f\,(Q_o, y_o, \Delta x, L, S, g \,......)$$

where y_o is the flow depth downstream, g is gravitational acceleration and the neglected variables, such as channel roughness, do not involve other dimensions and are either more-or-less constant or have a relatively negligible influence; in the interests of the required simplicity it is hoped the above variables will capture the essence of the phenomenon.

Applying dimensional analysis to the above yields

$$\frac{\Delta Q}{Q_o} = f\,(\frac{\Delta x}{L}, S, \frac{Q_o}{y_o^2\sqrt{(g y_o)}}, \frac{y_o}{L})$$

where the first two variables have an obvious significance, the first being the relative increase in discharge and the second the dimensional distance to the jam. The third parameter, the channel slope, has its major influence through its effect on the volume of water stored by the jam for a given channel and carrier discharge. As this slope varies significantly over the reach of interest, it may have some influence on the results. In the presentation in Section 6 of the report the slope has been handled by simply stratifying the presentation on the basis of jam position. The last two parameters, the first of which is a Froude number, were found to have a negligible effect.

The second requirement is for a simple expression for the surge travel time. In the expected that the surge travel time should depend primarily on the surge volume, the flow and depth downstream and the distance covered, so that the following functional relation can be written:

$$\Delta t = f\,(Q_o, y_o, L, \Delta x, S, g \,....)$$

As before the neglected parameters are presumed to be essentially constant or to have a relatively small influence on the phenomenon. Defining q as the discharge per unit width of channel, Q_o/B, the above relation can be rearranged to

$$\frac{\Delta t\, q}{y_o\, \Delta x} = f\left(\frac{\Delta x}{L}, S, \frac{Q_o}{y_o^2\sqrt{(g y_o)}}, \frac{y_o}{L}\right)$$

Now for a wide quasi-rectangular channel

$$q = C_* \, y_o \sqrt{(g\, y_o\, S)}$$

so that

$$y_o = \left(\frac{q^2}{C_*^2 g S}\right)^{1/3}$$

Substituting for y_o in the dimensional statement and reintroducing B this yields

$$\frac{\Delta t\, Q_o^{1/3}\, C_*^{2/3}\, (gS)^{1/3}}{B^{1/3}\, \Delta x} \approx f\left(\frac{\Delta x}{L}, S \ldots\right)$$

in which it has been assumed the parameter y_o/L is unlikely to have a significant influence on the surge celerity. On the assumption that C_* and B can be considered roughly constant over the reach, this can be written in dimensional form as

$$\frac{\Delta t\, Q_o^{1/3}}{\Delta x} \approx f\left(\frac{\Delta x}{L}, S \ldots\right)$$

This is the presentation used in Section 6, where Δt was taken in hours, Q_o in m^3/s and L in km. It was found that the channel slope had little influence on the travel time, nor did an ice cover.

GREAT SLAVE LAKE
HAY RIVER
Highway
Hay River
Big Island
Pointe Desmarais
Pointe de Roche
Kakisa
Tathlina Lake
Buffalo Lake
Buffalo River
Snake
Birch Creek
Yates
James
Little
Rapids
Creek
Whitesand
Cameron Hills
Mount Watt
Boyer
Ponton
Parma
28th Base Line
26th Base Line
30th Base Line
1st Base
Jackpot Creek
Swan Lake
IR 209
IR 211
SIXTH
N
700
600
800
900
1000
100
0 10 20 30 40 50 km