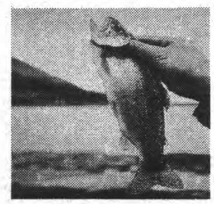
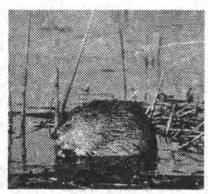
Canada Alberta Northwest Territories

Northern River Basins Study







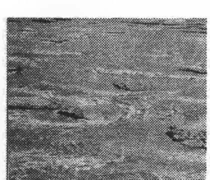




NORTHERN RIVER BASINS STUDY PROJECT REPORT NO. 76

A HYDRAULIC FLOOD ROUTING MODEL OF THE PEACE RIVER,

HUDSON HOPE TO PEACE POINT





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Prepared for the
Northern River Basins Study
under Project 1154-C1

by

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PREFACE:

The Northern River Basins Study was initiated through the "Canada-Alberta-Northwest Territories Agreement Respecting the Peace-Athabasca-Slave River Basin Study, Phase II - Technical Studies" which was signed September 27, 1991. The purpose of the Study is to understand and characterize the cumulative effects of development on the water and aquatic environment of the Study Area by coordinating with existing programs and undertaking appropriate new technical studies.

This publication reports the method and findings of particular work conducted as part of the Northern River Basins Study. As such, the work was governed by a specific terms of reference and is expected to contribute information about the Study Area within the context of the overall study as described by the Study Final Report. This report has been reviewed by the Study Science Advisory Committee in regards to scientific content and has been approved by the Study Board of Directors for public release.

It is explicit in the objectives of the Study to report the results of technical work regularly to the public. This objective is served by distributing project reports to an extensive network of libraries, agencies, organizations and interested individuals and by granting universal permission to reproduce the material.

NORTHERN RIVER BASINS STUDY
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Whereas the above publication is the result of a project conducted under the Northern River Basins Study and the terms of reference for that project are deemed to be fulfilled,
IT IS THEREFORE REQUESTED BY THE STUDY OFFICE THAT;
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F. J. Wrona — 10 Jan 96

(Dr. Fred J. Wrona, Science Director) (Date)

Whereas it is an explicit term of reference of the Science Advisory Committee "to review, for scientific content, material for publication by the Board",
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P. A. Larkin — Jan 9/96

(Dr. P. A. Larkin, Ph.D., Chair) (Date)

Whereas the Study Board is satisfied that this publication has been reviewed for scientific content and for immediate health implications,
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this publication be released to the public, and that this publication be designated for: **[ ] STANDARD AVAILABILITY [ ] EXPANDED AVAILABILITY**

Lucille Partington — Jan-11/96

(Lucille Partington, Co-chair) (Date)

Robert McLeod. — 11/01/96

(Robert McLeod, Co-chair) (Date)

A HYDRAULIC FLOOD ROUTING MODEL OF THE PEACE RIVER, HUDSON HOPE TO PEACE POINT

STUDY PERSPECTIVE

Construction of the Bennett Dam in British Columbia in 1967 altered the natural flow patterns of the Peace River. The effects of this change are discernable most immediately downstream of the dam, but also in the Peace - Athabasca Delta, almost 2000 km downstream and in the Slave River Delta, a further 500 km downstream. The effects of flow regulation on the river morphology, deltas and aquatic habitat are described in companion reports. Additionally, remote sensing is being investigated as a tool for assessing aquatic habitat. However, an accurate flow model is necessary to assist in these assessments. Earlier models had been developed using the Streamflow Synthesis and Reservoir Regulation (SSARR) hydrologic model. However, these models were only able to generate discharges at specific sites, specifically at the Water Survey of Canada (WSC) gauging sites. These SSARR models also had problems matching the modelled flows with the WSC gauge data in some instances.

This report describes a hydraulic flood routing model developed to accurately model the open water river discharge of moderate floods. This new model is capable of modelling the open water discharge at intermediate sites along the Peace River where no discharge data exists. In this first stage the model covers the Peace River from Hudson Hope to Peace Point. Subsequent work will focus on extending the model to the Slave River delta and collecting additional cross section data on the Peace River. It will now be possible to predict the discharge at various points along the Peace and Slave rivers more accurately than presently available models. The addition of a freeze up component to this model would increase its future utility.

Related Study Questions

10. *How does and how could river flow regulation impact the aquatic ecosystem?*

13.a) *What predictive tools are required to determine the cumulative effects of man-made discharges on the water and aquatic habitat?*

13.b) *What are the cumulative effects of man-made discharges on the water and aquatic environments?*

14. *What long term monitoring programs and predictive models are required to provide an ongoing assessment of the state of the aquatic ecosystems? These programs must ensure that all stakeholders have the opportunity for input.*

REPORT SUMMARY

The objective of this study was to develop a preliminary hydraulic flood routing model of the Peace River, between the Bennett Dam, in British Columbia, and Peace Point in Wood Buffalo National Park, Alberta. Although Alberta Environmental Protection hydrologists have successfully developed a hydrologic flood routing model of this reach using the "SSARR" model, output from models of this type is limited to discharge hydrographs at select sites. As several of the other components of the Northern River Basins Study (NRBS) require estimates of flow parameters not available from hydrologic models, such as stage and velocity, as well as discharge hydrographs at intermediate sites, a hydraulic flood routing model was needed. An additional advantage of this hydraulic flood routing model is that it has the ability to provide for an evaluation of the effects of ice on the propagation of flood hydrographs and, being fully dynamic, it can be used to route extreme events such as dam break floods and surges resulting from ice jam releases.

The project began with the development of a geometric data base describing the study reach. Under the terms of reference for this study, only available data (collected by other agencies) were used. This included surveyed cross section data and National Topographic Survey (N.T.S.) mapping. B.C. Hydro cross sections surveys extended from the dam to the B.C./Alberta Border. Localized cross section surveys were available downstream of the border at the Dunvegan Bridge, the town of Peace River, Fort Vermilion and at Peace Point. The largest break in available cross section surveys extended from the town of Peace River to Fort Vermilion, a distance of more than 400 km. Because of this paucity of data, an approximate model of the channel geometry had to be developed from other data sources, in particular: water surface slopes and channel top widths obtained from 1:250,000 scale NTS maps. Given the approximate nature of the geometric model and the fact that the hydraulic model was based on a one-dimensional approximation, a rectangular channel section was assumed. Comparisons to actual surveys confirmed that the surveyed river cross sections were well represented by this classical wide, rectangular channel approximation. The final geometric model consists of more than 1100 computational nodes describing channel width, effective bed elevation and channel roughness. No consideration of flood plain geometry could be provided at this stage, due to the limited field data available.

The hydraulic flood routing model used was the cdg-1D finite element model developed at the University of Alberta by F. Hicks and P. Steffler. The model provides for a solution of the fully dynamic, one-dimensional open channel flow equations (modified St. Venant equations). Although the model is capable of handling highly dynamic flood events (such as dam break floods or surges resulting from ice jam releases), the test scenarios examined for this preliminary study were simpler "diffusive" waves. As outlined in the terms of reference for this study, because of the exploratory nature of the research, the specific range of test scenarios was modified in consultation with Mr. John Taggart Alberta Environmental Protection (AEP) and Dr. Terry Prowse, Hydraulics/Hydrology/Sediment Project Leader, NRBS. After an examination of available hydrologic data the final range of tests was reduced to two events, one moderate and one large, specifically the 1980 spring runoff event and the 1987 summer flood event. Evaluations of non-regulated flows and the effects of ice on flood propagation were not considered warranted, given the lack of recorded data and the extensive gaps in surveyed geometry. However, it was intended that the model should provide enough information to assess where further surveys are needed, to facilitate future tests of this type.

The only calibration parameter involved in the development of the hydraulic model was the channel resistance coefficient, specifically Mannings n. Initial values for the parameter were based on the data provided by Kellerhals, Neill and Bray (1972) for 1:2 year flood events at gauge sites. Agreement between measured and computed flood hydrographs was good for both the moderate (1980) and extreme (1987) flood events and no further refinement, or "calibration" of the model was considered warranted until additional data is obtained in the unsurveyed reaches. It is stressed that because of the limited data available (both in terms of these unsurveyed reaches and the lack of overbank geometry) the model is still somewhat empirical.

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The authors would like to thank Dr. Terry Prowse, Hydraulics/Hydrology/Sediment Project Leader, NRBS, who supervised the contract, and provided effective guidance and support in its performance. Thanks are also due to Mr. Jim Choles, River Engineering Branch, Alberta Environmental Protection who obtained the available cross section data for the study and advised on the contract administration. The authors would also like to thank Mr. John Taggart, Hydrology Branch, Alberta Environmental Protection for providing the flood event test scenarios used in this study, and for his advice regarding Peace River flood routing considerations.

1.0 INTRODUCTION

The Northern River Basins Study (NRBS) seeks to assess the effects of flow regulation by the W.A.C. Bennett Dam on the Peace River under both open water and ice covered conditions, and the impact of this flow regulation on the geomorphologic evolution of the river. This requires a flood routing model capable of providing details of the water levels, velocities and discharges occurring at any point along the Peace River as a function of time. Although Alberta Environmental Protection (AEP) hydrologists have successfully developed a *hydrologic* flood routing model of this reach, output from their routing model is limited to discharge hydrographs at select sites because the hydrologic routing approach considers the effects of momentum conservation on the propagation of flood waves in a conceptual way. A *hydraulic* flood routing model is required to provide the type of information required by the NRBS.

Hydraulic models may be described as *deterministic*, based on physical laws and physical data. An adequate hydraulic model of a river reach requires not only a sophisticated mathematical model of the flow, it requires adequate information describing the channel geometry (cross section shape, bed slope, etc.) and its resistance characteristics (on the bed, banks and floodplains). In this case, details of channel geometry were scarce. Although survey data was available upstream of the B.C./ Alberta border, in Alberta cross section surveys have been conducted at very few sites. Specifically, cross section data in Alberta was available at Dunvegan, Peace River, Fort Vermilion and Peace Point only. Virtually hundreds of kilometers of river are unsurveyed in the intermediate reaches between these locations. Therefore, the major goal of the this project was to synthesize a data base for the entire study reach, including these intermediate portions, based on topographic map data and then to test the data base using the model to assess where future cross section surveys are required. Given the approximate nature of the geometric model and the fact that the hydraulic model was based on a one-dimensional approximation, a rectangular channel section was assumed.

Many successful hydraulic flood routing models have been developed, though none which incorporate the effects of ice on the flow are currently available commercially. Recent research in the Civil Engineering Department at the University of Alberta has led to the development of a numerically robust unsteady, open channel flow model which has already been used to assess the potential impact of ice jam release surges on the Hay River, NWT. This model employs a Petrov-Galerkin finite element method known as the characteristic-dissipative-Galerkin (cdg) scheme. Comparisons of this numerical scheme to more conventional, commercially available code, have been conducted, confirming the superiority of the CDG scheme in terms of both solution accuracy and numerical stability. In this study, the "cdg-1D" model, based on the application of this numerical scheme to the one-dimensional (fully-dynamic) equations of open channel flow, was used. This numerical model was executed on a 66 MHz, 486DX/2, IBM compatible computer running under the NeXTStep (UNIX) operating system.

Although this hydraulic model is capable of handling highly dynamic flood events (such as dam break floods and surges resulting from ice jam releases), the test scenarios prescribed by the terms of reference for this preliminary study were simpler "diffusive" waves in order to validate the hydraulic model before proceeding to more complex flow scenarios (for which no measured data is available for comparison). Specifically, the model was to be tested for both a moderate and an extreme flood event, as prescribed by Mr. John Taggart of AEP. The hydraulic model was to be used to route these flood events under open water and ice covered conditions, for both regulated and unregulated inflows, and the results of the open water analyses were to be compared with the results from AEP's *hydrologic* model. However it was recognized at that time that, because of the exploratory nature of the research, the specific range of test scenarios might require modification at the discretion of Mr. Taggart and

Dr. Terry Prowse, Hydraulics/Hydrology/Sediment Project Leader, NRBS. Upon evaluation of the limited available hydrologic data, Mr. Taggart recommended that the range of tests be reduced to the simulation of a moderate and an extreme event under *regulated, open water* conditions only. Given that the primary objective of this preliminary study was to synthesize the geometric database and validate the underlying hydraulic model before proceeding to consideration of ice and regulation effects, Dr. Prowse agreed that the final range of tests should be reduced to two events, specifically: the 1980 spring runoff event and the 1987 summer flood event.

Section 2 of this report presents the details of the development of this geometric database. Descriptions of the numerical method used and the equations modelled are provided in Section 3. Section 4 presents the results of the two flood routing simulations, including comparisons to measured streamflows and the hydrologic (SSARR) model results. Conclusions and recommendations are provided in Section 5.

2.0 DEVELOPMENT OF THE INPUT DATABASE FOR THE HYDRAULIC MODEL

2.1 INTRODUCTION

In this section details describing the development of the river geometry are presented, including the methods and assumptions involved in establishing this database. In addition, details of the recorded WSC data are reviewed as is the synthesis of tributary inflows (based on the approach used by AEP, in the development of their hydrologic model of the Peace River).

2.2 DEVELOPMENT OF THE GEOMETRIC DATABASE

The study reach extended from the WSC gauge at Hudson Hope (28 km downstream of W.A.C. Bennett Dam) to Peace Point in Wood Buffalo National Park, a distance of just over 1,100 km in terms of length measured along the channel centerline.

2.2.1 Channel Distances

River stations, or locations along the channel length, were obtained by marking out 1 km intervals on the 1:250,000 scale maps with dividers. The origin was specified as the downstream face of the W.A.C. Bennett Dam, and the stations were specified in kilometers (km) downstream of this origin. For consistency with earlier and future investigations, the stations were measured along the channel centreline, rather than along the thalweg as the latter is a more subjective criteria when limited cross section data is available. The difference between the channels stations obtained using these two criteria was marginal in this case, and the choice of channel centreline as the longitudinal axis was consistent with the assumption of a rectangular cross section shape. Each of the surveyed cross sections was referenced to this stationing system, as were all major tributaries and key sites of interest. Table 1 presents the location of these key sites along the river reach, in terms of their distances downstream of the dam.

Table 1. Location of key sites along the Peace River

Location	Station (km)
Peace River at Hudson Hope	28
Halfway River confluence	65
Moberly River confluence	103
Peace River at Fort St. John	110
Pine River confluence	120
Peace River at Taylor	121
Beaton River confluence	141
Kiskatinaw River confluence	154
British Columbia-Alberta Border	*166*
Clear River confluence	186
Peace River at Dunvegan Bridge	295
Smoky River confluence	388
Heart River confluence	394
Peace River at Peace River	395
Notikewin River confluence	558
Peace River near Carcajou	650
Peace River at Fort Vermilion	808
Boyer River confluence	819
Wabasca River confluence	865
Peace River at Peace Point	1107

2.2.2 Effective Bed Profile

Water surface slopes were obtained from 1:250,000 scale N.T.S. maps by identifying locations where the topographic contours intersected the river channel. The corresponding stations, in terms of distance downstream of the dam, were then used to determine water surface slopes. Table 2, below provides the water surface slopes obtained in this way and Figure 1 illustrates the map water surface profile along with the thalweg defined by the surveyed cross sections.

Table 2. Water surface slopes based on N.T.S. maps

Reach (km)	Water Surface Slope
24 to 71	0.00065
71 to 125	0.00056
125 to 213	0.00035
213 to 316	0.00030
316 to 414	0.00031
414 to 535	0.00025
535 to 887	0.00009
887 to 1083	0.00012

Effective bed elevations were obtained for each of the surveyed cross sections by first determining the water surface width and hydraulic mean depth (flow area/top width) at the 1:2 year flood level. The 1:2 year flood flows were estimated using the median of the pre-regulated flow records at WSC gauge sites along the Peace River, as described in Table 3 below:

Table 3. Estimates of 1:2 year flood discharges along the Peace River
(based on Water Survey of Canada published records)

Location (km)	Period of Record (years)	Median flow (m^3/s)
Peace River at Hudson Hope (27.6 km)	27	5,920
Peace River at Taylor (121.5 km)	26	6,935
Peace River at Dunvegan Bridge (295 km)	10	7,550
Peace River at Peace River (395 km)	31	8,380
Peace River near Carcajou (650 km)	8	9,755
Peace River at Fort Vermilion (808 km)	17	8,980
Peace River at Fifth Meridian (958 km)	7	9,200
Peace River at Peace Point (1107 km)	11	10,000

where "m^3/s" means cubic metres per second. The flow area and hydraulic mean depth were determined based on a steady, gradually varied flow analysis of each surveyed reach. These analyses were done using the U.S. Army Corps of Engineers HEC-2 model, and were based on Mannings roughness values obtained by Kellerhals, Neill and Bray (1972) for 1:2 year flood events, as summarized in Table 4. No refinement of these Mannings n values were considered warranted at this early stage, given the purpose of this analysis.

Table 4. Mannings *n* values used in the preliminary analysis
(after on Kellerhals, Neill and Bray, 1972)

Location (km)	Mannings n
Peace River at Hudson Hope (27.6 km)	0.031
Peace River at Taylor (121.5 km)	0.049
Peace River at Dunvegan Bridge (295 km)	0.021
Peace River at Peace River (395 km)	0.022
Peace River near Carcajou (650 km)	0.023
Peace River at Fort Vermilion (808 km)	0.017
Peace River at Peace Point (1107 km)	0.023

Effective bed elevations were defined at each surveyed cross section as: the computed (HEC-2) 1:2 year water surface elevation minus the hydraulic mean depth. To establish the effective bed profile at even 1 km increments for the hydraulic model, a best fit line was drawn through the effective bed points from the surveyed cross sections. Effective bed levels between the surveyed reaches were estimated by projecting values in the surveyed reaches using the water surface slopes obtained from the 1:250,000 NTS maps. Figure 2 shows the effective bed profile obtained by this method, illustrating the consistency of this approach.

2.2.3 Channel Widths

The channel widths used in the hydraulic model were also obtained from the NTS maps, by measuring the channel top width with scale and dividers at one kilometer intervals along the channel centreline. These top widths were then smoothed through the calculation of 3 point moving mean widths. Figure 3 presents the channel top widths based on the NTS maps.

2.2.4 Channel Resistance

Channel resistance, specifically Mannings *n*, was the only calibration parameter required for this hydraulic flood routing model. To initiate the calibration process, channel resistance was estimated based on the values presented by Kellerhals, Neill and Bray (1972) for 1:2 year flood events, as summarized in Table 4. Table 5, below, presents the values used in the various Peace River sub-reaches (obtained assuming the local values cited in Table 4 were valid halfway to each adjacent site).

Table 5. Mannings *n* values used in the hydraulic flood routing model
(based on the data from Kellerhals, Neill and Bray, 1972)

Location (km)	Mannings *n*
28 to 75	0.030
75 to 210	0.045
210 to 345	0.025
345 to 1107	0.020

The final geometric model consisted of more than 1100 computational nodes describing channel width, effective bed elevation and channel resistance. No consideration of flood plain geometry could be provided at this time, due to the limited field data available.

2.3 AVAILABLE HYDROLOGIC DATA

The National Topographic Series (NTS) 1:250,000 scale maps show more than 80 Peace River tributaries within the study reach. However, only a fraction of these streams are gauged. This means that flood routing models, both hydrologic and hydraulic are constrained by a lack of data. Consequently, it impossible to assess the magnitude of the error in modeling the Peace River in certain reaches, because the difference between modeled and observed stream flows are comprised of both model errors and ungauged (unquantified) lateral inflows.

In this section the available data is discussed, including details of how the lateral inflows were quantified and of the sites along the Peace River for which gauge data was available for comparison to the computed results.

2.3.1 WSC Gauge Data Available on the Peace River Tributaries

For consistency, the tributary inflows used in this *hydraulic* flood routing model were identical to those used by AEP in their *hydrologic* flood routing model. Table 6 presents the tributaries considered in this analysis, the numbers of the WSC gauges from which the data were obtained and the multiplication factor used by AEP to transpose the tributary gauge data downstream to the confluence with the Peace River.

Table 6. Peace River tributaries considered in the flood routing models

Location	WSC	Factor
Halfway River near Farrell Creek	07FA006 (1987)	1.00
Halfway River near Farrell Creek (lower)	07FA001 (1980)	1.00
Moberly River near Fort St. John	07FB008	1.40
Pine River at East Pine	07FB001	1.00
Beaton River near Fort St. John	07FC001	1.03
Kiskatinaw River near Farmington	07FD001	1.26
Clear River near Bear Canyon	07FD009	1.00
Smoky River near Watino	07GJ001	1.02
Heart River near Mampa	07HA003	1.00
Notikewin River at Manning	07HC001	1.39
Boyer River near Fort Vermilion	07JF002	1.00
Ponton River above Boyer River	07JF003	1.26
Wabasca River at Walden Lake Road	07JD002	1.10

Additional tributary inflow data were available from gauges on the Alces River (at the 22nd Baseline), the Saddle River (near Woking), and the Whitemud River (near Dixonville). However, as these data were not used in the AEP hydrologic flood routing model, no multiplication factors were provided to transpose the gauge data downstream to the confluence in a manner consistent with the data from the other tributaries. Therefore, these tributaries were not considered in the hydraulic model simulations.

2.3.2 WSC Gauge Data Available on the Peace River

A number of Peace River streamflow gauges were operational during the 1980 and 1987 runoff events, including: the Peace River at Hudson Hope (which was used as the upstream boundary condition for the flood routing computations since dam outflows are not published) at station 28 km; the Peace River above Pine River at station 120 km; the Peace River near Taylor at station 122 km; the Peace River at Dunvegan Bridge at station 295 km; the Peace River at the town of Peace River at station 395 km; and the Peace River at Peace Point located at station 1107 km. Comparison to the data from these gauges was used to assess the quality of the hydraulic model simulation. Unfortunately, there were no data available at Carcajou (650 km) or Fort Vermilion (808 km) as these gauges were discontinued in 1967 and 1978, respectively.

It is important to note that although portions of the basin drainage area in the downstream reach are not gauged, gauge records between Peace River and Peace Point indicate a net loss of water between these two sites (personal communication: Mr. J. Taggart, AEP, 1994). Because of the ungauged local inflows, the net loss would actually be greater than that indicated by integrating the runoff hydrographs. At this time it is not clear whether this loss reflects a consistent measurement error or a genuine physical process.

3.0 NUMERICAL MODEL

3.1 INTRODUCTION

In this study the cdg-1D hydraulic flood routing model, developed in the Civil Engineering Department at the University of Alberta, was used to model the propagation of flood flows along the Peace River. This model employs a Petrov-Galerkin finite element method known as the characteristic-dissipative-Galerkin scheme (Hicks and Steffler, 1990, 1992) to solve the one-dimensional unsteady open channel flow equations. Details of the equations modelled are provided in section 3.2, while the implementation of the numerical scheme is described in section 3.3.

3.2 EQUATIONS MODELLED

The hydraulic flood routing model was based on the St. Venant equations (Henderson, 1966), which were modified to provide a conservation formulation applicable to rectangular channels of varying width (Hicks and Steffler, 1990):

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad [3.1]$$

$$\frac{\partial Q}{\partial t} + \frac{\partial QU}{\partial x} + \frac{\partial}{\partial x}\left(\frac{gAH}{2}\right) - \frac{gAH}{2B}\frac{dB}{dx} = gA\,(S_o - S_f) \qquad [3.2]$$

where:

A = cross sectional area perpendicular to flow;
Q = discharge;
U = cross sectionally averaged longitudinal velocity;
H = depth of flow;
B = width of rectangular cross section;
S_f = longitudinal boundary friction slope;
S_o = longitudinal channel bed slope;
g = acceleration due to gravity;
t = temporal coordinate; and
x = longitudinal coordinate.

This system of equations describing one-dimensional, unsteady open channel flow may also be written in matrix notation:

$$\frac{\partial\{\phi\}}{\partial t} + \frac{\partial\{F\}}{\partial x} + \{f_c\} = \{0\} \qquad [3.3]$$

where,

$$\{\phi\} \equiv \begin{Bmatrix} A \\ Q \end{Bmatrix} \; ; \; \{F\} \equiv \begin{Bmatrix} Q \\ \left(UQ + \frac{gAH}{2}\right) \end{Bmatrix} \; ; \text{ and, } \; \{f_c\} \equiv \begin{Bmatrix} 0 \\ -gA\left(S_o + \frac{H}{2B}\frac{dB}{dx} - S_f\right) \end{Bmatrix} \qquad [3.4]$$

A non-conservation form of the system may also be considered:

$$\frac{\partial\{\phi\}}{\partial t} + [A]\frac{\partial\{\phi\}}{\partial x} + \{f_n\} = \{0\} \qquad [3.5]$$

where,

$$[A] \equiv \frac{\partial\{F\}}{\partial\{\phi\}} = \begin{bmatrix} 0 & 1 \\ c^2 - U^2 & 2U \end{bmatrix} \qquad [3.6]$$

and,

$$\{f_n\} \equiv \left\{ \begin{matrix} 0 \\ -gA\left(S_o + \frac{H}{B}\frac{dB}{dx} - S_f\right) \end{matrix} \right\} \qquad [3.7]$$

The modified (conservation) formulation of the St. Venant equations has the significant advantage over more conventional (non-conservation) formulations in that it has been shown to be more effective in ensuring conservation of both mass and longitudinal momentum over a broad spectrum of complex flow scenarios (Hicks and Steffler, 1990, 1995).

3.3 NUMERICAL SOLUTION TECHNIQUE: cdg-1D

3.3.1 Background

In this study, the system of equations represented by equation [3.3] were solved using the finite element method. Although many successful hydraulic flood routing models have been developed based on the finite difference method, commercially available finite difference models are based on non-conservation formulations of the governing equations. Furthermore, none of the available models incorporate the effects of ice on the flow. Recent research in the Civil Engineering Department at the University of Alberta has led to the development of a numerically robust finite element model which has already been used to assess the potential impact of ice jam release surges on the Hay River, NWT. Comparisons of this numerical scheme to more conventional, commercially available finite difference code as well as other finite element schemes (Hicks and Steffler, 1990, 1995) have confirmed the superiority of this finite element scheme in terms of both solution accuracy and numerical stability.

3.3.2 Finite Element Implementation

The finite element equations were derived using the Galerkin weighted residual method. The simplest implementation is the Bubnov-Galerkin method (analogous to centered finite differences). In this method the test functions are simply set equal to the basis functions which is analogous to centered differences, that is,

$$\frac{\partial \phi}{\partial x} = \theta \left(\frac{\Phi_{j-1}^{n+1} - \Phi_{j+1}^{n+1}}{2\Delta x} \right) + (1-\theta) \left(\frac{\Phi_{j-1}^{n} - \Phi_{j+1}^{n}}{2\Delta x} \right) \qquad [3.8]$$

where the indices n and j denote the temporal and spatial discretizations, respectively. θ represents the implicitness factor such that $\theta = 1$ represents a fully implicit formulation. Also,

$$\frac{\partial \phi}{\partial t} = \frac{\Phi^{n+1} - \Phi^{n}}{\Delta t} \qquad [3.9]$$

where,

$$\Phi = \frac{\Phi_{j-1} + 4\Phi_{j} + \Phi_{j+1}}{6} \qquad [3.10]$$

3.3.3 Characteristic-Dissipative-Galerkin Scheme

In open channel flow applications, the Bubnov-Galerkin formulation has been shown to be useful for modeling relatively flat waves but it performs poorly in the vicinity of steep gradients in the solution (Katopodes, 1984). An alternative is to use the Petrov-Galerkin method, in which upwind weighted test functions are used to introduce *selective* artificial dissipation, smoothing out spurious, short wavelength oscillations while preserving the physical wave behavior. Essentially, this is equivalent to a Bubnov-Galerkin formulation of the extended system,

$$\underbrace{\left(\frac{\partial\{\phi\}}{\partial t} + \frac{\partial\{F\}}{\partial x} + \{f_c\}\right)}_{\Leftarrow \text{ original system } \Rightarrow} \underbrace{- \omega \frac{\Delta x}{2}[W]\frac{\partial}{\partial x}\left(\frac{\partial\{\phi\}}{\partial t} + [A]\frac{\partial\{\phi\}}{\partial x} + \{f_n\}\right)}_{\Leftarrow \text{ upwinding terms } \Rightarrow} = \{0\} \qquad [3.11]$$

In which ω is an 'upwinding coefficient' or diffusion parameter, while the matrix, $[W]$, controls the distribution of the upwinding. It should be noted that the upwinding terms are formed from derivatives of the non-conservation form of the original system. Artificial dissipation is introduced through the second derivative in x, and is balanced to third order by the other upwinding terms when a semi-implicit formulation is used. This process corresponds to $\theta = 0.5$.

The Petrov-Galerkin formulation employed in the investigation was the characteristic-dissipative-Galerkin (CDG) scheme originally introduced by Brooks and Hughes (1982) as the Streamline Upwind Petrov-Galerkin (SU/PG). In this approach, the numerical diffusion was incorporated using an upwinding term which was determined based upon the sign of the flow direction. Adaptation of this concept to the problem of open channel flow is defined by (Hicks and Steffler, 1990, 1992):

$$[W] \equiv \frac{[A]}{\left|[A]\right|} = [M]\left[\frac{\lambda_{\iota}}{|\lambda_{\iota}|}\right][M]^{-1} = \begin{bmatrix} \frac{1}{2c} & -\frac{1}{2c} \\ \frac{U+c}{2c} & \frac{-(U-c)}{2c} \end{bmatrix} \begin{bmatrix} \frac{U+c}{|U+c|} & 0 \\ 0 & \frac{U-c}{|U-c|} \end{bmatrix} \begin{bmatrix} -(U-c) & 1 \\ -(U+c) & 1 \end{bmatrix} \qquad [3.12]$$

A constant value of 0.25 for the upwinding parameter, ω, minimizes dissipation of long wavelengths while achieving good phase accuracy. Phase accuracy may be optimized by employing a value of $\omega = 0.5$, with slightly increased dissipation. As it has been shown that the effect of varying ω on phase and amplitude is only marginal (Hicks and Steffler, 1990, 1992) a constant value of 0.5 was used in this investigation.

4.0 RESULTS OF THE NUMERICAL SIMULATION

4.1 INTRODUCTION

As stated earlier, two flood events were considered in this preliminary model evaluation. The first was the 1980 spring runoff event and the second was the 1987 summer flood. Input data for each simulation included the geometric data describing the channel as well as lateral inflow (tributary) hydrographs. In addition, two boundary conditions (discharge upstream, and stage downstream) and initial conditions at every computational node (stage and discharge) had to be specified for each event. The gauge site at Hudson Hope was taken as the upstream boundary of the computational domain, with the WSC data from the gauge providing the inflow boundary condition. The model was extended 100 km downstream of Peace Point (assuming a constant width and slope) so as to allow for an estimated stage as the downstream boundary condition. The numerical model was used to calculate the initial conditions for each steady flow test, by calculating a gradually varied flow profile for constant inflow and tributary discharges, based on observed flows on the day the simulation started.

For each event, calculated results were output at select sites, so as to facilitate a comparison to WSC gauge data. In addition, where information was available, these results were compared to the results obtained by AEP using the SSARR model. It is important to note that because mass is not conserved between the Peace River and Peace Point gauges (for the measured data) part of the SSARR model calibration involved withdrawing flow between the two gauges manually. However, for consistency, the SSARR results presented for Peace Point do not include this loss function (since the hydraulic routing model was based only on measured data).

4.2 MODEL RESULTS

4.2.1 1980 Spring Flood Event

The 1980 flood event simulation extended from May 25 to July 15, a period of 52 days. Peak discharge magnitudes were smaller than 1:2 year flood flows. Therefore, this event might be better described as a "small" rather than as a "moderate" flood event. The initial simulation was conducted with a time step increment of 6 hours for a total of 208 time steps. Computational time, using a 486 DX/2 66MHz PC compatible computer was just over 1 hour. The channel roughness values presented Table 5 were used in the initial run, with roughness intended as the only model calibration parameter.

Figure 4 shows the discharge hydrographs obtained from the model, as compared to WSC gauge data and the AEP SSARR model. General agreement with the WSC data measured at these sites was as good or better than the AEP SSARR model, even without calibration of Mannings n in the hydraulic model. The timing of the flood peaks are exact, and peak magnitudes are only slightly lower than measured values from Peace River, upstream. This is consistent with what was expected, given that the values used for channel roughness were based on 2 year flood data and actual discharges were lower in this case. An appropriate increase in channel roughness to account for this effect would refine the model results. At Peace Point, the model overestimates the flood peak and predicts an early arrival. This overestimation of the peak could be due, in part, to the fact that mass is not conserved in the gauge records. However, it is likely that the difference is also due, in part, to the approximate nature of the geometric model between Peace River and Peace Point.

Figure 5 shows the change in calculated stage throughout the simulation period, relative to the initial stage calculated for the steady flow (initial) condition. This type of information is not available from hydrologic flood routing models (such as SSARR). This analysis indicates that maximum stage increases in the order of 2 m occurred during this flood event.

4.2.2 1987 Summer Flood Event

The 1987 flood event simulation extended from July 29 to September 1, a period of 35 days. Peak discharge magnitudes were only slightly higher than 1:2 year flood flows. Therefore, this event might be better described as a "moderate" rather than as an "extreme" flood event. The initial simulation was conducted with a time step increment of 6 hours for a total of 140 time steps. Computational time, again using a 486 DX/2 66MHz PC compatible computer ,was just under 1 hour. The initial channel roughness values presented Table 5 were used in the simulation.

Figure 6 shows the discharge hydrographs obtained from the model, as compared to WSC gauge data and the AEP SSARR model. Again, even without calibration of the roughness parameter, agreement with the WSC measured data is good. In this case, with event discharges more comparable with 2 year flood flows, the calculated peak magnitudes are much closer to the measured flows even at Peace Point. Again, results would be refined if adequate geometric data were available between Peace River and Peace Point. Figure 7 shows the change in calculated stage throughout the simulation period. Maximum stage increases in the order of 4 to 5 m were indicated in this case.

4.3 DISCUSSION OF RESULTS

These two flood routing simulations illustrate the validity of the underlying hydraulic model for diffusive wave scenarios. Based on this preliminary analysis, further refinement through calibration of channel roughnesses was not deemed warranted, as results for moderate flood flows (in the order of 1:2 year flood events) are in good agreement with measured data. However, model reliability could be enhanced if additional geometric data were to be collected between Peace River and Peace Point. At that time, further calibration could be conducted to refine the model for small and extreme flood events.

4.4 PROPOSED FIELD SURVEYS

Figurte 8 illustrates the location of the existing surveys at the towns of Peace River and Fort Vermilion, as well as at Peace Point. These surveys, and water surface slopes from the NTS maps, indicate one or more breaks in bed slope between these two sites. Identifying the location of the break(s) is important to the refinement of the hydraulic model. The break or breaks in bed slope, can likely be found based on geomorphological changes. About 13 to 15 km upstream of the Whitemud River confluence (measured along the river channel), the river changes from a relatively straight channel with few islands, to an irregularly meandering channel with occasional islands. A few sections upstream and downstream of this change would capture any slope break. (5 or 6 cross sections total, over a 15 to 20 km distance). A more detailed profile through the same reach would provide as good, or better, information. Another geomorphological change occurs near Carcajou. Here, the river changes by becoming more irregular with an almost continuous succession of islands. Again, a few sections upstream and downstream of this change should capture any slope break. A detailed profile through the same (or extended) distance would provide excellent additional information.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The primary objective of this study was to develop and verify a hydraulic flood routing model for the Peace River from Hudson Hope, B.C. to Peace Point, Alberta. Although this hydraulic model is capable of handling highly dynamic flood events (such as dam break floods and surges resulting from ice jam releases), the test scenarios prescribed by the terms of reference for this preliminary study were simpler "diffusive" waves because only diffusive wave data is available for model verification. Two events were simulated in this investigation, specifically: the 1980 spring runoff event and the 1987 summer flood event.

A major component of this project involved the development of a geometric model of the study reach using available cross section surveys supplemented with N.T.S. map data. The final geometric model consisted of more than 1100 computational nodes describing channel width, effective bed elevation and channel roughness. The dearth of surveyed river geometry in the reach between the town of Peace River and Peace Point is seen as the main source of unreliability in the geometric database.

The only calibration parameter involved in the development of the hydraulic model was the channel resistance coefficient, specifically Mannings *n*. Initial values for the parameter were based on the data provided by Kellerhals, Neill and Bray (1972) for 1:2 year flood events at gauge sites. Agreement between measured (WSC) and computed flood hydrographs was good for both flood events in the reach from Hudson Hope to the town of Peace River. Differences between measured and computed hydrographs were slightly larger at Peace Point. However, given the fact that mass is not conserved between the gauges at the town of Peace River and at Peace Point (necessitating the inclusion of a loss function in the AEP SSARR model), and the paucity of surveyed channel geometry between these two sites, no further refinement, or "calibration" of the model was considered warranted at this time.

Based on these investigations it is concluded that the hydraulic flood routing model adequately predicts flood hydrographs for moderate flood events, though the collection of additional field data between the town of Peace River would enhance the reliability of the model downstream of the town of Peace River. It is stressed that because of the limited data available (both in terms of these unsurveyed reaches and the lack of overbank geometry) the model is still somewhat empirical.

5.0 REFERENCES

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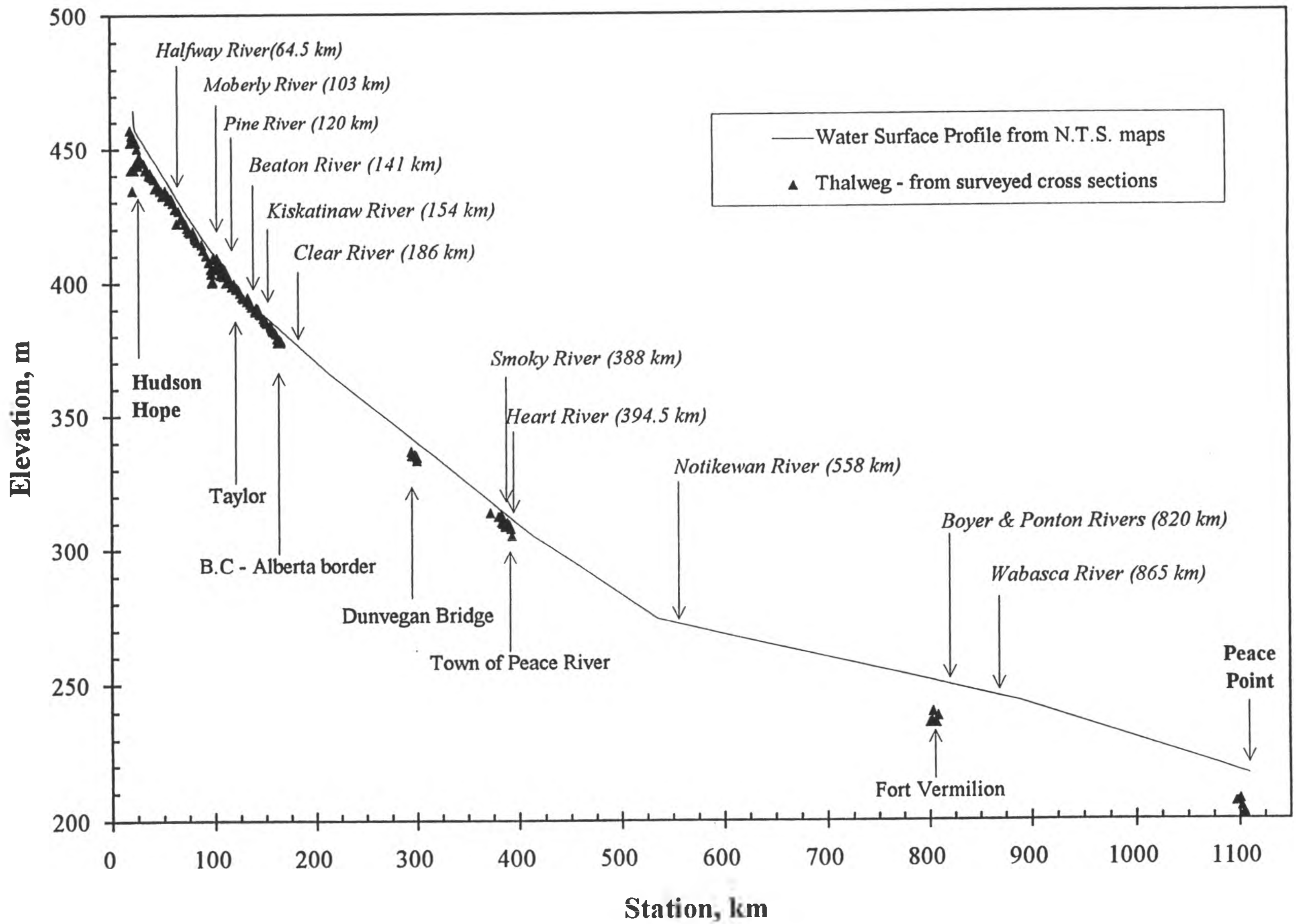


Figure 1. Profile of the Peace River based on field surveys and N.T.S. maps

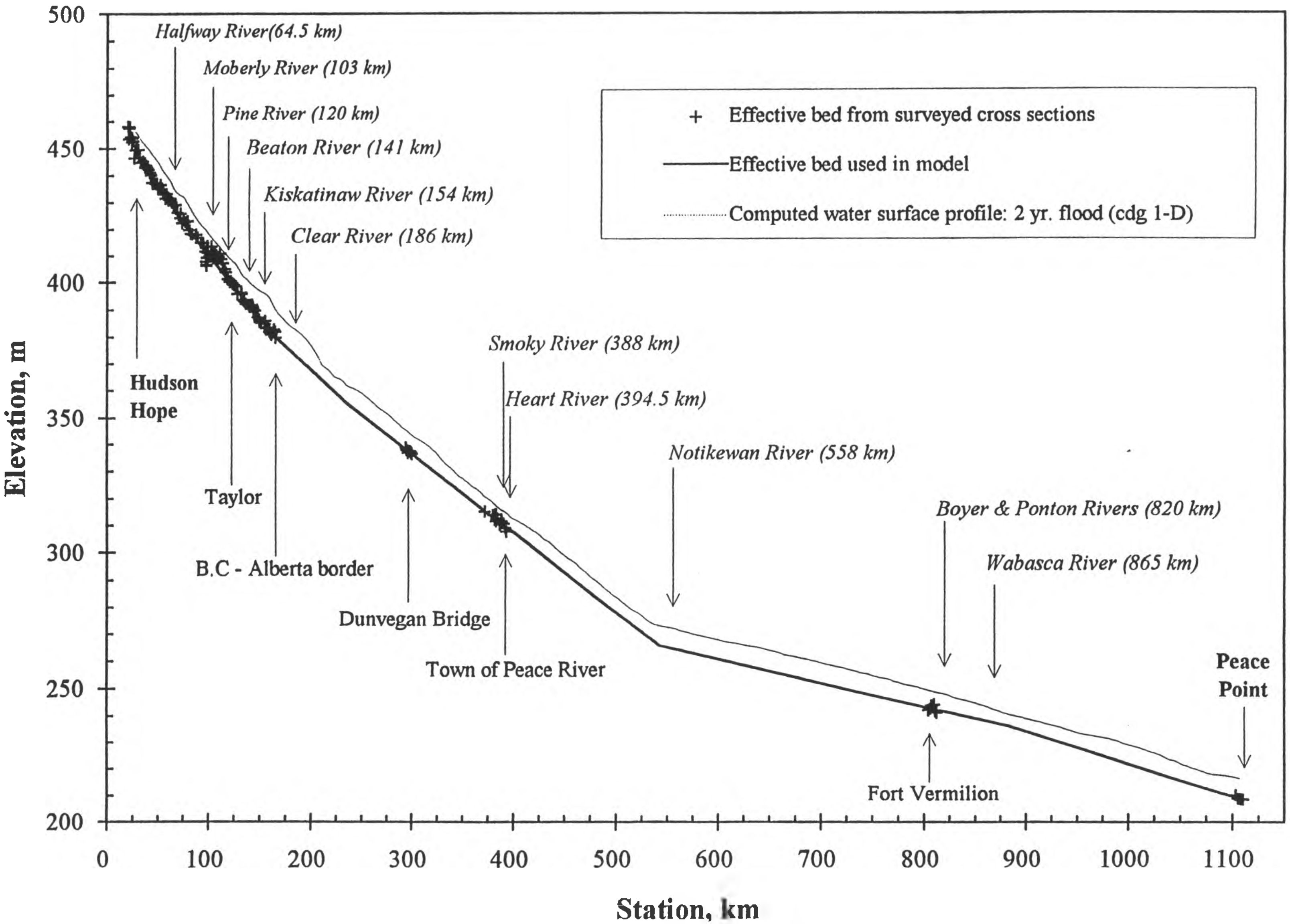


Figure 2. Effective bed profile used in the hydraulic flood routing model

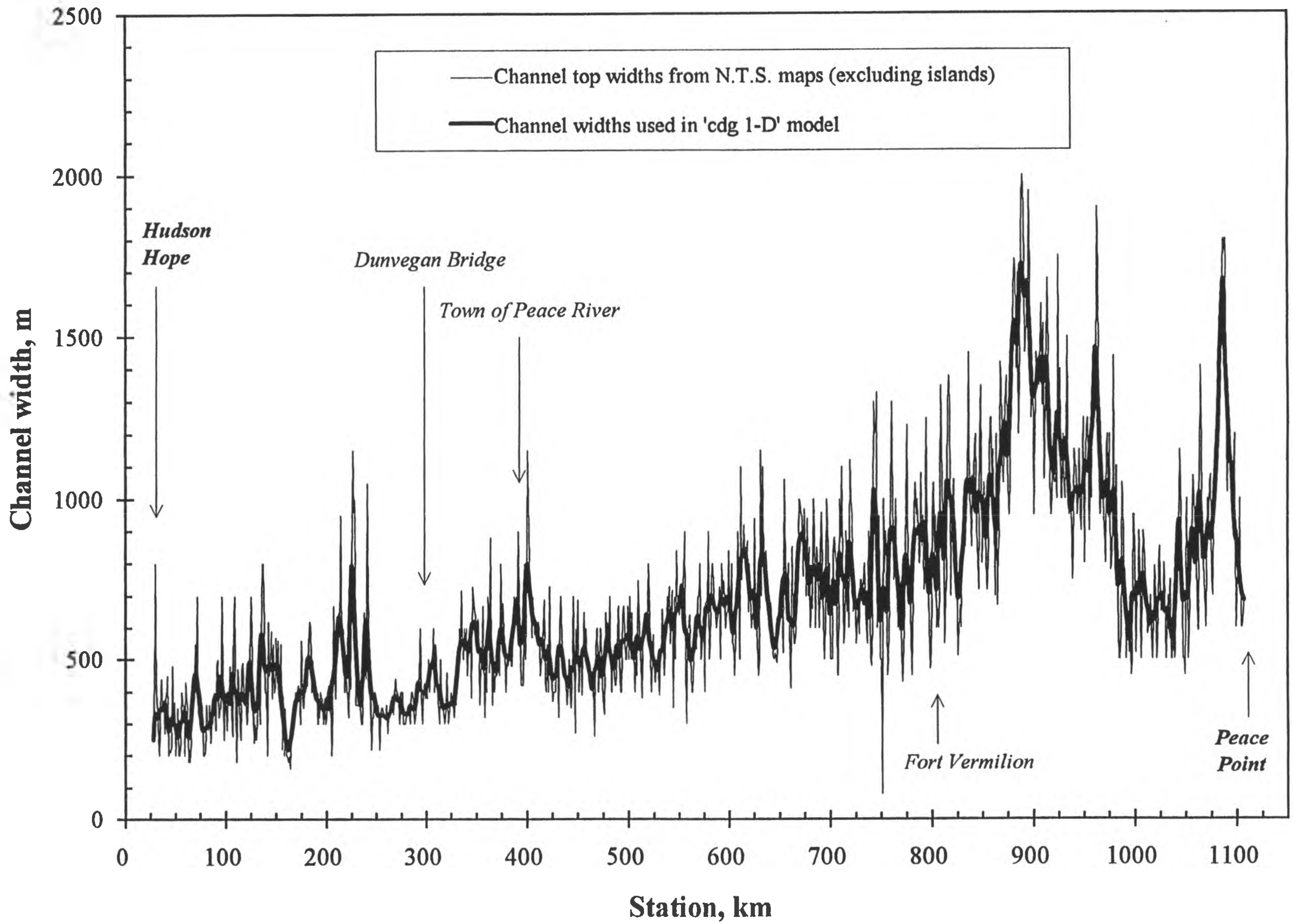


Figure 3. Channel top widths obtained from the N.T.S. maps

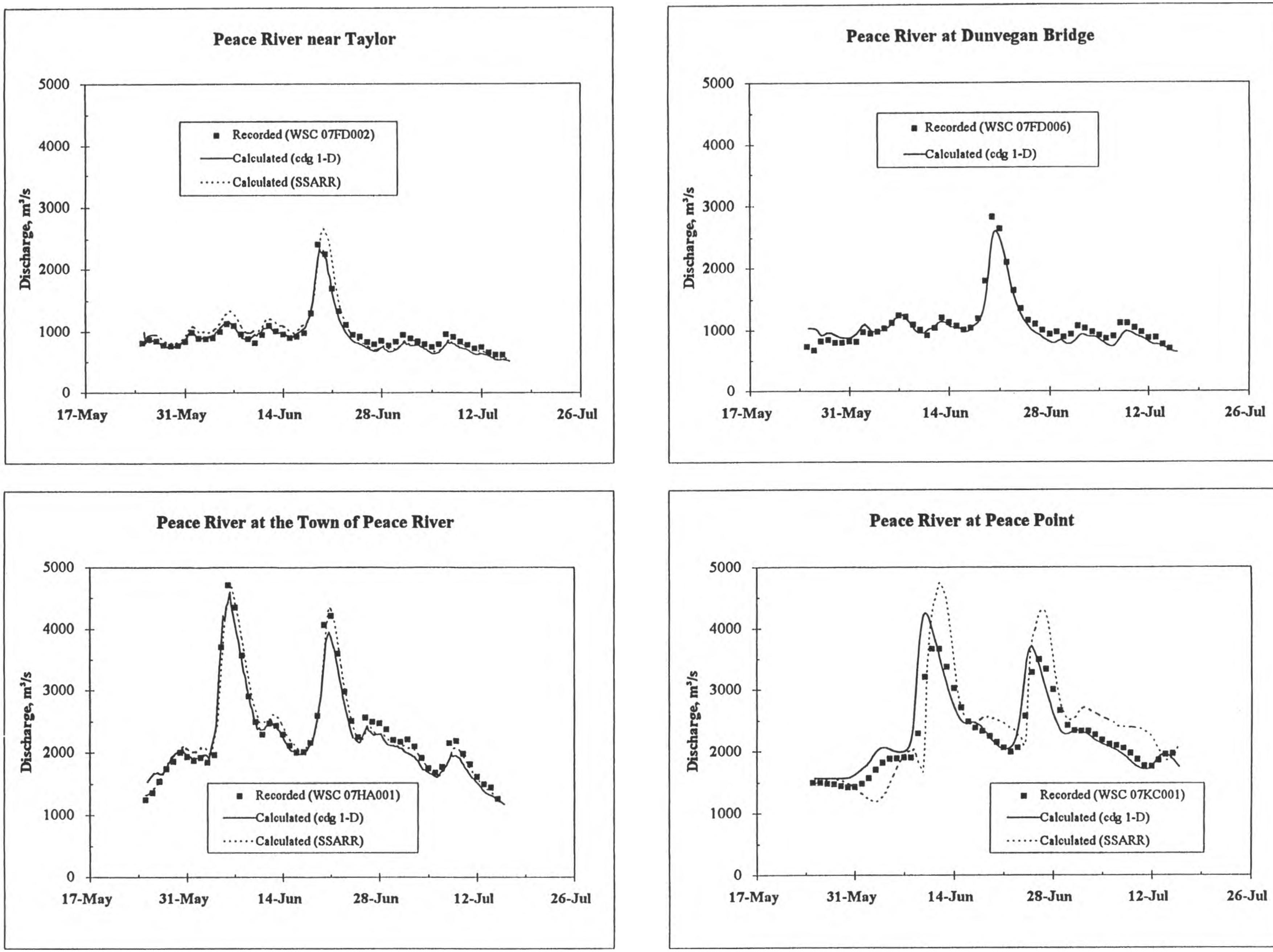
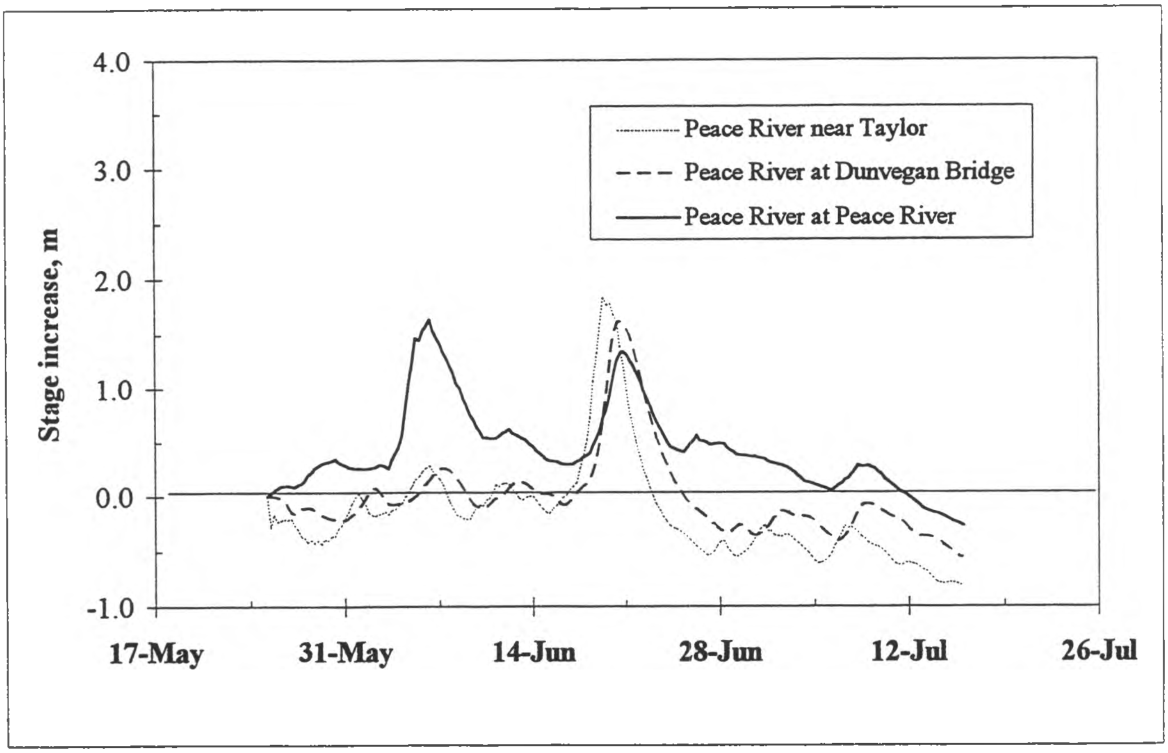


Figure 4. Comparisons of measured and computed discharge hydrographs (1980 event)



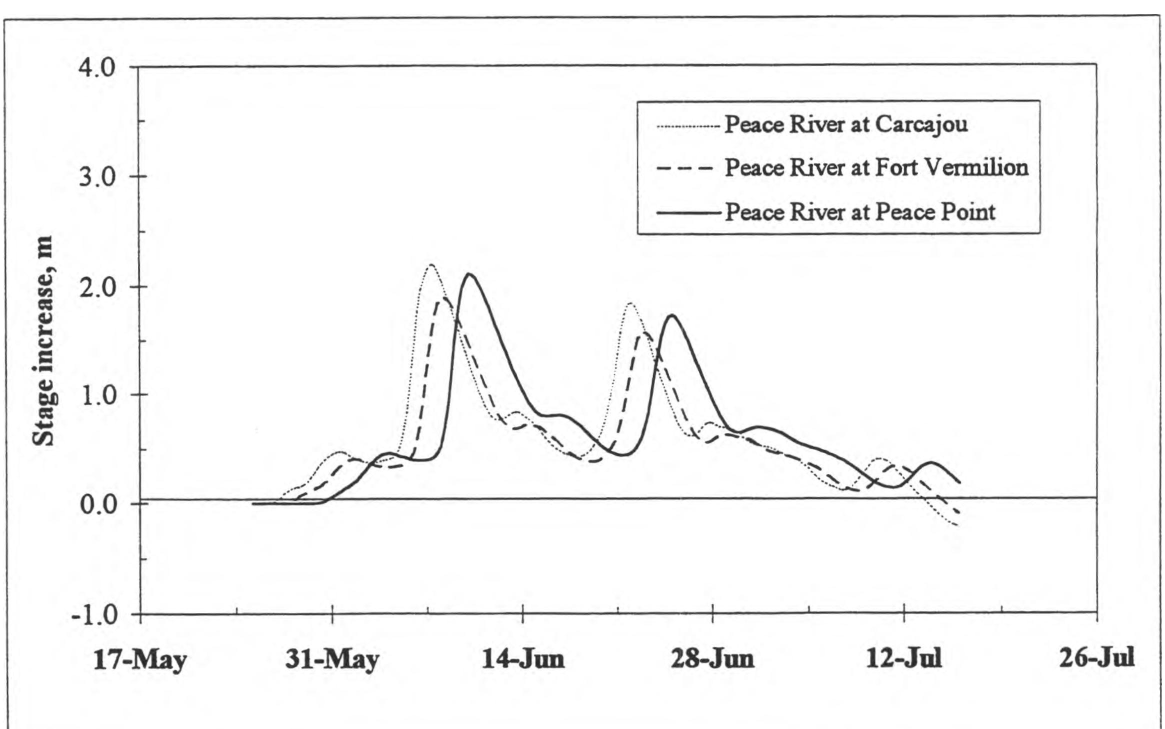


Figure 5. Computed stage increases (1980 event)

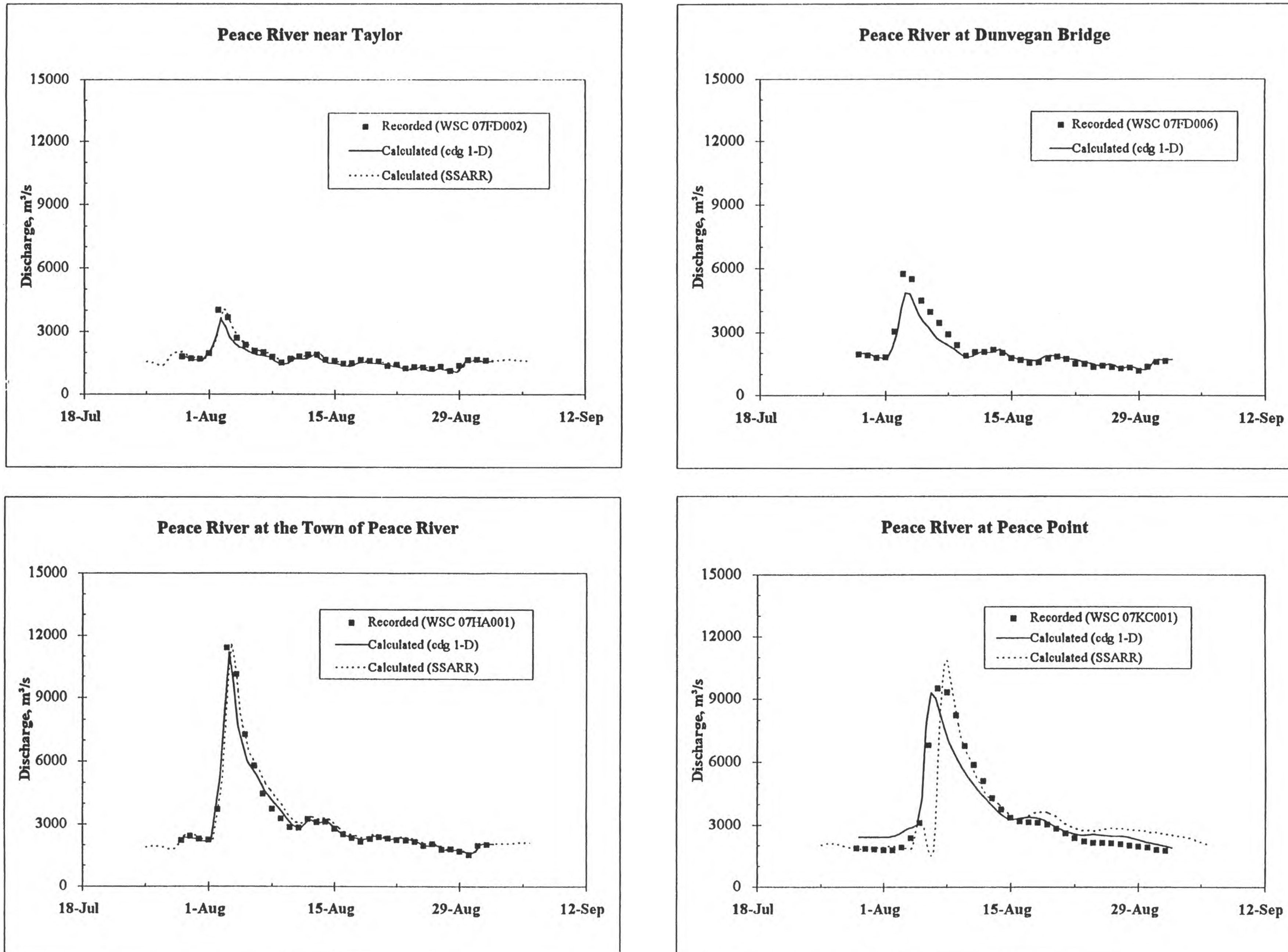
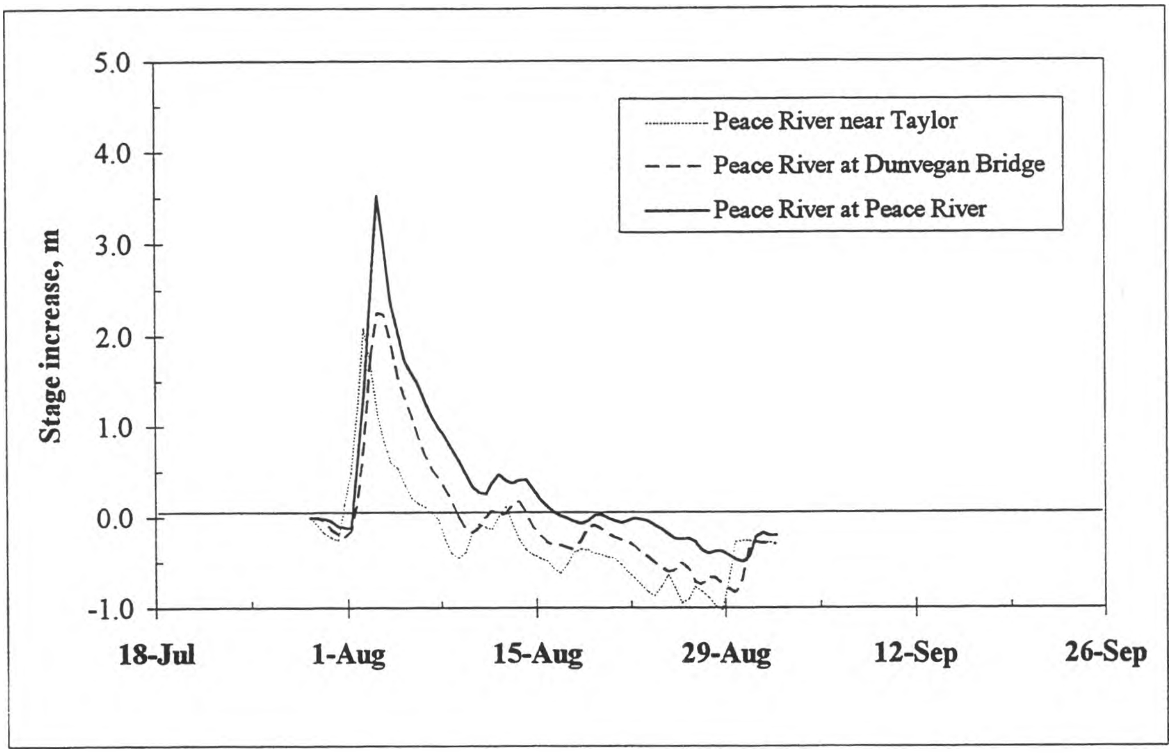


Figure 6. Comparisons of measured and computed discharge hydrographs (1987 event)



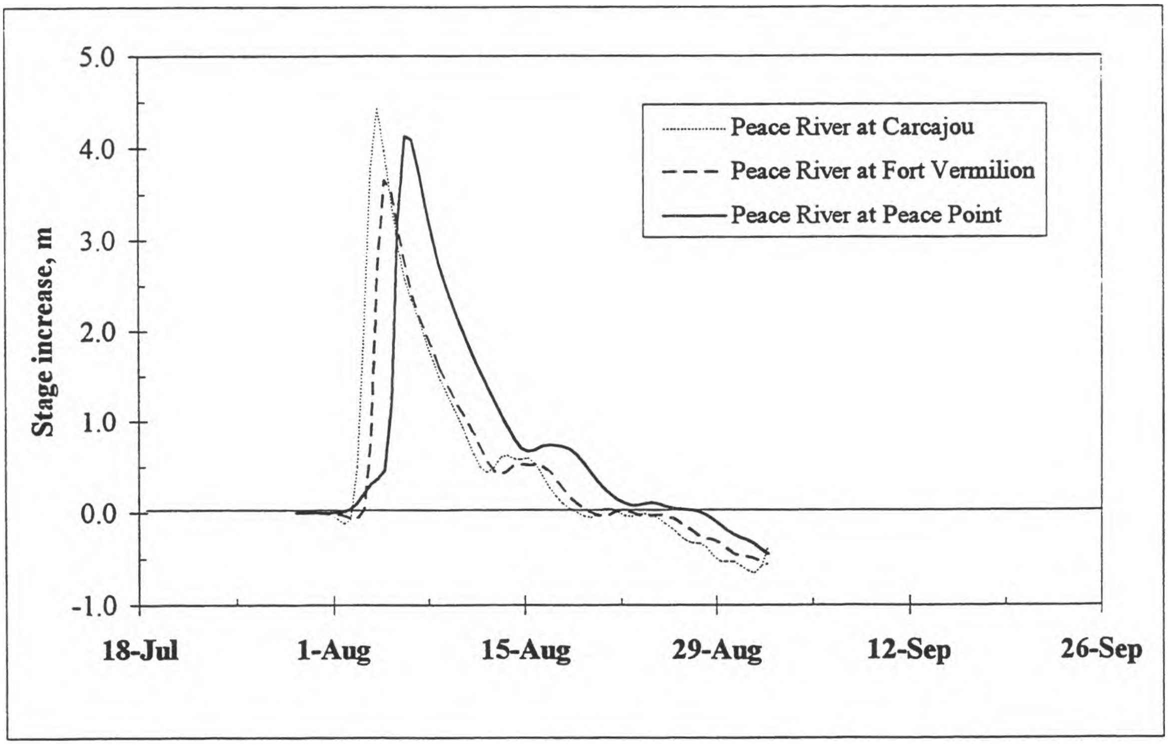


Figure 7. Computed stage increases (1987 event)

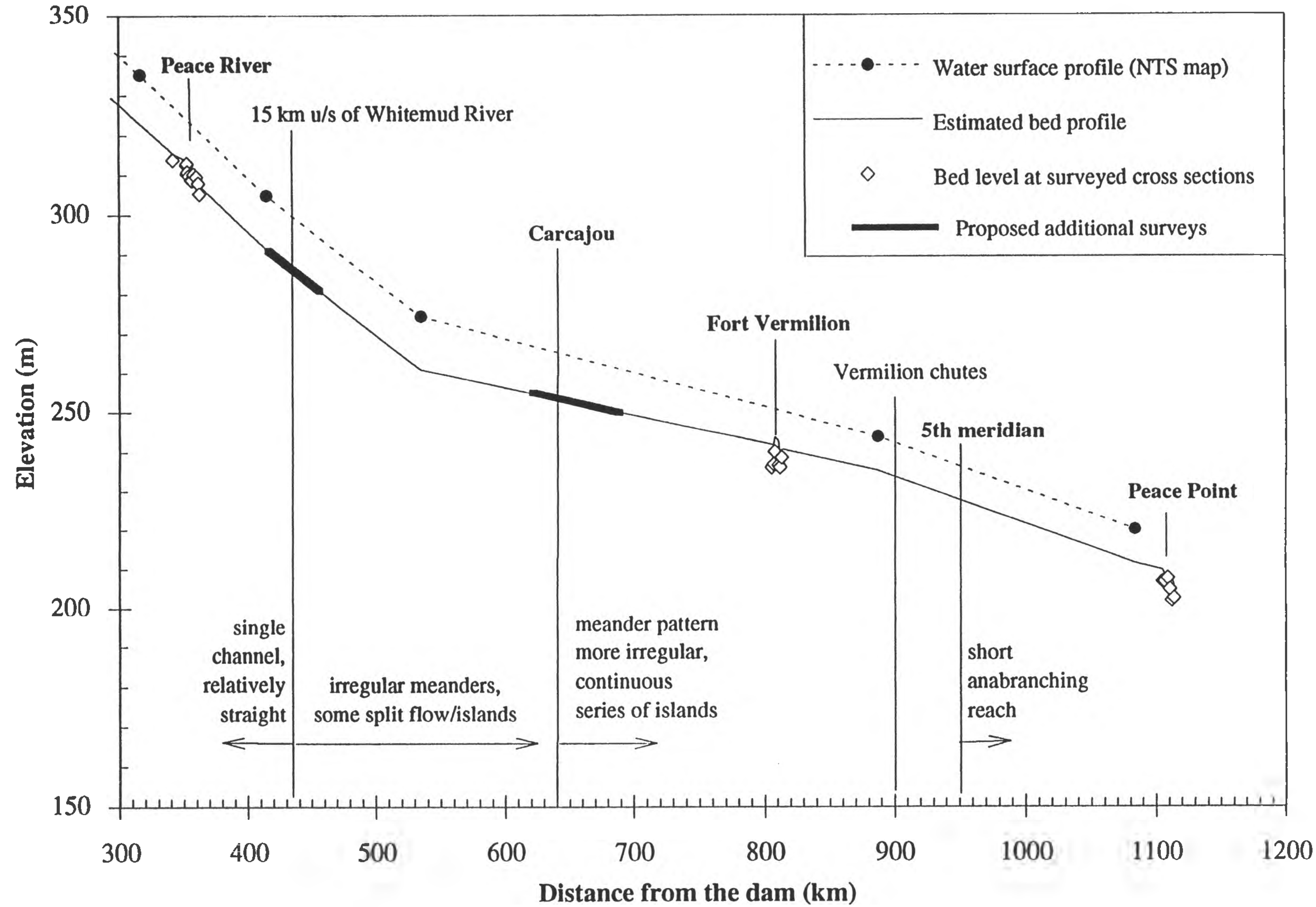


Figure 8. Locations of proposed surveys on the Peace River in Alberta

APPENDIX A

TERMS OF REFERENCE

NORTHERN RIVER BASINS STUDY

ASSIGNMENT NO. 6 - TERMS OF REFERENCE

SCHEDULE A

Project 1154-C1: Peace River Flow Analysis

I. Objective

The Northern River Basins Study seeks to assess the effects of flow regulation by the Bennett Dam on the Peace River under ice covered conditions, and the impact of this flow regulation on the geomorphologic evolution of the river. An assessment of the effects of regulation has been successfully conducted by Alberta Environment for open water conditions using standard hydrologic flood routing techniques. However, hydrologic flood routing techniques neglect the effects of momentum conservation on the propagation of flood waves and, therefore, cannot assess the effects of an ice cover or sediment transport processes on the hydraulics of flow. Such an analysis requires the use of a hydraulic flood routing technique.

Many successful hydraulic flood routing models have been developed though none which incorporate the effects of ice on the flow are currently available commercially. Furthermore, unlike conventional flood routing problems which are diffusive by nature, consideration of the impact of ice jam surge releases on flood flows represents a very dynamic problem, which presents particular difficulty in terms of obtaining numerical solutions. Recent research in the Civil Engineering Department at the University of Alberta has led to the development of a numerically robust unsteady, open channel flow model which has already been used to assess the potential impact of ice jam release surges on the Hay River, NWT. This model employs a Petrov-Galerkin finite element method known as the Characteristic-Dissipative-Galerkin (CDG) scheme. Comparisons of this numerical scheme to more conventional, commercially available code have been conducted, confirming the superiority of the CDG scheme in terms of both solution accuracy and numerical stability.

The objective of this study would be to develop a hydraulic model of the Peace River, capable of routing floods between the Bennett Dam in British Columbia and Peace Point in Wood Buffalo National Park, Alberta. Specifically, the model would be capable of assessing the effects of ice on the propagation of both natural and regulated flows through the Peace River.

II. Requirements

This project would begin with the development of a geometric data base describing the study reach. As detailed surveyed geometry is only available at a few sites, intermediate cross sections will be synthesized based on topographic contour mapping and the existing survey data.

Following assembly of the geometric data base, preliminary model verification would be conducted for open water conditions. Channel roughness would be the only calibration variable and results would be verified based on hydrometric records and the original open water analysis conducted by Alberta Environment.

As a minimum, the flood routing analysis would be conducted for both a medium and an extreme flood event under four scenarios: regulated and unregulated inflows under open water and simple ice covered conditions. Results of the open water analyses would be compared with the hydrologic routing results.

III. Project Organization

This project would commence October 1, 1993 and continue for a period of 4 months. The research would be conducted by the research assistant, Ms. N. Yasmin, working under the direct supervision of the principal investigator, Dr. F. Hicks (C.V. attached).

Both the principal investigator and the research assistant would interact with Mr. J. Taggart and Mr. J. Choles of Alberta Environment in obtaining and assembling the existing cross section and streamflow data.

Because of the exploratory nature of the research, the specific range of test scenarios may require modification. This would be coordinated with Dr. Terry Prowse, Hydraulics/Hydrology/Sediment Project Leader, NRBS and Mr. J. Taggart of Alberta Environment.

IV. Reporting Requirements

The results will be reported in conformance with NRBS standards.

V. Cost Estimate

Budget Category	Cost
F. Hicks (3 days @ $260/d)	$780.00
N. Yasmin (4 months @ $2000/m)	$8,000.00
Benefits	$439.00
Supplies, report production, photocopying, etc.	$380.00
Research overhead @ 25% (NRBS negotiated rate)	$2,399.75
Total	$11,998.75

NOTE: No amount for the purchase or collection of field data is anticipated or allowed for in the budget.