PONOKA FLOOD RISK MAPPING STUDY

Submitted to:

ALBERTA ENVIRONMENTAL PROTECTION
RIVER ENGINEERING BRANCH

For

CANADA-ALBERTA
FLOOD DAMAGE REDUCTION PROGRAM

Submitted by:

HYDROTECH CONSULTING LTD.
and
B.K. HYDROLOGY SERVICE

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EXECUTIVE SUMMARY

The Ponoka Flood Risk Mapping Study has been conducted as part of the Canada-Alberta Flood Damage Reduction Program. The primary purpose of this study was to prepare flood risk and flood frequency maps for a reach of the Battle River flowing through the Town of Ponoka.

Background data for this study included flood frequency estimates, base mapping, channel and floodplain cross section surveys, and highwater marks from recent major floods. Based on recorded flow data, large floods at Ponoka occur during open-water periods. Water surface profiles for various flood events were computed using the U.S. Corps of Engineers HEC-2 computer model. The model was calibrated using highwater marks surveyed during the 1982 and 1990 floods.

Model results were provided in a tabular form listing flood levels, in figures showing water profiles along the study reach, and on two maps indicating the extent of flooding. The maps, using a 1:5000 scale orthophoto as a base, are of specific interest to the general public. The use of the orthophoto as a base to the map allows for quick and easy recognition of flood prone areas. The flood frequency map shows the extent of flooding for the 1:10, 1:50, and 1:100 year floods. The flood risk map shows the floodway and flood fringe. New development is discouraged within the floodway but may be permitted in the flood fringe with adequate flood proofing.

The results of the study show that most of the present landuse within the flood risk area is pasture and natural vegetation cover. Only a limited area, which is presently developed, is affected by the 1:100 year design flood. Most of this affected development is within the flood fringe and could be protected by flood proofing.

ACKNOWLEDGEMENTS

This study is dedicated to Mr. Jacques Penel, Principal, Hydrotech Consulting Ltd. of Edmonton. Jacques had completed most of this study when he was diagnosed with cancer in March 1992. Within a few weeks of receiving this diagnosis, he passed away. During these few weeks, Jacques made a supreme effort to complete the study and was able to produce a first draft report.

Mr. Terry Winhold of the River Engineering Branch of Alberta Environmental Protection managed the study, assisted by Mr. Brian Yaremko and Mr. Bryce Haimila who provided technical support and produced the final study maps.

Dr. Faye Hicks, Associate Professor of the Civil Engineering Department at the University of Alberta, provided advise on the calibration of the HEC-2 model for open water conditions.

Mr. Bernie Kallenbach of B.K. Hydrology Service prepared the final report based on Jacques draft report and study material.

Mrs. Liesl Mitchell of Think Design and Drafting prepared the figures for the report.

Water Survey of Canada provided gauge data and flow measurements on the Battle River at Ponoka.

Alberta Transportation provided data on the bridges in Ponoka crossing the Battle River.

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1.0 INTRODUCTION

1.1 Flood Damage Reduction Program

In April 1989, the Federal Government of Canada and the Provincial Government of Alberta signed "An Agreement Respecting Flood Damage Reduction and Flood Risk Mapping in Alberta." This agreement initiated the Canada-Alberta Flood Damage Reduction Program. This program is intended to reduce flood damage by using non-structural methods. To achieve this goal, urban areas that are subject to flooding are identified and mapped. This information allows for the planning of various alternatives to reduce the potential for flood damage. These alternatives may include such methods as land use planning, zoning bylaws, floodproofing and flood preparedness.

The Canada-Alberta Flood Damage Reduction Program includes the following components:

1. Identify, map and designate flood risk areas in urban communities across the province.

2. Increase awareness of flood risk among the general public, industry and government agencies through a public information program.

3. Regulate new development in flood risk areas through the use of new federal and provincial government policies.

4. Encourage municipalities to develop zoning bylaws recognizing the designated flood risk areas.

As part of the Canada-Alberta Flood Damage Reduction Program, the River Engineering Branch of Alberta Environmental Protection commissioned Hydrotech Consulting Ltd. to undertake the Ponoka Flood Risk Mapping Study.

1.2 Study Objectives

The purpose of this study is to produce flood risk and flood frequency maps for the reach of the Battle River through the Town of Ponoka. The specific objectives of this study are:

1. Conduct a review of the history of flooding in the Town of Ponoka.

2. Conduct a hydraulic analysis to calculate open water flood levels for various return period floods and delineate the extent of flooding on a flood frequency map.

3. Determine flood fringe and floodway limits for the 1:100 year flood event and delineate results on a flood risk map.

It is intended that these flood risk maps will be used by town planners, government agencies and the public for planning future floodplain management and development.

1.3 Study Area

The Town of Ponoka is located about 100 km south of Edmonton on Highway 2A. The study reach on the Battle River starts at the Highway 2A bridge and extends downstream to the vicinity of the Town of Ponoka sewage lagoons. This river reach is about 8100 m in length and extends about 1000 m upstream and 2100 m downstream of the town's corporate limits. An overview of the study area is provided in Figure 1.

2.0 HISTORY OF FLOODING

2.1 General

The Water Survey of Canada (WSC) hydrometric station, *Battle River near Ponoka* (Station No. 05FA001) has monitored river flow over two extended periods. The gauge was first established in 1913 and operated till 1932. The gauge was reestablished in 1966 and has remained in operation to the present time. During most of the time the gauge was in operation, only mean daily flows were recorded. Annual maximum instantaneous discharges are only available for 14 years.

Since 1902, several large flood events have also occurred when the Water Survey of Canada gauge 05FA001 was not operation. During some of these floods, highwater marks were recorded which have been used to estimate flood discharges. These flood estimates will be discussed further in Section 5.6.

2.2 Historic Floods

The largest documented flood events on the Battle River at Ponoka are listed in Table 1. This includes two flood events in 1902 and 1948 when the WSC gauge was not in operation. Besides these documented floods, several other flood events at Ponoka have been noted in various historical records. These include the following events:

1. In 1908, a bridge was washed out.

2. In 1917, highwater eroded part of the bank of an approach road.

3. In 1920, highwater overtopped an approach grade.

4. In 1927, high spring ice flow required dynamite.

5. In 1954, the highwater level was at the approximate elevation of the south abutment bridge footing on the Highway 2A bridge.

Unfortunately, since no well-defined highwater marks were recorded during these floods, it is not possible to estimate discharges for these events.

TABLE 1
Battle River at Ponoka - Largest Documented Floods

Annual Maximum Instantaneous Discharge (m^3/s)	Date	Status
324	1902	Estimated from highwater mark
287	July 4, 1990	Recorded by WSC
232	1948	Estimated from highwater mark
108	April 19, 1974	Recorded by WSC
98.9	May 9, 1920	Estimated using daily discharge
90.7	April 24, 1982	Recorded by WSC

2.3 Recent Floods

A review of the floods in Table 1 indicates that three of the largest flood events have occurred in the past 20 years. The July 4, 1990 flood is the largest recorded flood by Water Survey of Canada and the second largest documented flood. Documentation of this flood includes surveyed highwater marks and photo of the flooding in Ponoka during the flood peak. A selection of these flood photos is provided in Appendix A.

A variety of documentation is also available for the 1982 and 1974 floods. Readily available data include surveyed highwater marks at several locations within the Town of Ponoka.

2.4 Floods Levels under Ice Cover Conditions

During the period the Water Survey of Canada gauge was in operation, flood levels under ice conditions have not exceeded the 1:5 year flood level for open water conditions. Larger floods (greater than the 1:5 year flood) along this reach of river have occurred during open water conditions. A more extensive review of water levels under ice conditions is provided in Appendix C.

A review of the historical data also indicates that ice jams have not typically been a problem along this reach of the Battle River. The only record of ice jam problems was in 1927, as noted in Section 2.2, when dynamite was used on high spring ice flows.

Based on the above observations, it appears that larger flood events and corresponding highwater levels along this reach of the Battle River typically occur during open water periods. Therefore, calculations of the 1:100 year flood in subsequent sections of this report are based on open water conditions.

3.0 AVAILABLE DATA

3.1 Hydrology Report

A hydrology report, *Flood Frequency Analysis - Ponoka Floodplain Study*, was prepared by the Hydrology Branch of Alberta Environment in 1992. The results of this study are summarized below.

The data base for the hydrology study included the recorded flow at the Water Survey of Canada station 05FA001 for the period of 1913 to 1930, 1967, and 1969 to 1990. Flood flow estimates based on recorded highwater marks were also used for the 1902 and 1948 floods (See Section 5.6).

Annual maximum instantaneous discharges were not available for all years and were estimated using the following procedure. A least square fit curve was computed between maximum daily and maximum instantaneous discharge based on 14 years where both parameters are available. This curve was then used to estimate the missing annual maximum instantaneous discharges for years where the maximum daily discharges were available.

Flood frequency estimates were derived using the annual maximum instantaneous flows as described above. The results of this analysis are listed in Table 2.

TABLE 2
Battle River at Ponoka - Flood Frequency Estimates

Return Period (years)	Maximum Instantaneous Discharge (m^3/s)
2	31
5	83
10	137
20	207
50	331
100	452

3.2 Base Mapping and River Cross Section Surveys

Base mapping for this study was provided by Alberta Environmental Protection. The mapping consists of 1:5000 scale orthophoto maps produced from October 1989 aerial photography. Contours are provided at one meter intervals and elevations are provided in meters above sea level, North American Datum, 1927.

A total of 27 cross section surveys were also provided by Alberta Environmental Protection. W.S. Barlow Surveying Ltd. of Spruce Grove conducted the ground survey of the channel cross sections. Extension of these cross sections on the floodplain was done by Western Photogrammetry Ltd. of Edmonton using photogrammetric techniques.

Based on a review of the cross section data by Hydrotech Consulting Ltd., it was decided to generate 10 additional cross sections to improve the definition of the river channel upstream and downstream of the bridges. The higher elevation portions of these cross sections were generated using contour information from the 1:5000 scale orthophoto maps. The channel bottom for these cross sections was obtained by interpolation between nearby surveyed cross sections. The cross section number and location are shown on the Flood Risk Map and the Flood Frequency Map provided at the end of this report.

3.3 Highwater Marks

Surveyed highwater mark information is available for three recent flood events. These floods are:

1. April 19, 1974 flood with a discharge of 108 m^3/s

2. April 24, 1982 flood with a discharge of 90.9 m^3/s

3. July 4, 1990 flood with a discharge of 287 m^3/s

A review of the 1974 flood indicated that the flood documentation was incomplete. Since the flow for the 1974 flood is similar to the 1982 flood, it was decided that the more recent highwater mark data would be used for model calibration. Table 3 and Table 4 list the highwater mark survey data for the 1982 and 1990 flood events.

TABLE 3

Battle River at Ponoka - Highwater Mark Survey for April 24, 1982 Flood

General Location	Highwater Mark Reference	Corresponding Cross Section	Station (m)	Highwater Mark Elevation (m)
70 m u/s of section 2, near sewage lagoon	Bat 82.8	u/s of 2	576	801.32
160 m d/s of section 6	Bat 82.7	d/s of 6	1995	801.65
at lift Station A, left bank	Bat 82.6	8	2906	801.86
d/s side of 50 Ave. bridge, left approach	Bat 82.5	10.2	4109	802.08
u/s side of 50 Ave. bridge, left approach		10.3	4129	802.23
dam site, u/s of 50 Ave. bridge		10.5	4319	802.15
d/s side of pedway bridge	Bat 82.4	13.2	5392	802.4
u/s side of pedway bridge		13.3	5401	802.5
2 m d/s Highway 53 bridge	Bat 82.3	14.2	5705	802.54
2 m u/s Highway 53 bridge		14.3	5725	802.61
78 m d/s of CPR bridge	Bat 82.2	d/s of 17.1	6374	802.88
40 m d/s of CPR bridge		d/s of 17.1	6412	802.91
1 m d/s of CPR bridge		17.2	6452	803.05
2 m u/s of CPR bridge		u/s of 17.3	6470	803.04
20 m u/s of CPR bridge		17.4	6488	802.99
45 m u/s of CPR bridge		u/s of 17.4	6513	803.28
156 m d/s of Hwy. 2A bridge	Bat 82.1	20	7893	803.96
30 m d/s of Highway 2A bridge		u/s of 21.1	8019	804.07
d/s side of Hwy. 2A bridge		21.2	8049	804.11
u/s side of Hwy. 2A bridge, left bank		21.3	8067	804.12
u/s side of Hwy. 2A bridge, at fenceline		21.4	8087	804.15
65 m u/s of Hwy. 2A bridge		u/s of 21.4	8132	804.16

TABLE 4
Battle River at Ponoka - Highwater Mark Survey for July 4, 1990 Flood

General Location	Highwater Mark Reference	Corresponding Cross Section	Station (m)	Highwater Mark Elevation (m)
70 m u/s of section 2, near sewage lagoon	90-BR-15	u/s of 2	576	802.195
150 m d/s of section 6	90-BR-14	d/s of 6	1995	802.928
at lift station A	90-BR-13	8	2906	803.174
d/s of 50 Ave. bridge, left bank	90-BR-12	10.2	4109	803.451
3 m d/s of Hwy. 53 bridge	90-BR-11	14.2	5706	803.718
2 m u/s of Hwy. 53 bridge		14.3	5725	803.708
30 m d/s of CPR bridge	90-BR-10	17.1	6422	804.399
25 m u/s of CPR bridge		17.4	6488	804.658
50 m d/s of Hwy. 2A bridge, left bank	90-BR-9	21.1	7999	805.780
1 m d/s of Hwy. 2A bridge, left bank		21.2	8049	805.754
1 m u/s of Hwy. 2A bridge, left bank		21.3	8067	806.088

In addition to the recent flood highwater mark data, two historical surveyed highwater marks are available. These historical highwater marks are:

1. For the 1902 flood, a highwater mark of 804.03 m near the existing Highway 53 bridge.

2. For the 1948 flood, a highwater mark of 803.48 m near the existing Highway 53 bridge.

These two historical highwater marks are used to estimate the discharges for the 1902 and 1948 flood events (See Section 5.6).

3.4 Rating Curve

As indicated previously, the Water Survey of Canada hydrometric station, *Battle River near Ponoka* (Station No. 05FA001) has monitored river flow over two extended periods. The gauge was first established in 1913 and operated till 1932. The gauge was reestablished in 1966 and has remained in operation to the present time. During these periods, the gauge has been moved several times. A summary of these moves are provided in Table 5.

TABLE 5
Gauge History for the WSC Station Battle River near Ponoka (05FA001)

Date	Legal Description	Geodetic Datum (m)	Gauge Location
May 7, 1913	SW4-43-25-W4M	None	First established gauge
April 7, 1922		None	Moved 0.12 km u/s of first site
April 1, 1923		None	Moved to opposite side of river
May 1, 1925	SE9-43-25-W4M	None	Moved 0.8 km d/s of previous site
June 16, 1966	SE36-42-26-W4M	None	Gauge reestablished about 3 km u/s of previous site
August 7, 1976	NE31-42-25-W4M	799.707	Gauge reestablished about 24 m d/s of CPR bridge
October 1, 1990		798.707	moved 0.1 km d/s of previous site

Of specific interest to this study is the gauge record covering the period of 1976 to 1990. During this time period, the gauge data is related to geodetic elevation and it includes the 1982 and 1990 flood events. The gauge was located downstream of the CPR bridge which corresponds to about cross section 17.1 used for the HEC-2 modelling described in Section 5. The rating curve for the gauge during this period is shown in Figure 2 (WSC Table No. 10, August 13, 1990). The curve extends to a flow of 303 m^3/s which corresponds to about a 1:40 year flood event.

3.5 Flood Photography

Aerial photography of the July 4, 1990 flood at Ponoka includes:

1. Oblique photography taken around 11:00 A.M.

2. 1:5000 scale stereo photographs taken between 9:53 and 9:54 A.M. by Geographic Air Survey Ltd. of Edmonton (Line 7-SW, Photos AS4018-187 to 199).

A selection of the oblique aerial photographs is provided in Appendix A.

Based on flow records from the Water Survey of Canada gauge 05FA001, the flow on July 4, 1990 at 9:54 A.M. was about 200 m^3/s. On July 4, 1990 at 11:00 A.M., the flow was about 220 m^3/s. The July 4, 1990 flood peak occurred around 6:20 P.M. with a flow of 287 m^3/s.

4.0 RIVER AND VALLEY FEATURES

4.1 General

The Battle River originates at Battle Lake and flows southeast towards Ponoka. At Ponoka, the river drains about 1830 km^2 of mainly agricultural land. At the upstream end of the study reach around the Highway 2A bridge, the valley is about 250 m wide. About 300 m downstream of the Highway 2A bridge to just upstream of the CPR bridge, the river valley narrows to a bottom width of 30 m and top width of 100 m. In the vicinity of the CPR bridge, the valley widens to an average of 300 m at the bottom and 500 m at the top. Along this stretch of the valley, which extends through the present developed urban area, high bluffs are encountered on the east valley slope and terraced features are common along the west valley slope.

4.2 Channel Characteristics

Along the study reach, the Ponoka River has a well-defined channel. Upstream of the Highway 53 bridge, the channel is narrow and the river exhibits few meanders. Downstream of the pedway bridge, the channel widens and meanders are more pronounced. Several oxbows are also found along this downstream reach that are characteristic of actively meandering channels.

4.3 Floodplain Characteristics

Along the study reach, the floodplain within the river valley is well defined. The vegetation consists of a mixture of pasture, bushes, shrubs and the occasional clump of trees. The few buildings located within the floodplain are near the floodplain boundary. At the 100 year flood level, floodplain widths vary between 80 m to 100 m in the narrow valley section upstream of the CPR bridge. Downstream of the CPR bridge to the sewage lagoons, the floodplain width ranges between 300 to 500 m. The sewage lagoons, located downstream of Ponoka, restrict the floodplain width to between 80 m to 250 m.

5.0 CALCULATION OF FLOOD LEVELS

5.1 HEC-2 Program

The water surface profiles for this study were calculated using the HEC-2 program (Version 4.6.2 released in May 1991). This program was developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. The HEC-2 program calculates water surface profiles for steady, gradual varied flow in man-made or natural channels. Other program features include providing information for floodplain mapping, floodplain management, flood insurance evaluations, and floodway encroachment studies.

The HEC-2 program has the following capabilities:

1. Calculation of subcritical and supercritical flow profiles.

2. Division of flow between channel and floodplain regions.

3. Modelling the effects of flow obstructions such as bridges, culverts, weirs, etc.

4. Assessment of the effects of floodplain development, floodway encroachments and channel improvements.

Additional features of the HEC-2 package include a formatted data editor, a data error checking program, standard and optional output, and plotting displays.

The HEC-2 program computes water levels using the one-dimensional energy equation. It calculates energy loss due to friction using Manning's equation. Starting at a known water level, the program uses the standard Step Method to solve the energy equation between successive pairs of cross sections. By proceeding stepwise along the channel, the program determines the water level at each cross section.

Program limitations include the following:

1. The flow is steady.

2. The flow is gradually varied. Rapidly varied flow cannot be simulated.

3. The flow is one dimensional. Flow at rapid expansions or contractions and flow on large floodplains may be two or three dimensional.

4. The flow boundaries are fixed. Natural channel boundaries are usually mobile, especially during flood events.

5. The slope of the channel must be small.

6. Super-elevation at river bends is not simulated.

5.2 Geometric Data

5.2.1 Cross Section Data

The HEC-2 model of the Battle River through Ponoka uses a total of 37 cross sections. Cross section locations were plotted on the 1:5000 scale orthophoto maps provided at the end of this report. Distances between cross sections were determined from the orthophoto maps using an electronic planimeter. Distances along the main channel were measured along the river thalweg. Floodplain distances were measured as straight lines between the averaged weighted centroid of the floodplain at each cross section. The total chainage along the main channel between cross section 1 to cross section 21.4 is 8087 m.

The spacing and alignment of the cross sections was selected by Alberta Environmental Protection, River Engineering Branch. Cross section spacing was found to be adequate. In a few instances, conveyance change between cross sections was found to be outside the acceptable range for the 1:100 year flood. A review of cross section data indicated that those high conveyance changes were due to major floodplain shifts between left and right banks (or vice versa). Based on further review, it was decided that these high conveyance changes would have a negligible effect on computed water levels.

5.2.2 Bridges

Five bridges are located along the study reach of the Battle River. Information about the bridges was provided by Alberta Environment from surveys conducted by W.S. Barlow Surveying Ltd. of Spruce Grove. Additional bridge data was obtained during a March 3, 1991 site visit by Hydrotech Consulting Ltd. A selection of photos taken of the bridges during this site visit are provided in Appendix B.

Using the above information, the bridge data was coded into the HEC-2 model for the Battle River. An initial review of the bridge data indicated that each of the bridges may cause one or more special flow conditions such as pressure flow through bridge opening, weir flow over bridge deck, submerged bridge approaches, and bridge piers located within the flow path. To account for these flow conditions, all five bridges were modelled using the Special Bridge routine in the HEC-2 model. This routine uses a trapezoidal approximation of the bridges opening and can model any combination of low flow, pressure flow, and weir flow. Bridge piers are accounted for by specifying a total width of flow obstruction due to the piers.

5.3 Hydraulic Parameters

5.3.1 Expansion and Contraction Coefficients

Expansion and contraction coefficients are used to compute energy losses associated with changes in the shape of river cross sections. The loss due the expansion of flow is usually larger than the loss due to contraction. Losses for short, abrupt transitions are larger than losses from gradual transitions. The HEC-2 manual suggests expansion and contraction losses of 0.3 and 0.1 for gradual transitions and 0.5 and 0.3 at bridges. Selected values for this study were a contraction coefficient of 0.3 and an expansion coefficient of 0.5. These values were chosen to account for the rapid cross section changes in the vicinity of the various bridges.

5.3.2 Manning's Roughness

The HEC-2 program uses the Manning's equation to calculate channel friction losses. Channel roughness is highly variable and depends on such factors as channel shape, bed and bank material, vegetation cover, and river stage. In order to consider this variability, it is important to calibrate Manning's n using known stages for specific discharges.

Direct calibration of overbank or floodplain roughness coefficients is usually not feasible since stage-discharge measurements are typically not available for only the floodplain region. Manning's n for the floodplain regions was chosen using values obtained from roughness descriptions in Chow (1959). Floodplain vegetation cover at each left and right overbank location was determined from aerial photographs. Table 6 lists Manning's n values for various floodplain vegetation cover.

TABLE 6
Manning's n for Floodplain Region

Vegetation Cover	Manning's n
Pasture and grass	0.055
Scattered bush	0.085
Trees and heavy bush	0.115

5.4 Calibration of Manning's Roughness

5.4.1 Methodology

A reliable calibration of Manning's n is a critical factor in accurately predicting flood levels. The methodology used was to adjust Manning's roughness in the HEC-2 program until known highwater elevations are duplicated by the model with an acceptable degree of accuracy.

In this study, Manning's n along the main channel was calibrated using the April 24, 1982 and July 4, 1990 highwater marks listed in Table 3 and Table 4. A calibration was also performed using the rating curve for the Water Survey of Canada gauge 05FA001 to estimate the variation in Manning's n with discharge. Combining these results, Manning's n values were determined for modelling 1:10 year to 1:100 year floods along the study reach.

5.4.2 Calibration Using 1990 Highwater Mark Data

The July 4, 1990 flood is well described by highwater marks distributed over most of the study reach. Two calibration procedures were used to calibrate to the 1990 flood discharge of 287 m^3/s. The first calibration used a single Manning's n value for the channel along the entire study reach. The second calibration divided the study reach into three separate segments based on channel characteristics. Manning's n values were determined for each channel segment. Better results were obtained using this second calibration procedure. Final Manning's n calibration values are listed in Table 7.

As indicated previously in Section 4, the upper and lower reaches of the river exhibit distinct river morphology and topography: The upper reach of the river is characterized by a narrow and fairly straight channel whereas the lower reach exhibit a more meandering channel configuration with wide floodplain. Thus, it seems appropriate that the calibration resulted in a higher channel roughness for the lower reach of the river.

TABLE 7
Manning's n for Main Channel

Location	Cross Section Range	Manning's n
Downstream of pedway bridge	1 to 12	0.051
Between Hwy. 53 bridge and pedway bridge	13.1 to 14.4	0.049
Upstream of Highway 53 bridge	15 to 21.4	0.041

The final calibration results for the 1990 flood are listed in Table 8 and are plotted in Figure 3. The differences between the highwater marks and the calculated water level profile ranged from +0.19 m (cross section 21.3) to -0.13 m (cross section 14.3). The largest differences between surveyed and computed water levels occur near bridges where water levels change rapidly.

Table 8
Model Calibration based on 1990 Flood Highwater Marks

Corresponding Cross Section	Station (m)	Highwater Mark Elevation (m)	Computed Elevation (m)	Elevation Difference (m)
u/s of 2	576	802.195	802.23	-0.04
d/s of 6	1995	802.928	802.95	-0.02
8	2906	803.174	803.15	+0.02
10.2	4109	803.451	803.36	+0.09
14.2	5705	803.718	803.83	-0.11
14.3	5725	803.708	803.84	-0.13
17.1	6422	804.399	804.49	-0.09
17.4	6488	804.658	804.60	+0.06
21.1	7999	805.780	805.8	-0.02
21.2	8049	805.754	805.73	+0.02
21.3	8067	806.088	805.9	+0.19

The rating curve for the Water Survey of Canada gauge 05FA001 shows that the geodetic elevation for a discharge of 287 m^3/s is 804.44 m. The surveyed elevation of the highwater mark, located just six meters downstream of the gauge, is 804.399 m. The difference is less than 0.05 m which suggests a very good correlation between highwater marks and gauge reading.

5.4.3 Calibration Using 1982 Highwater Mark Data

The April 24, 1982 flood, with a peak discharge of 90.9 m^3/s, is significantly different in magnitude than the 1990 flood. Therefore, this flood provides a good check on the model calibration. The best results for the 1982 flood were obtained using the same Manning's n values determined for the 1990 flood. The similarity of the 1982 and 1990 flood calibration results will be explained further in the following section dealing with the variation in Manning's n versus discharge.

The final calibration results for the 1982 flood are listed in Table 9 and plotted in Figure 3. The differences between the highwater marks and the calculated water level profile ranged from +0.23 m (cross section 10.3) and -0.10 m (cross section 13.2). Similar to the 1990 flood calibration, the largest differences between surveyed and computed water levels occur near bridges.

TABLE 9
Model Calibration based on 1982 Flood Highwater Marks

Corresponding Cross Section	Station (m)	Highwater Mark Elevation (m)	Computed Elevation (m)	Elevation Difference (m)
u/s of 2	576	801.32	801.32	0.00
d/s of 6	1995	801.65	801.60	+0.05
8	2906	801.86	801.80	+0.06
10.2	4109	802.08	801.99	+0.09
10.3	4129	802.23	802.00	+0.23
10.5	4319	802.15	802.10	+0.05
13.2	5392	802.40	802.50	-0.10
13.3	5401	802.50	802.50	0.00
14.2	5705	802.54	802.59	-0.05
14.3	5725	802.61	802.59	+0.02
d/s of 17.1	6374	802.88	802.93	-0.05
d/s of 17.1	6412	802.91	802.98	-0.07
17.2	6452	803.05	803.05	0.00

TABLE 9 (cont.)

Corresponding Cross Section	Station (m)	Highwater Mark Elevation (m)	Computed Elevation (m)	Elevation Difference (m)
u/s of 17.3	6470	803.04	803.06	-0.02
17.4	6488	802.99	803.03	-0.04
u/s of 17.4	6513	803.28	803.06	+0.22
20	7893	803.96	803.99	-0.03
u/s of 21.1	8019	804.07	804.04	+0.03
21.2	8049	804.11	804.04	+0.07
21.3	8067	804.12	804.05	+0.07
21.4	8087	804.15	804.14	+0.01

The rating curve for the Water Survey of Canada gauge 05FA001 shows that the geodetic elevation for a discharge of 90.9 m^3/s is 802.92 m. The surveyed elevation of the highwater mark, located about 16 m downstream of the gauge, is 802.91 m. The difference is only 0.01 m which also suggests a very good correlation between highwater marks and gauge reading.

5.4.4 Calibration Using the WSC Rating Curve

A calibration of Manning's n versus discharge was conducted using the rating curve from the Water Survey of Canada 05FA001 established during the period of 1976 to 1990. During this period, the gauge was located downstream of the CPR bridge and about six meters upstream of cross section 17.1.

As computed previously, calibrated Manning's n values at the gauge are 0.041 for both the 1982 and 1990 flood events. Further calibrations were conducted using the WSC rating curve for various discharges corresponding to the 1:2 to 1:50 year flood event (the rating curve was extended from the maximum flow of 303 m^3/s to the 1:50 year flood flow of 331 m^3/s). Calibrated Manning's n values based on the 1990 flood were multiplied by a constant factor (called FN in the J2.6 HEC-2 field) until the profile matched the rating curve elevation for the given discharge. Results of this calibration are listed in Table 10 and plotted on Figure 4. The plot indicates that Manning's n increases in value for discharge up to near bankfull. Subsequently, Manning's n value decreases for larger discharges. These calibration results were extended to the 1:100 year flood flow of 452 m^3/s and are listed in Table 10 and shown on Figure 4. Interestingly, both the 1990 and 1982 flood magnitudes correspond to similar Manning's n values which explain the previous calibration results.

TABLE 10
Manning's n Calibration for Various Discharges at the WSC Gauging Station 05FA001

Flood Event	Discharge (m^3/s)	FN[1]	Manning's n	Rating Curve Elevation (m)	Computed Elevation (m)	Diff. Between Rating Curve and Computed (m)
1:2	31	0.84	0.0344	801.77	801.79	-0.02
1:5	83	0.93	0.0381	802.80	802.81	-0.01
1982 Flood	90.9	1.00	0.041	802.92	802.99	-0.07
1:10	137	1.15	0.0472	803.61	803.61	0
1:20	207	1.11	0.0455	804.12	804.11	0.01
1990 Flood	287	1.00	0.041	804.45	804.49	-0.04
1:50	331	0.87	0.0357	804.59	804.58	0.01
1:100	452	0.7	0.0287	n/a	804.94	n/a

[1] All Manning's n values used in the HEC-2 model are multiplied by the FN factor

5.5 Model Sensitivity

5.5.1 Initial Water Level

A sensitivity analysis was conducted on the effect of the initial water level at the downstream end of the study reach on the overall computed water surface profile. This analysis is required to show the effect of an error in the estimated initial water level on the predicted flood levels within the Ponoka corporate limits. The July 4, 1990 flood event was used for this analysis since it corresponded to a major flood event and results can be compared to recorded highwater marks.

The base simulation starting elevation at cross section 1 was 801.75 m. This elevation was derived from a best fit calibration using the first highwater mark located 576 m upstream of cross section 1. An additional four simulations were conducted using starting elevations 0.5 m and 1.0 m higher and lower than the base simulation starting level. The results of this analysis are listed in Table 11 and are plotted in Figure 5.

TABLE 11
Sensitivity Analysis of Water Surface Profiles to Initial Water Level

Cross Section	Station (m)	Water Levels for 1.0 m Higher (m)	Water Levels for 0.5 m Higher (m)	Base Simulation Water Levels (m)	Water Levels for 0.5 m Lower (m)	Water Levels for 1.0 m Lower (m)
1	0	802.75	802.25	801.75	801.25	800.75
2	496	802.90	802.49	802.16	801.93	801.87
3	806	803.03	802.69	802.45	802.31	802.28
4	1112	803.29	803.03	802.85	802.76	802.74
5	1533	803.31	803.05	802.88	802.79	802.77
6	2145	803.37	803.12	802.97	802.89	802.87
7	2521	803.43	803.20	803.07	803.00	802.98
8	2906	803.48	803.27	803.15	803.09	803.07
9	3508	803.53	803.34	803.23	803.17	803.16
10.1	4079	803.61	803.44	803.35	803.30	803.29
10.2	4109	803.63	803.46	803.36	803.32	803.30
10.3	4129	803.64	803.47	803.37	803.33	803.32
10.4	4169	803.67	803.52	803.44	803.40	803.39
10.5	4319	803.70	803.55	803.47	803.44	803.43
11	4749	803.75	803.62	803.54	803.51	803.51
12	5219	803.83	803.72	803.66	803.63	803.62
13.1	5352	803.88	803.77	803.72	803.69	803.69
13.2	5392	803.88	803.77	803.72	803.69	803.69
13.3	5401	803.88	803.77	803.72	803.69	803.69
13.4	5441	803.96	803.87	803.83	803.81	803.80
14.1	5645	803.99	803.90	803.86	803.84	803.84
14.2	5705	803.96	803.87	803.83	803.81	803.81
14.3	5725	803.97	803.88	803.84	803.82	803.82
14.4	5755	804.10	804.02	803.98	803.96	803.96
15	5978	804.20	804.13	804.09	804.07	804.07
16	6254	804.30	804.24	804.20	804.19	804.19
17.1	6422	804.54	804.50	804.49	804.48	804.48
17.2	6452	804.57	804.54	804.52	804.51	804.51
17.3	6468	804.61	804.57	804.56	804.55	804.55
17.4	6488	804.65	804.62	804.60	804.60	804.60
18	6928	805.15	805.13	805.13	805.12	805.12
19	7448	805.52	805.51	805.51	805.51	805.50
20	7893	805.78	805.77	805.77	805.77	805.77
21.1	7999	805.81	805.80	805.80	805.80	805.80
21.2	8049	805.75	805.74	805.73	805.73	805.73
21.3	8067	805.92	805.91	805.90	805.90	805.90
21.4	8087	800.38	806.18	806.17	806.17	806.17

The analysis indicates the following:

1. For the 0.5 m and 1.0 m lower simulation, computed water levels are within 0.10 m of the base simulation water levels at cross section 6. Cross section 6 is located close to the present town of Ponoka corporate limits.

2. For the 0.5 m and 1.0 m lower starting levels, water levels approach within 0.05 m of the base simulation water levels in the vicinity of the 50 Avenue bridge.

3. For the 0.5 m higher starting level, water levels at cross section 6 are about 0.15 m above the base simulation.

4. For the 0.5 m higher simulation, water levels approach within 0.05 m of the base simulation in the vicinity the pedway bridge.

5. For the 1.0 m higher simulation, water levels at cross section 6 are about 0.40 m higher than the base simulation.

6. For the 1.0 m higher simulation, water levels do not approach within 0.05 m of the base simulation until reaching the CPR bridge.

7. Underestimating the starting level results in smaller errors than overestimating the starting level.

8. Figure 5 indicates that the four highwater marks located downstream of the pedway bridge all fall within the limits defined by the +0.5 and -0.5 m starting water levels.

Based on the above, the study reach is quite sensitive to the starting level. However, recorded highwater marks for the 1982 and 1990 flood events provide a very good guideline in choosing starting water levels for larger flood events. Therefore, the errors are probably less than 0.3 m for estimating the starting water level for the critical flood events in this study. This corresponds to errors in flood level estimates with the town corporate limits of less than 0.1 m

Though not documented in this report, simulations performed using the 1:100 year design discharge produced similar results.

5.5.2 Main Channel Manning's n

A sensitivity analysis was conducted on the effect of the main channel Manning's n on the overall computed water surface profile. This analysis is required to show the effects of an error in estimating

Manning's n on the predicted flood levels along the study reach. The July 4, 1990 flood was also used for this second sensitivity analysis.

The development of the base simulation values of Manning's n along the study reach are described in Section 5.4. An additional four simulations were conducted by modifying the main channel Manning's n values by a fixed percentage. The resulting Manning's n values are 80%, 90%, 110% and 120% of the base simulation values. The results of this analysis are listed in Table 12 and are plotted in Figure 6.

The analysis indicates the following:

1. A 10% variation of the base simulation Manning's n values results in less than a 0.20 m change in water level compared to the base simulation.

2. Increasing the base simulation Manning's n values by 20% results in less than a 0.35 m change in water level.

3. Decreasing the base simulation Manning's n values by 20% results in less than a 0.45 m change in water level.

4. Figure 6 shows that all the 1990 flood highwater marks are very close to being contained within the 90% to 110% Manning's n profiles.

Based on the above, the study reach is quite sensitive to variations in Manning's n. However, similar to the conclusion for the initial water level sensitivity analysis, the 1982 and 1990 flood provide a very good guideline in choosing Manning's n for larger flood events. Therefore, the error in estimating Manning's n values is probably less than 10%. This corresponds to errors in flood level estimates of less than 0.2 m.

Though not documented in this report, simulations performed using the 1:100 year design discharge produced similar results.

TABLE 12
Sensitivity Analysis of Water Surface Profiles to Manning's n Values

Cross Section	Station (m)	Water Levels for Manning's n at 80% (m)	Water Levels for Manning's n at 90% (m)	Base Simulation Water Levels (m)	Water Levels for Manning's n at 110% (m)	Water Levels for Manning's n at 120% (m)
1	0	801.75	801.75	801.75	801.75	801.75
2	496	802.03	802.09	802.16	802.22	802.29
3	806	802.20	802.33	802.45	802.57	802.68
4	1112	802.66	802.76	802.85	802.95	803.04
5	1533	802.68	802.79	802.88	802.98	803.08
6	2145	802.75	802.87	802.97	803.08	803.18
7	2521	802.85	802.96	803.07	803.18	803.28
8	2906	802.92	803.04	803.15	803.26	803.37
9	3508	802.99	803.11	803.23	803.34	803.46
10.1	4079	803.08	803.22	803.35	803.47	803.59
10.2	4109	803.10	803.23	803.36	803.48	803.60
10.3	4129	803.08	803.24	803.37	803.50	803.62
10.4	4169	803.23	803.32	803.44	803.55	803.66
10.5	4319	803.26	803.36	803.47	803.58	803.69
11	4749	803.33	803.43	803.54	803.66	803.77
12	5219	803.43	803.54	803.66	803.77	803.89
13.1	5352	803.48	803.60	803.72	803.83	803.95
13.2	5392	803.47	803.59	803.72	803.84	803.96
13.3	5401	803.47	803.59	803.72	803.84	803.96
13.4	5441	803.63	803.72	803.83	803.93	804.03
14.1	5645	803.65	803.75	803.86	803.97	804.08
14.2	5705	803.61	803.72	803.83	803.94	804.06
14.3	5725	803.62	803.73	803.84	803.95	804.07
14.4	5755	803.77	803.87	803.98	804.09	804.20
15	5978	803.85	803.97	804.09	804.21	804.33
16	6254	803.91	804.06	804.20	804.34	804.47
17.1	6422	804.22	804.35	804.49	804.61	804.73
17.2	6452	804.27	804.39	804.52	804.64	804.76
17.3	6468	804.32	804.43	804.56	804.68	804.79
17.4	6488	804.34	804.46	804.60	804.74	804.86
18	6928	804.83	804.98	805.13	805.27	805.40
19	7448	805.17	805.34	805.51	805.67	805.81
20	7893	805.43	805.60	805.77	805.93	806.08
21.1	7999	805.46	805.63	805.80	805.96	806.11
21.2	8049	805.37	805.56	805.73	805.90	806.06
21.3	8067	805.48	805.71	805.90	806.08	806.23
21.4	8087	805.82	806.00	806.17	806.32	806.45

5.6 1902 and 1948 Flood Discharge Estimates

As mentioned previously, flood discharges were estimated for both the 1902 and 1948 flood events using recorded highwater mark elevations. Initial estimates for these two floods had been produced prior to this study using the HEC-2 model developed in the 1979 Ponoka Floodplain study (Ref. 3). These estimates were 324 m^3/s for the 1902 flood and 250 m^3/s for the 1948 flood. These flood estimates were considered preliminary and needed to be verified with the current model.

The 1902 highwater mark is located between what is now the pedway bridge (cross section 13.4) and the Highway 53 bridge (cross section 14.1). Conditions along the river in 1902 that would affect the computed discharge include:

1. None of the existing bridges were in place.

2. A bridge built in 1900 and replaced in 1937, was located just upstream of the existing 50 Avenue bridge at approximately cross section 10.3. The bridge had a main span of 60 feet. No data is available regarding the bridge approach.

3. When the Highway 53 bridge was constructed at a much later time, extensive channelization was undertaken. Using airphoto interpretation, the old channel was drawn and distances between cross sections were adjusted to reflect the change in channel length.

Taking these factors into account, the HEC-2 program was modified and run with and without the 1900 bridge. With the 1900 bridge, the discharge was estimated at 300 m^3/s while without the bridge the discharge was estimated at 330 m^3/s. Due to the uncertainty about the 1900 bridge approaches and the likely low impact of the small structure on the flood level, it was decided that the initial estimate of 324 m^3/s was reasonable.

The 1948 highwater mark is located immediately downstream of what is now the pedway bridge (cross section 13.1). Conditions along the river in 1948 that would affect the computed discharge include:

1. The present 50 Avenue bridge was in place.

2. All other bridges were located upstream of the 1948 highwater mark and therefore have no influence on the computed flood discharge.

Considering these factors, the HEC-2 program was modified and run. The 1948 flood was estimated at 232 m^3/s compared to the initial estimate of 250 m^3/s. The 232 m^3/s estimate was recommended since it is based on more up-to-date data.

5.7 Computed Water Surface Profiles

The calibrated HEC-2 model was used to compute water surface profiles for the 1:10, 1:50 and 1:100 year flood events. Initial water levels at cross section 1 were extrapolated from the calibrated starting levels for the 1982 and 1990 flood events. The computed water surface profiles are listed in Table 13 and shown in Figure 7.

Modelling results relating to the various bridge crossings include:

1. The 50 Ave. bridge is submerged during the 1:100 year flood and the bridge low chord is under water during the 1:50 year flood. The bridge approaches are low and are already submerged during the 1:10 year flood.

2. The bridge low chord on the pedway bridge is under water during the 1:100 year event. The east approach path is low and is already submerged during the 1:10 year flood.

3. The bridge low chord on the Highway 53 bridge is more than three meters above the 1:100 year flood level. Both road approaches are also above the 1:100 year flood level.

4. The bridge low chord on the CPR bridge is under water during the 1:100 year flood. The bridge approaches are also above the 1:100 year flood level.

5. The bridge low chord on the Highway 2A bridge is under water during both the 1:50 and 1:100 year flood event. The south approach road is also submerged during the 1:100 year flood event.

TABLE 13
Computed Water Surface Elevations

Cross Section	Station (m)	1:10 Year Flood Levels (m)	1:50 Year Flood Levels (m)	1:100 Year Flood Levels (m)
1	0	801.34	801.89	802.30
2	496	801.54	802.29	802.69
3	806	801.74	802.53	802.84
4	1112	801.92	803.04	803.64
5	1533	801.95	803.07	803.67
6	2145	802.04	803.16	803.73
7	2521	802.15	803.25	803.82
8	2906	802.23	803.33	803.89
9	3508	802.31	803.41	803.97
10.1	4079	802.44	803.52	804.06
10.2	4109	802.45	803.53	804.07
10.3	4129	802.46	803.54	804.11
10.4	4169	802.55	803.60	804.12
10.5	4319	802.59	803.64	804.16
11	4749	802.68	803.71	804.22
12	5219	802.82	803.82	804.31
13.1	5352	802.91	803.87	804.35
13.2	5392	802.93	803.86	804.33
13.3	5401	802.93	803.86	804.33
13.4	5441	803.03	803.98	804.45
14.1	5645	803.06	804.01	804.47
14.2	5705	803.06	803.97	804.38
14.3	5725	803.06	803.98	804.40
14.4	5755	803.11	804.14	804.63
15	5978	803.17	804.25	804.76
16	6254	803.32	804.34	804.79
17.1	6422	803.61	804.58	804.95
17.2	6452	803.66	804.62	804.98
17.3	6468	803.67	804.67	805.31
17.4	6488	803.68	804.72	805.41
18	6928	804.18	805.21	805.70
19	7448	804.48	805.59	806.05
20	7893	804.72	805.87	806.33
21.1	7999	804.76	805.89	806.34
21.2	8049	804.75	805.80	806.19
21.3	8067	804.77	806.02	806.42
21.4	8087	804.89	806.35	806.89

5.8 Flood Frequency Map

A flood frequency map was prepared showing the extent of flooding along the Battle River for the 1:10, 1:50 and 1:100 year flood events. These flood lines were plotted on the 1:5000 scale orthophoto map of the study area. The map also shows legal boundaries, major street names, railroad, and other notable land features. The flood frequency map is located in the map pocket at the end of the report.

Flood lines are based on existing conditions and do not account for future development (encroachment) within the flood fringe. Flood limits were determined by identifying the intersection points of the computed water surface and the ground level at each cross section. Contour lines and surface features shown on the orthophoto map were used as guide for interpolating flood lines between cross sections.

Due to the relatively steep valley wall at several locations, flood lines often plotted at about the same location. To avoid confusion, only the higher flood line is shown where the flood lines coincide.

A review of the flood frequency map indicates that the flood limits are well defined and are generally located at or near the valley wall for each of the flood events which are plotted. The map also shows that significant flooding will occur along the minor drainage channel which joins the Battle River upstream of the Highway 53 bridge.

6.0 FLOODWAY DETERMINATION

6.1 Terminology

The following terms are defined according to the Alberta Environment publication, *Hydrologic and Hydraulic Guidelines for Floodplain Delineation*, 1990. These terms have specific application to flood risk mapping studies conducted under the Canada-Alberta Flood Damage Reduction Program.

1. **Flood Risk Area** - is defined as the area which would be flooded by the 1:100 year design flood. Within a flood risk zone, a distinction is made between the floodway and the flood fringe areas.

2. **Floodway** - is defined as the stream channel and that portion of the floodplain required to convey the 100 year design flood under constricted conditions, assuming no flow in the flood fringe. Flow within the floodway is typically deep, fast and destructive. Due to these features, new development in the floodway is discouraged.

3. **Flood Fringe** - is defined as the portion of the floodplain between the floodway and the outer boundary of the 100 year design flood. Flow within the flood fringe is slow and shallow and typically conveys only a minor portion of the overall flood discharge. Due to these features, development in the flood fringe may be allowed if such development is adequately flood proofed.

6.2 Floodway Criteria

Floodway criteria were obtained from the Alberta Environment publication, *Hydrologic and Hydraulic Guideline for Floodplain Delineation*, 1990. These criteria were applied to the 1:100 year design flood. Specific criterion that apply to this study are:

1. Water levels will typically rise when the floodway is narrower than the flood risk area. Maximum allowable rises in water level is 0.3 m.

2. In general, the floodway will include all areas where the flood depth exceeds one meter or the flood flow velocity exceeds one meter per second. However, these criteria may be relaxed in order to achieve a hydraulically smooth floodway boundary resulting in areas being transferred to the flood fringe zone. Areas where flood depths are greater than one meter but are largely ineffective in conveying flood flows, may also be transferred to the flood fringe zone.

3. In river reaches where channel flow velocities are already excessive under existing conditions, encroachment should be minimized so these velocities are not further increased.

6.3 Methodology

The following methodology was applied to determine the floodway in this study:

1. The flow distribution option in the HEC-2 program was used to calculate the velocity distribution at each cross section. Results showed that water depths were always greater than one meter in areas where the flow velocity was equal to one meter per second. Therefore, for this study, the one meter depth criterion governed.

2. An initial floodway boundary was delineated following the one meter flood depth contour and checked using the HEC-2 program.

3. At several locations, the initial floodway boundary was smoothed, resulting in some localized areas with flood depths greater than one meter being transferred to the flood fringe. This smoothed floodway was also checked using the HEC-2 program.

6.4 Results

The final floodway limits are shown on the Flood Risk Map located in the map pocket at the end of the report. The preparation of this map is discussed further in Section 7.

A comparison of 100 year flood levels computed with and without the floodway constriction are listed in Table 14. A review of this data indicates that the only water level rise due to floodway constriction is 0.01 m (cross section 14.4) which easily meets the floodway water level maximum rise criterion of 0.3 m.

The floodway one meter depth criterion had the greatest influence in setting the final floodway limits. This criterion was used to establish the floodway limits along most of the study reach. Hydraulic smoothing altered the floodway limits at a few locations and transferred low areas that did not effectively convey flood flows to the floodway fringe.

TABLE 14
Comparison of 1:100 Year Existing and Encroached (Floodway) Water Levels

Cross Section	Station (m)	Existing Water Levels (m)	Encroached Water Levels (m)	Elevation Difference (m)
1	0	802.30	802.30	0
2	496	802.69	802.69	0
3	806	802.84	802.84	0
4	1112	803.64	803.64	0
5	1533	803.67	803.67	0
6	2145	803.73	803.73	0
7	2521	803.82	803.82	0
8	2906	803.89	803.89	0
9	3508	803.97	803.97	0
10.1	4079	804.06	804.06	0
10.2	4109	804.07	804.07	0
10.3	4129	804.11	804.11	0
10.4	4169	804.12	804.12	0
10.5	4319	804.16	804.16	0
11	4749	804.22	804.22	0
12	5219	804.31	804.31	0
13.1	5352	804.35	804.35	0
13.2	5392	804.33	804.33	0
13.3	5401	804.33	804.33	0
13.4	5441	804.45	804.45	0
14.1	5645	804.47	804.47	0
14.2	5705	804.38	804.38	0
14.3	5725	804.40	804.40	0
14.4	5755	804.63	804.64	0.01
15	5978	804.76	804.76	0
16	6254	804.79	804.79	0
17.1	6422	804.95	804.95	0
17.2	6452	804.98	804.98	0
17.3	6468	805.31	805.31	0
17.4	6488	805.41	805.41	0
18	6928	805.70	805.70	0
19	7448	806.05	806.05	0
20	7893	806.33	806.33	0
21.1	7999	806.34	806.34	0
21.2	8049	806.19	806.19	0
21.3	8067	806.42	806.42	0
21.4	8087	806.89	806.89	0

7.0 FLOOD RISK MAP

7.1 General

A flood risk map was prepared using the 1:5000 scale orthophoto map as a base. This map is located in the map pocket at the end of the report. Information on the flood risk map includes:

1. Floodway area / boundary

2. Flood fringe area / boundary

3. The 1:100 year flood level at each cross section based on the flow constricted to the floodway.

4. Legal boundaries, major street names, railroad, and other notable land features.

Due to the relatively steep valley wall at several locations, the floodway boundary and the flood risk boundary often plot at about the same location. To avoid confusion, only the flood risk boundary is plotted where these lines coincide.

The mapping accuracy of flood levels is about plus or minus 0.5 m vertical distance. This is based on delineation of flood lines between cross sections using the one meter contour lines. Besides possible mapping errors, the HEC-2 modelling also produces small errors in calculating flood levels. These errors are caused by such factors as estimating hydraulic parameters and program limitations. These computational errors are judged to be less than 0.3 m which is within the mapping accuracy.

7.2 Areas Affected by the Floodway and the Flood Fringe

Areas affected by the floodway or flood fringe include the following:

1. At the downstream end of the study reach, several of the sewage lagoons located on the east floodplain are located within the flood fringe.

2. Near cross section 5, on the west floodplain, several farm buildings are located within the floodway. These buildings are situated outside the Town of Ponoka corporate limits.

3. Near cross section 11, on the west floodplain and within the meander loop, several small buildings and a residence associated with a farming operation are located within the floodway.

4. On the east floodplain, a number of existing homes located along 46 Street and 46A Street are situated in the flood fringe. At the end of 46A Street, three residential lots are located just within the floodway limits.

5. On the east floodplain, between cross sections 14 and 15, the floodway encroaches on a parking lot and roadway.

6. On the east floodplain, near cross section 16, several homes located along 35 Avenue are located in the flood fringe.

7. On the west floodplain, near cross section 16, several buildings associated with a feedlot operation are located within the flood fringe.

8.0 REFERENCES

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FIGURES

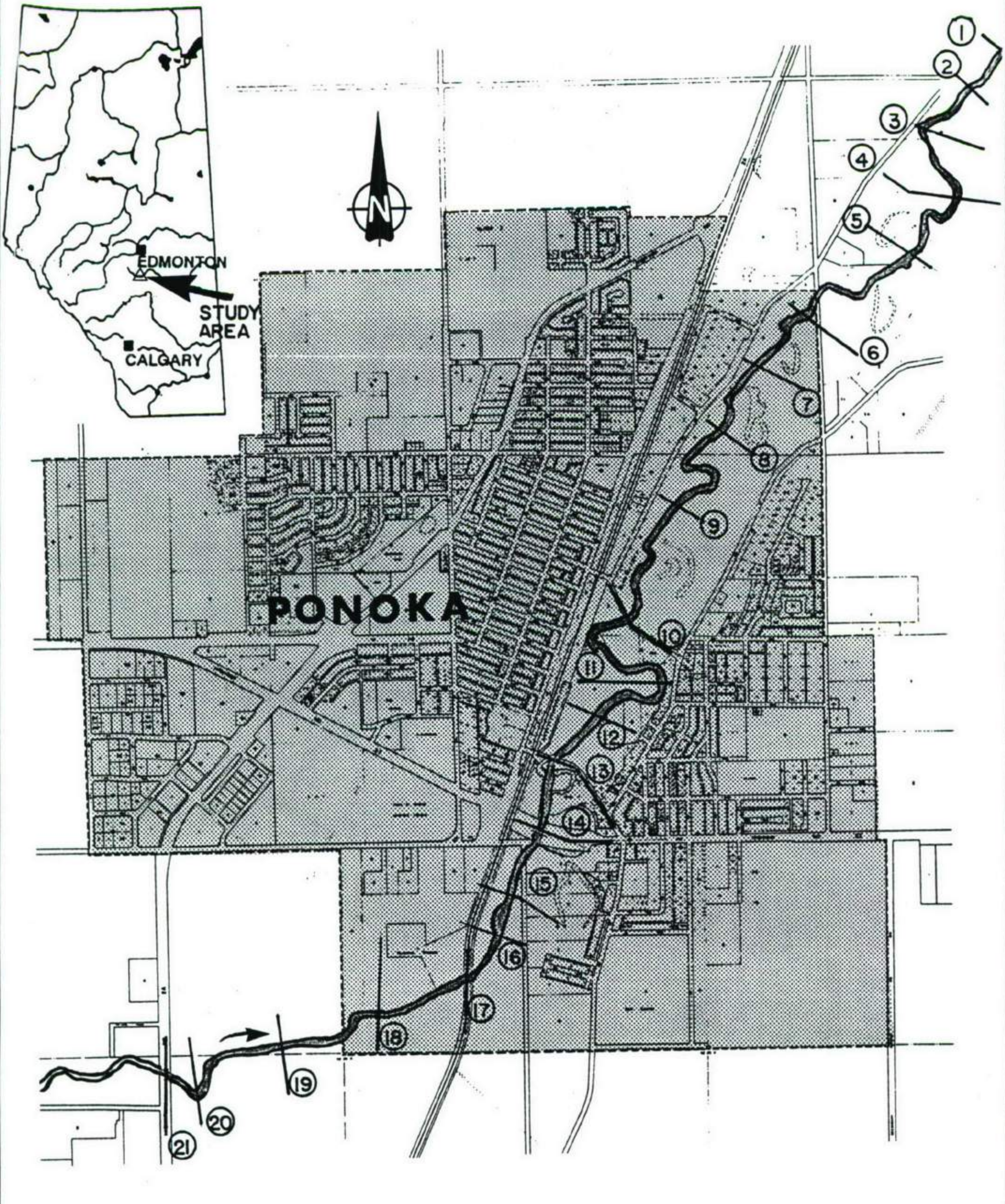

FIGURE 1

LOCATION PLAN AND STUDY REACH

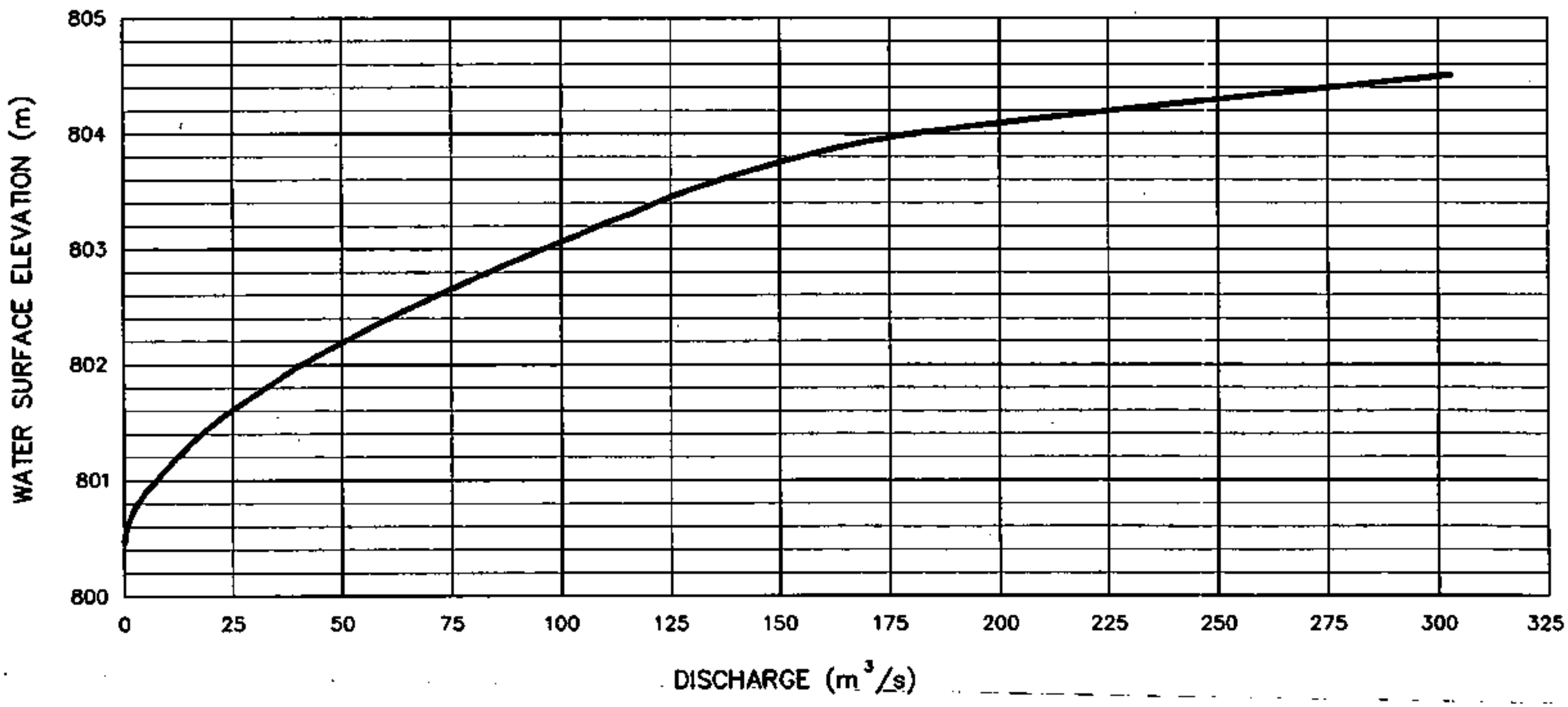

FIGURE 2

STAGE–DISCHARGE AT
WSC STATION 05FA001

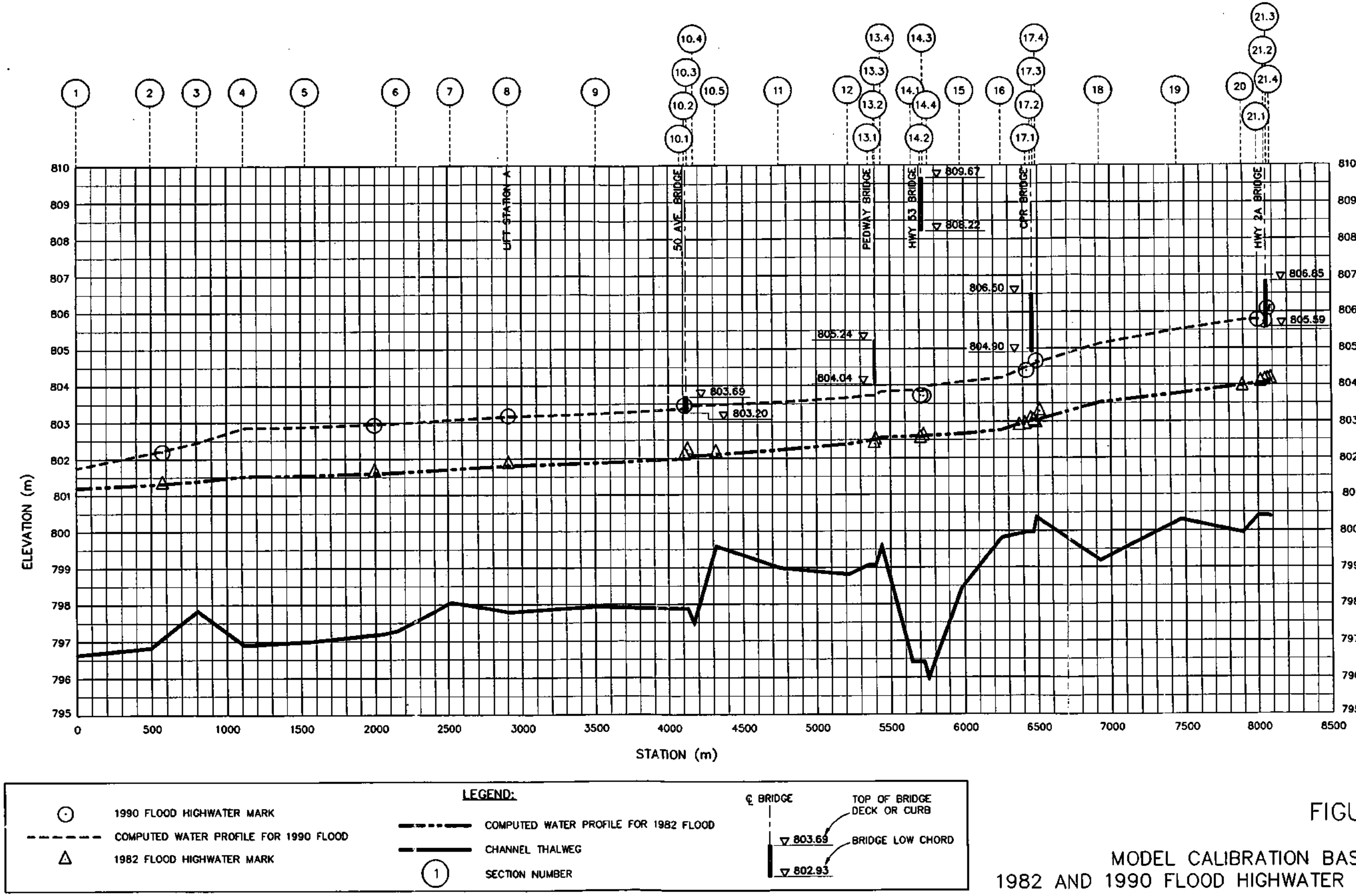

FIGURE 3

MODEL CALIBRATION BASED ON 1982 AND 1990 FLOOD HIGHWATER MARKS

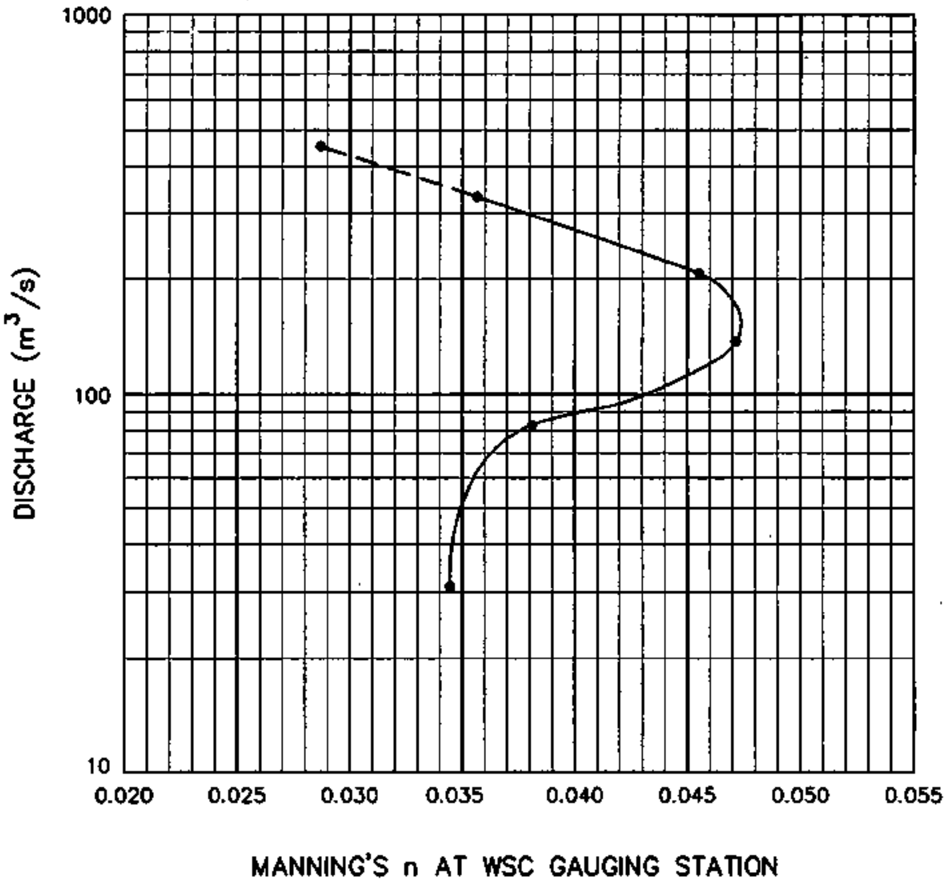

FIGURE 4

MANNING'S n CALIBRATION FOR VARIOUS DISCHARGES AT WSC GAUGING STATION 05FA001

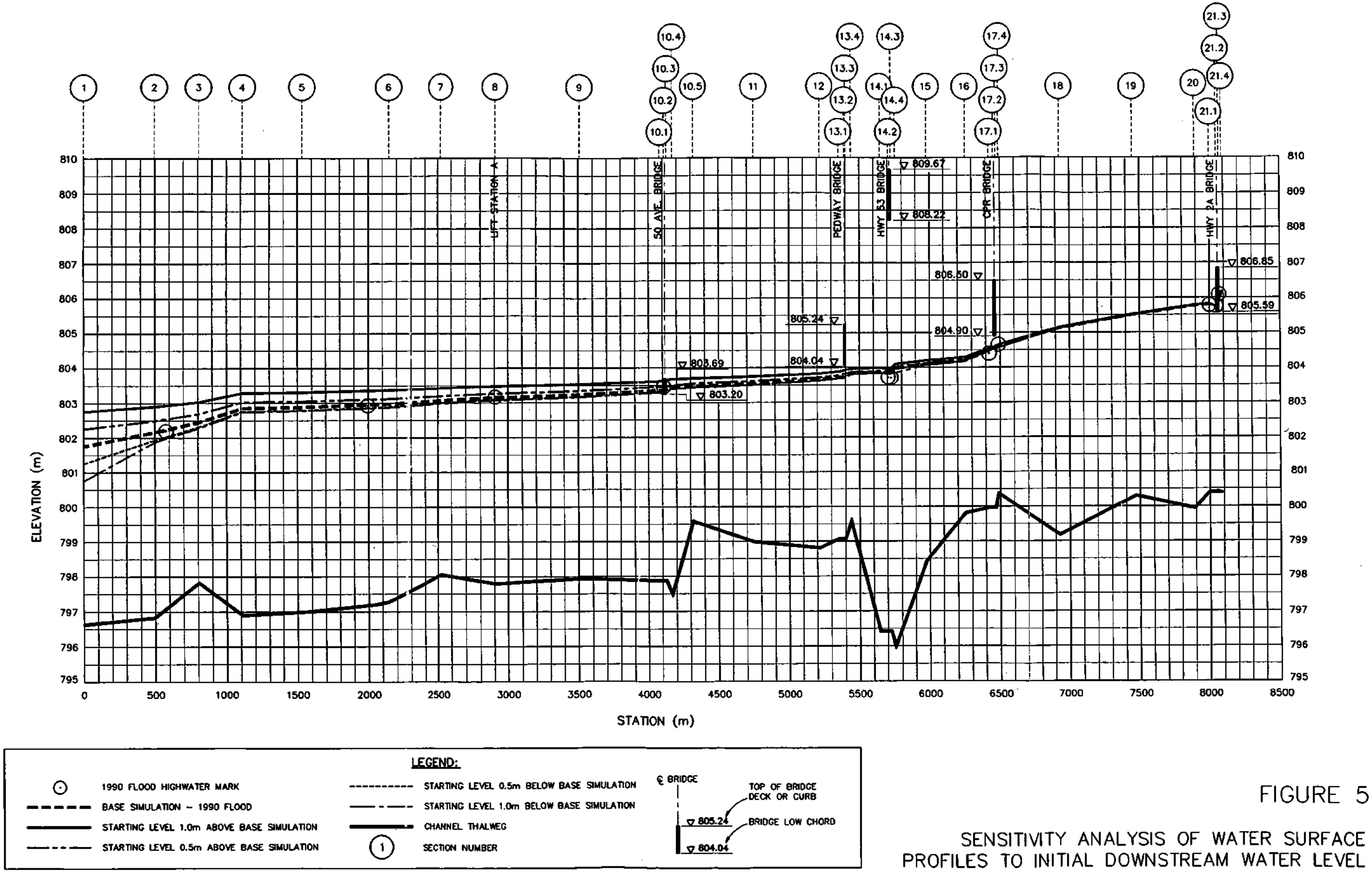

FIGURE 5

SENSITIVITY ANALYSIS OF WATER SURFACE PROFILES TO INITIAL DOWNSTREAM WATER LEVEL

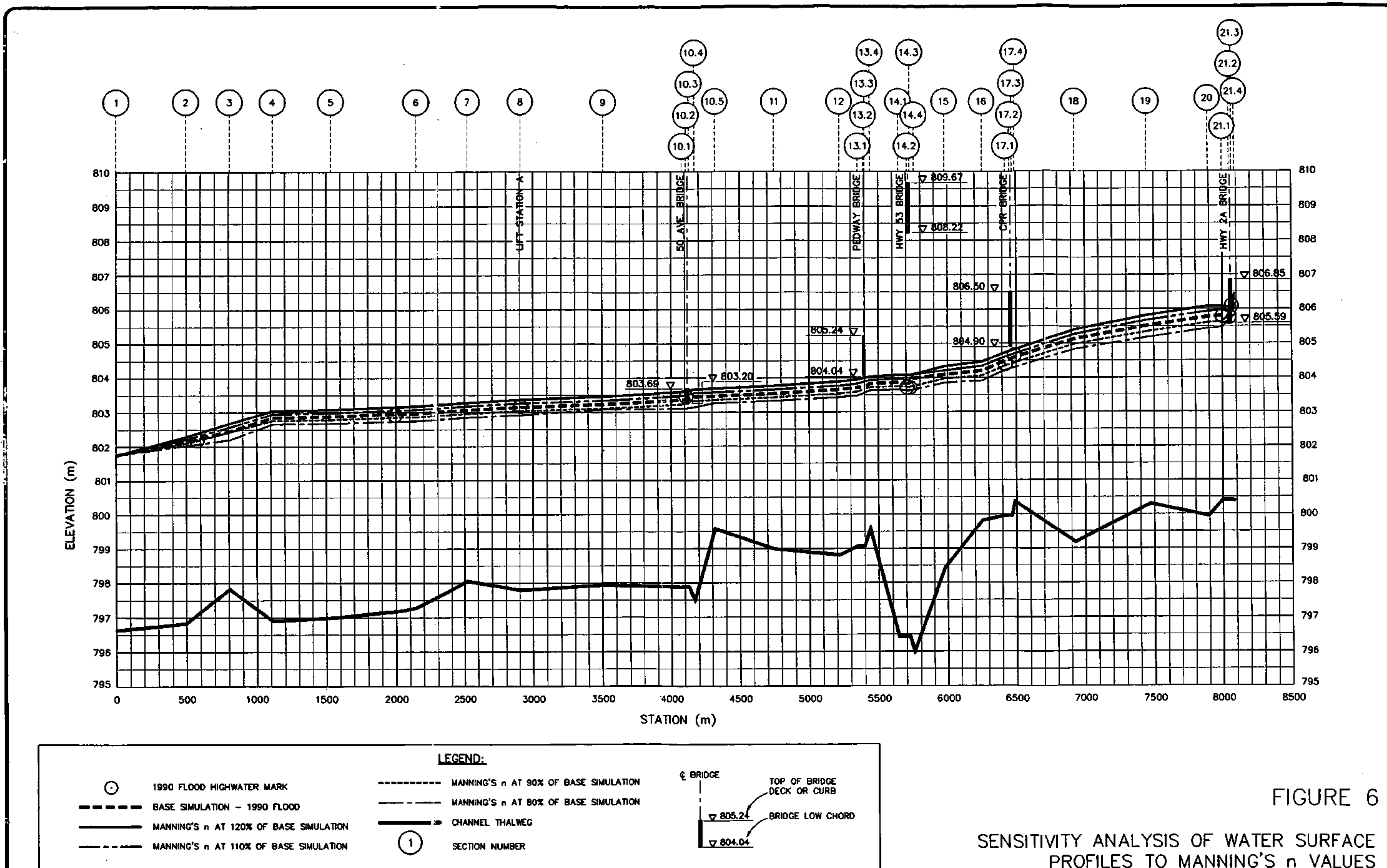

FIGURE 6

SENSITIVITY ANALYSIS OF WATER SURFACE PROFILES TO MANNING'S n VALUES

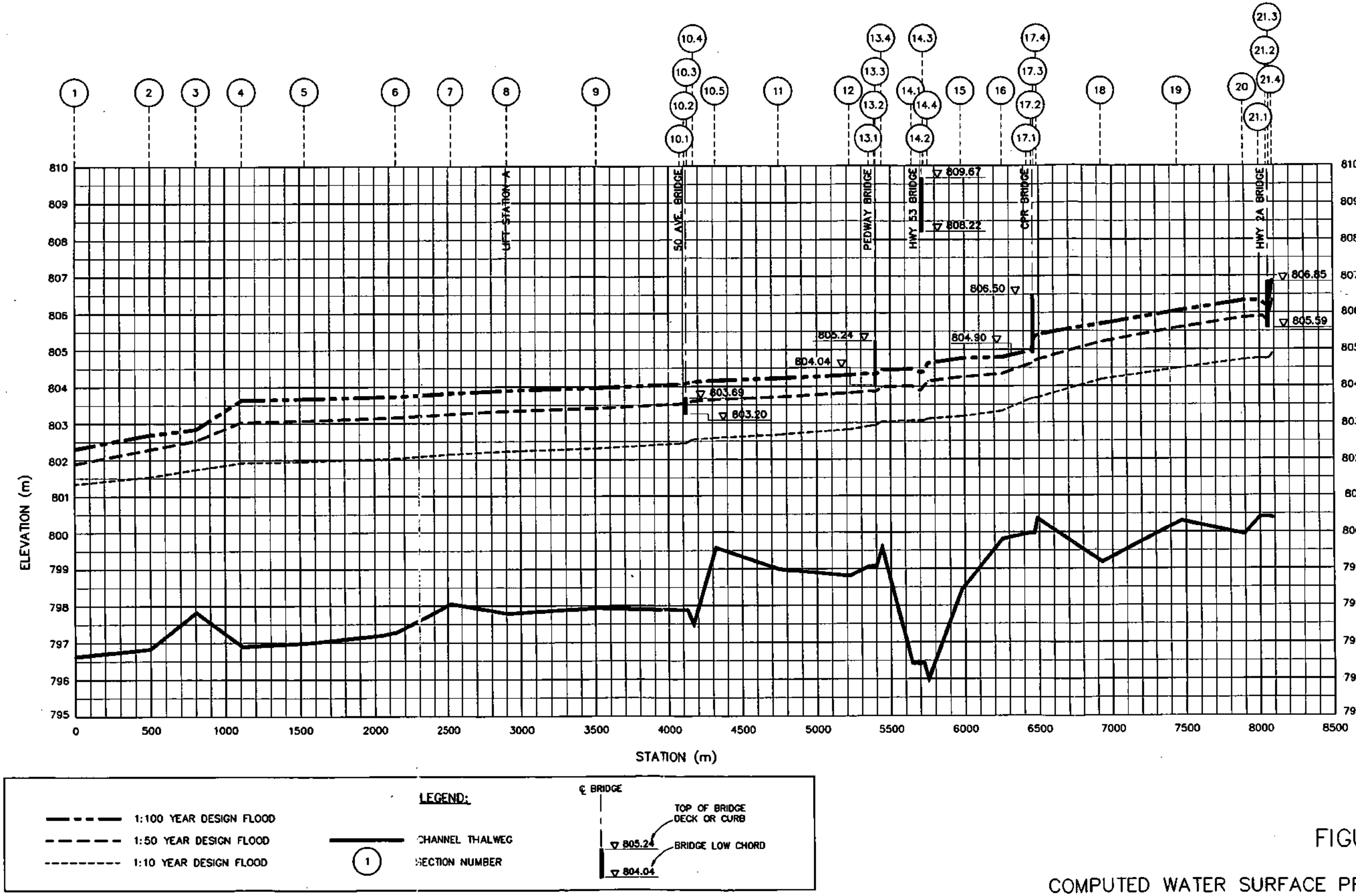

FIGURE 7

COMPUTED WATER SURFACE PROFILES

MAPS

Canada-Alberta FLOOD DAMAGE REDUCTION PROGRAM

Environment Canada | Environnement Canada | Alberta ENVIRONMENT

SCALE 1:5000

0 | 200 | 400 | 600 m

Projection: 3° Transverse Mercator, Scale factor 0.9999 at Reference Meridian 114°

Hydrographic feactures derived from Oct 1989 aerial photography, except as noted.

Contour interval: 1 METRE. Where the ground is fully obscured, contours are dashed.

Elevations: in metres above mean sea level, North American Datum, 1927

1 metre contour

5 metre index contour

Depression contour

927.9 Spot height in metres

Legend :

Flood Fringe / Floodway / Flood Fringe — Flood Risk Area

803.73 — 1:100 Year Flood Level (m)

24 — cross-section number and location

Town of Ponoka corporate limit

HYDROTECH
CONSULTING LTD.

Battle River at Ponoka Flood Risk Mapping Study

FLOOD RISK MAP

Date : OCT. 1994 | Job Number : A 158 | Drawing FR-1

SCALE 1:5000

0 200 400 600 m

Projection: 3° Transverse Mercator, Scale factor 0.9999 at Reference Meridian 114°

Hydrographic feactures derived from Oct 1989 aerial photography, except as noted.

Contour interval: 1 METRE. Where the ground is fully obscured, contours are dashed.

Elevations: in metres above mean sea level, North American Datum, 1927

1 metre contour

5 metre index contour

Depression contour

927.9 Spot height in metres

Legend :

1:10 year flood limit

1:50 year flood limit

1:100 year flood limit

24 cross – section number and location

Town of Ponoka corporate limit

Note :

Where one or more of the flood limits is not shown, it can be assumed to be coincident with the next higher flood limit

Canada-Alberta FLOOD DAMAGE REDUCTION PROGRAM

Environment Canada · Environnement Canada · Alberta ENVIRONMENT

HYDROTECH CONSULTING LTD.

Battle River at Ponoka Flood Risk Mapping Study

FLOOD FREQUENCY MAP

Date : OCT. 1994 Job Number : A 158 Drawing FF-1

APPENDIX A

1990 Flood Photographs

PLATE A-1 FLOOD OVERVIEW
LOOKING DOWNSTREAM FROM SECTION X19

PLATE A-2 FLOOD OVERVIEW
LOOKING UPSTREAM FROM SECTION X1

PLATE A-3 SECTION X1, LOOKING UPSTREAM

PLATE A-4 SECTION X5, LOOKING NORTH WEST

PLATE A-5 PEDWAY AT SECTION X13, LOOKING WEST

PLATE A-6 SECTION X16, LOOKING NORTH

PLATE A-7 C.P.R. BRIDGE AT SECTION X17, LOOKING NORTH

PLATE A-8 HIGHWAY 2A BRIDGE AT SECTION X 21, LOOKING NORTH

APPENDIX B

Photographs of Bridges

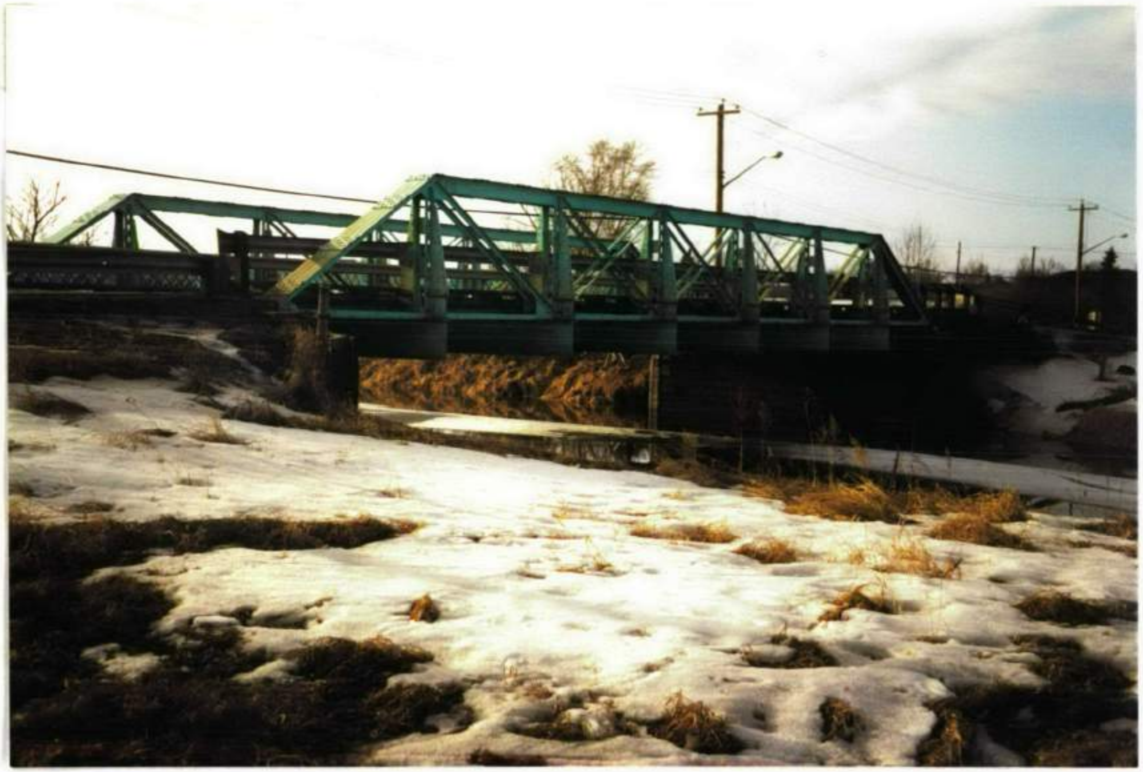

PLATE B-1. 50A AVENUE BRIDGE (SECTION X10)
LOOKING UPSTREAM, MARCH 3, 1992

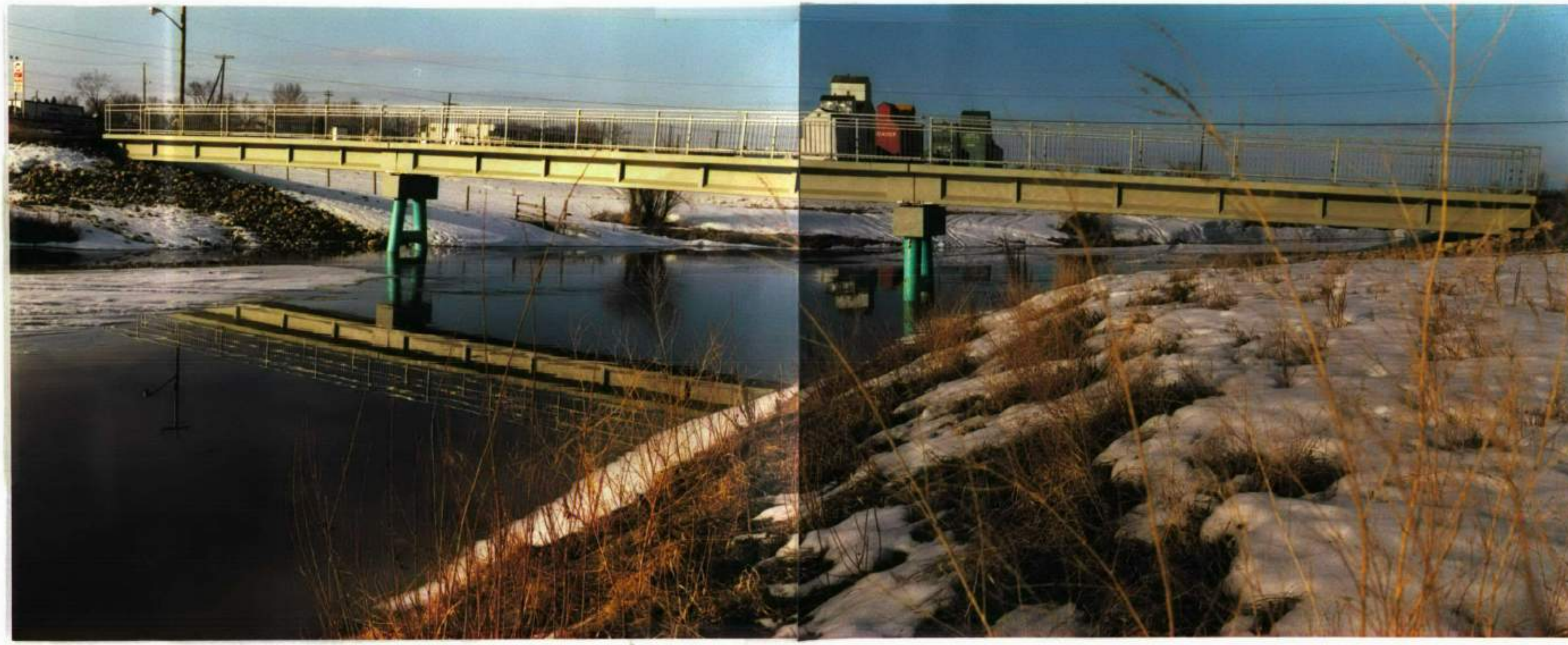

PLATE B-2. PEWAY BRIDGE (SECTION 13)
LOOKING DOWNSTREAM, MARCH 3, 1992

PLATE B-3 HIGHWAY 53 BRIDGE (SECTION 14)
LOOKING DOWNSTREAM, MARCH 3, 1992

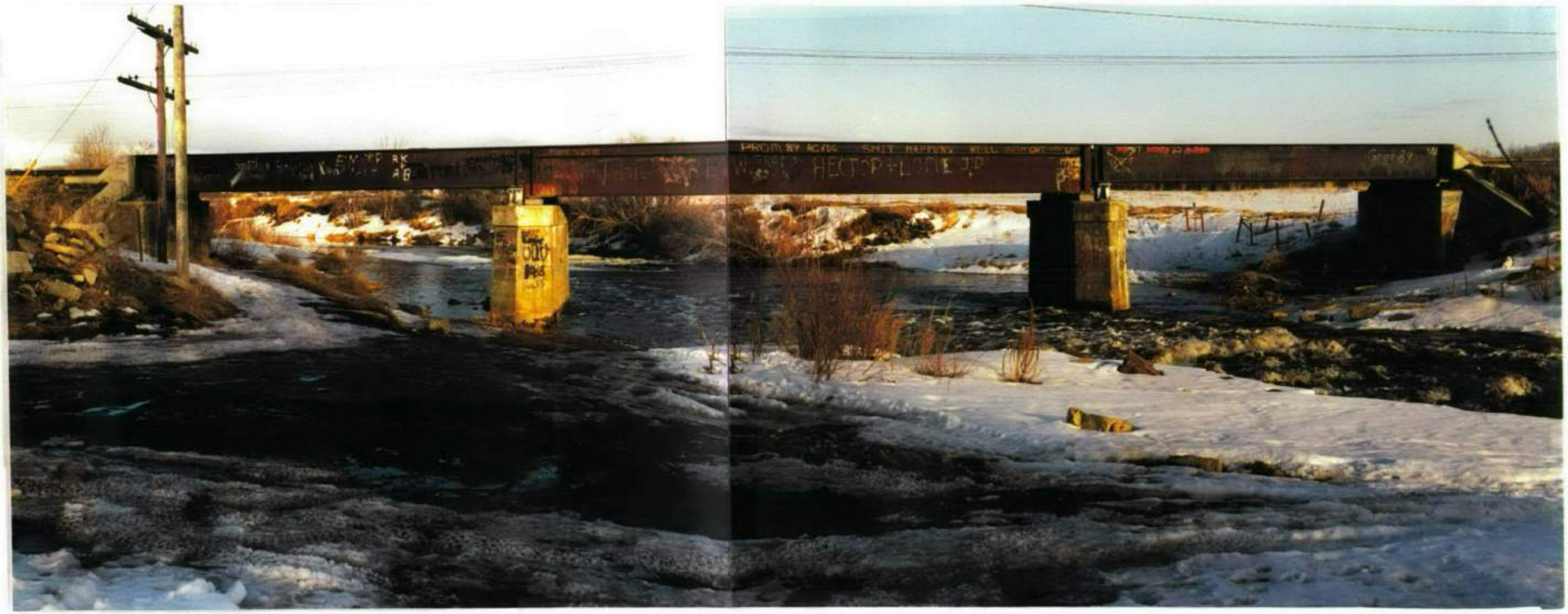

PLATE B-4 C.P.R. BRIDGE (SECTION 17)
LOOKING DOWNSTREAM, MARCH 3, 1992

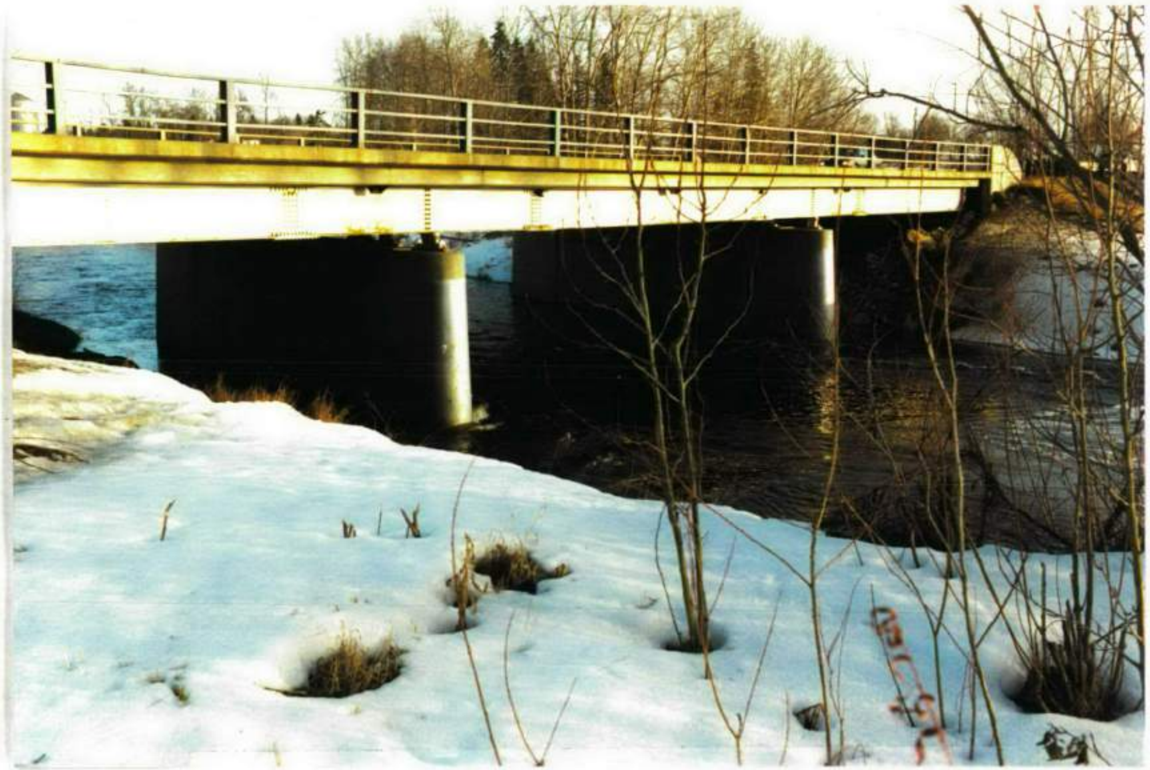

PLATE B-5 HIGHWY 2A BRIDGE (SECTION 21)
LOOKING DOWNSTREAM, MARCH 3, 1992

APPENDIX C

Comparison of Annual Maximum Stages for Ice and Open Water Conditions

This appendix compares maximum annual stages for ice and open water conditions. This comparison checks if flooding under ice conditions should be considered for determining the 1:100 year design flood.

Maximum instantaneous stage for ice conditions during spring breakup can be significantly greater than the corresponding mean daily stages. There may also be some differences between maximum instantaneous and mean daily stages during open water conditions. Unfortunately, maximum instantaneous stages and discharges are only available for a few of the Water Survey of Canada flow records. It was assumed for this analysis that the difference between maximum instantaneous and mean daily stages for open water conditions is small. Therefore, comparisons of ice versus open water condition stages could be made regardless of the type of measurement.

Annual maximum stages under ice and open water conditions are listed in Table C-1. Priority was given to listing maximum instantaneous values. If these were not available, mean daily values are provided. As indicated in the main report in Table 5, the gauge has been located at four main sites. The moves between gauge sites are also reflected in the data shown in Table C-1.

Some observations that may be made from this information include:

1. Ice condition stages are higher than open water stages in 21 of the 39 years of record (to 1990). These years will be referred to as +ice years in the further discussion.

2. The maximum difference between stages for +ice years was 1.3 m as recorded in 1928.

3. From Table 2 in the main report, the 1:2 year flood is 31 m^3/s and the 1:5 year flood is 83 m^3/s. Only six +ice years have flows greater than the 1:2 year flood and none of the +ice years have a flow greater than a 1:5 year flood.

4. At gauge site #1, the maximum stage under ice conditions was 4.00 m. This gauge height corresponds to less than the 1:5 year flood level for open water conditions at this location.

5. At gauge site #2, the maximum stage under ice conditions was 4.10 m. This gauge height corresponds to about the 1:5 year flood level for open water conditions at this location.

6. At gauge site #3, the maximum stage under ice conditions was 3.59 m. This gauge height corresponds to about the 1:5 year flood level for open water conditions at this location.

7. At gauge site #4, the maximum stage under ice conditions was 3.02 m. This gauge height corresponds to less than the 1:5 year flood level for open water conditions at this location.

Based on the above, recorded flood levels under ice conditions have not exceeded the 1:5 year open water flood levels. Larger flood events along this reach or river, with corresponding highwater levels, occur during open water conditions. Therefore, calculations of 1:100 year flood water levels should be based on open water conditions.

TABLE C-1
Comparison of Annual Maximum Stages for Ice and Open Water Conditions

Year	Ice Out Date[1]	Ice Conditions			Open Water Conditions		
		Gauge Height[2] (m)	Discharge[2] (m^3/s)	Date	Gauge Height[2] (m)	Discharge[2] (m^3/s)	Date
Gauge Site #1 (May 7, 1913 to April 30, 1925)							
1914	April 15	1.65	1.7	April 15	4.26	55.5	June 10
1915	April 7	2.67	1.8	March 24	3.81	55.8	June 15
1916	April 15	2.56	14.4	March 27	3.90	58.9	Sept. 8
1917	April 13	4.00	56.6	April 13	4.14	59.5	April 14
1918	April 12	2.40	2.3	March 29	1.34	7.1	April 14
1919	April 9	2.18	12.4	April 8	2.55	28.3	April 14
1920	April 28	3.96	63.7	April 28	4.63	90.9	May 9
1921	April 18	2.81	32.3	April 16	1.55	11.0	April 19
1922	April 9	1.35	0.0	February 22	0.82	2.4	June 6
1923	April 18	2.78	0.7	April 13	2.63	6.3	July 11
1924	April 28	1.31	1.9	April 17	1.08	3.4	April 29
1925	April 8	2.69	0.2	February 6	3.89	55.2	April 10
Gauge Site #2 (May 1, 1925 to 1932)							
1926	April 8	2.18	2.2	March 25	2.48	46.4*	June 23
1927	April 25	4.10	74.8	April 18	3.94	78.4*	July 10
1928	April 21	3.34	53.0	March 26	2.04	23.8	June 2
1929	April 24	1.36	8.7	April 18	1.25	7.4	April 2
1930	April 2	0.84	1.5	April 2	0.79	1.0	April 3

[1] Water Survey of Canada records indicate that ice was present in the river up to and including this date. For a given discharge, gauge readings will typically be lower for open water conditions compared to ice conditions.

[2] Gauge height and discharge data are given as mean daily readings except values marked with a asterisk. Values with an asterisk indicate maximum instantaneous gauge height or discharge.

TABLE C-1 (Cont.)

Year	Ice Out Date[1]	Ice Conditions			Open Water Conditions		
		Gauge Height[2] (m)	Discharge[2] (m^3/s)	Date	Gauge Height[2] (m)	Discharge[2] (m^3/s)	Date
Gauge Site #3 (June 16, 1966 to August 6, 1976)							
1967	April 26	2.07	17.4	April 26	1.83	15.3	April 27
1969	April 10	3.59*	66.0	April 10	3.13	56.4	April 11
1970	April 11	2.97*	28.3	April 11	2.49	31.7	April 12
1971	April 15	2.83	30.6	April 15	3.11*	55.5*	April 16
1972	April 13	2.83*	17.8	April 9	1.56	12.5	April 14
1973	April 7	2.05	13.3	April 4	2.52*	29.2*	July 4
1974	April 18	3.3	61.4	April 18	3.99*	108.*	April 19
1975	April 23	3.17*	20.0	April 22	2.1	19.2	April 24
1976	April 20	2.03	9.7	April 9	1.06	2.5	April 21
Gauge Site #4 (August 7, 1976 to Present)							
1978	April 10	1.70*	13.8	April 1	1.23	5.1	April 11
1979	April 20	1.55*	7.8	March 17	1.54*	15.6*	April 22
1980	April 14	1.6	12.1	April 11	1.74*	17.3*	July 4
1981	April 1	1.85	11.8	March 18	3.09*	65.3*	August 2
1982	April 21	2.5	35.4	April 21	3.34*	90.7*	April 24
1983	April 6	2.11*	19.2	April 5	1.61	15.8	April 9
1984	March 31	1.42	7.3	March 27	1.52*	11.1*	Sept. 25
1985	April 4	3.02*	54.5	April 4	2.62	48.0	April 5
1986	April 5	1.74	13.0	March 5	2.52*	47.5*	July 21
1987	April 7	2.19*	27.3	April 6	1.73	17.3	April 8
1988	February 29	0.84	0.1	January 6	2.6	50.0	July 9
1989	April 19	1.93*	33.4	April 15	1.46	11.9	April 20
1990	April 2	2.26	38.9	April 2	4.75*	287.*	July 4