Two-Dimensional Modeling of the Red River Floodway

by Ninel S. Gonzalez

A thesis submitted to the Faculty of Graduate Studies in partial fulfillment of the requirements for the degree of

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Abstract

The flood protection works (Shellmouth Reservoir, Portage Diversion and the Red River Floodway) were designed to protect the City of Winnipeg from a flood event with a peak discharge of 169,000 cfs. Control of Red River flows through Winnipeg is provided by forcing Red River waters to flow into the floodway channel. Records show that the peak discharge of a flood event (in Winnipeg) has always been due to a combination of high flows from the Assiniboine and Red Rivers. However, Red River flows have been historically higher (greater than 50%) than the contribution made by the Assiniboine River. The Red River Floodway is the component of the flood control works system with the greatest capacity and the one offering immediate flood relief to the City from the Red River floodwaters.

After the flood of 1997 the adequacy of the floodway to provide protection against a larger flood event became questionable. The role the floodway plays in providing flood protection to the City of Winnipeg cannot be emphasized enough. The safety of the City depends on the adequate and reliable performance of this channel.

A numerical model of the floodway channel has been established using the one-dimensional HEC-RAS and the two-dimensional FESWMS programs. The models were calibrated using water level measurements collected during the 1997 flood. The calibrated models were then used to evaluate the performance of the floodway channel under high flow conditions. Of particular interest were the flow corresponding to the maximum design discharge of the channel (Q = 100,000 cfs) and the flow associated with an 1826 flood event. The results determined the current maximum capacity of the channel and identified spill locations along the east and west dykes. The existing floodway channel does not have the capacity to convey the maximum design discharge. Alternatives to confine the flow within the channel and increase its capacity were also examined. These include raising of the dykes and use of composite cross sections throughout the channel. Results indicated that although some of these alternatives help confine the flow within the channel, they create higher water levels throughout, in particular at the inlet. The removal of bridges is an option that helps reduce water levels. However, the freeboard provided under such conditions is not considered to be adequate and therefore the possibility of dyke overtopping due to wind set up exists.

It is recommended that the design of the floodway channel be reconsidered in order to accommodate for higher flows and provide adequate flood protection to the City without sacrificing the areas upstream of the floodway.

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Nomenclature

A	$[L^2]$	cross sectional area
A_e	$[L^2]$	area of the element in which pier is located
A_p	$[L^2]$	below-water cross sectional area of the pier projected normal to the direction of the approaching stream flow
c_e, c_c	[-]	expansion/contraction coefficients
C	[-]	coefficient of discharge
C_d	[-]	pier drag coefficient
C_s	[-]	weir submergence coefficient
C_w	[-]	discharge coefficient for free weir flow
f	[-]	known function
F_d	$[ML/T^2]$	total drag force exerted by pier
g	$[L/T^2]$	gravitational acceleration
h	[L]	difference between energy gradient elevation upstream and the water surface elevation downstream
h_e	[L]	energy head loss
H	[L]	water depth
K_w	[-]	weir coefficient
L	[L]	discharge weighted reach length
L_w	[L]	length of the weir segment
$\underline{L}$	[-]	differential operator
n	[-]	roughness coefficient (Manning's)
q	$[L^3/T]/[L]$	discharge per unit of length
Q	$[L^3/T]$	total discharge
Q_w	$[L^3/T]$	total flow over weir
r	[-]	domain
R	[L]	hydraulic radius
S_f	[-]	friction slope
u	[L/T]	depth average velocity component acting in the x-direction
U	[-]	dependent variable
v	[L/T]	depth average velocity component acting in the x-direction
V	[L/T]	velocity

W	[-]	weighting function
Z	[L]	Elevation of the bottom of the channel
Z_c	[L]	crest elevation of weir segment
Z^h_e	[L]	elevation of the energy head at the upstream node
x,y,z		cartesian coordinate system
α	[-]	velocity weighting coefficient
α_x	[-]	arctan $[(\partial Z)/(\partial x)]$
α_y	[-]	arctan $[(\partial Z)/(\partial y)]$
α_z	[-]	arccos $(1-\cos^2\alpha_x-\cos^2\alpha_y)$
β	[-]	momemtum flux correction coefficients
ρ	$[M/L^3]$	water mass density
τ	$[M/T^2L]$	shear stress components due to turbulence
τ_b	$[M/T^2L]$	bed shear stress components
τ_s	$[M/T^2L]$	surface shear stress components
τ'_{bx}	$[M/T^2L]$	modified bed shear stress components (x-direction)
τ'_{by}	$[M/T^2L]$	modified bed shear stress components (y-direction)
ε	[-]	residual error
Ω	[-]	coriolis effect

CHAPTER 1

Introduction

The Red River originates in North Dakota and flows in a northward direction into Lake Winnipeg, in Manitoba. The river is very sinuous and has an average slope of approximately 0.01%. As the river flows toward Lake Winnipeg, it passes through several U.S. and Canadian urban centers: Fargo, Grand Forks, Winnipeg; as well as many rural communities.

The topography of the Red River valley is quite flat and therefore susceptible to flooding. The flatness of the area and its lack of natural barriers make the opportunities for natural storage non-existent. This has led to the implementation of flood control works to protect both urban and rural communities, especially those along the Red River.

In the United States flood protection works include dam structures and levee systems. In Canada, eight communities south of the City of Winnipeg are surrounded by ring dykes. These communities are: Emerson, Dominion City, Letellier, St. Jean Baptiste, Morris, Rosenort, Brunkild and St. Adolphe. A building in the Red River valley that is outside of these communities is required to be protected by ring dykes or to be at a specified elevation. Prior to 1997, the level of protection required was 1979 flood level plus 2 ft of freeboard. Additional ring dykes just south of the City are being considered as future flood protection measures.

The City of Winnipeg, the capital and economic center of Manitoba, has a population of 676,432 (Statistics Canada as of July 1, 1998). It is protected by a flood control system consisting of the Red

River floodway, the Portage Diversion, the Shellmouth Dam and Reservoir, and primary and secondary dyking systems.

During a 10 year span (1962-1972) work was undertaken for the implementation and completion of the different components of the flood works system. The system was designed for a maximum capacity of 169,000 cfs, an event thought to have a return period of 160 years. Figure 1.1 shows the location of the 3 major components of the system.

The following is a brief description of the different components making up this system. The years of construction are shown in brackets.

The Red River Floodway (1962 – 1968)

An excavated channel, approximately 29 miles in length that leaves the Red River upstream of the City of Winnipeg (at St. Norbert), and diverts a portion of the Red River east of the city. The diverted flow joins the river downstream of Winnipeg in the Lockport area.

The Portage Diversion (1965 – 1970)

Is an 18-mile long channel with a design capacity of 25,000 cfs. The channel diverts flows from the Assiniboine River, upstream of Portage la Prairie, into Lake Manitoba. The inlet and outlet drop structures regulate flow through the diversion channel. The diversion of water from the Assiniboine River reduces the flow into the City of Winnipeg.

The Shellmouth Dam and Reservoir (1964 – 1972)

It is located in the upper end of the Assiniboine River, near the Saskatchewan – Manitoba border. This earthen dam is 75 ft high and 4,000 ft long. The dam provides a reservoir with an approximate storage capacity of 500,000 acre-ft and provides both water supply and flood reduction benefits for areas along the lower Assiniboine River.

The Primary and Secondary Dyking Systems

The primary dykes are permanent earthen dykes built along the entire reach of the Red River (through the City of Winnipeg), and portions of the Assiniboine and Seine Rivers. The primary line of defense consists of approximately 110 km of primary dyke with a minimum elevation of 26.5 ft at James Avenue.

The secondary dykes can be either permanent or temporary and protect those properties outside of the primary dyking system. The secondary dykes are usually situated on private property. The elevation of the primary dyking system limits Red River flows through the City of Winnipeg to about 77,000-80,000 cfs if a 2 ft of freeboard is to be maintained.

The Red River Floodway offers immediate flood relief to the City of Winnipeg from Red River floodwaters. It is the component of the flood control works system with the greatest capacity. The following section provides a more detailed look at the Red River floodway channel.

1.1 The Red River Floodway

The purpose of the Red River Floodway is to ensure that Red River flows through the City of Winnipeg will remain within the river channel. Excess flows are diverted around the city through the floodway channel. As previously stated, the floodway has an approximate length of 29 miles. It leaves the Red River just south of the city (in the St. Norbert area), bordering Winnipeg to the east of Transcona and re-enters the Red River downstream of the city (in the Lockport area).

The diversion of water from the Red River into the floodway is controlled by the inlet control structure. Located just downstream of the floodway inlet, the control structure spans across the Red River and consists of two radial gates, separated by a concrete pier. The gates are approximately 112.5 ft long, 34.8 ft high, and are stored in the submerged position. The gates are operated so as to maintain 'natural water level conditions' upstream of the floodway.

The inlet plug is an earthen weir located at the entrance of the floodway. It is approximately 27 ft above the bottom of the river channel. During flood flows, water from the river flows (naturally) over the plug and into the floodway. The 7-ft high plug prevents ice from entering the floodway, therefore mitigating ice jams in the channel.

The floodway channel is grassed, with an average top width ranging from 700-1,000 ft and has an average depth of 30 ft. The cross sectional dimensions of the channel are quite uniform, except for the length passing through the Birds Hill area. The channel, through this natural depression, becomes narrower and deeper. Under design conditions the floodway channel will carry a total flow of 60,000 cfs, with an approximate water level of 770.25 ft at the inlet and 752.21 ft at the outlet. Dykes extending to the east and west of the channel help maintain floodwaters within the channel, preventing the flow from inundating the adjacent low-lying farmland and eventually entering the City of Winnipeg. The dykes are grassed, with 1:6 (v:h) side slopes.

The outlet drop structure is located at approximately 1,200 ft upstream of the Red River – Floodway junction. The structure consists of a concrete spillway with a stilling basin. There is approximately an 18 ft water surface drop in the floodway channel (from the inlet to the outlet). The Red River drops approximately 32 ft at the point where water flows from the floodway into the Red River. The outlet drop structure helps to minimize erosion problems by controlling the upstream water level and lowering velocities in the channel.

The floodway channel is crossed by 13 railway and highway bridges; these are as follows:

- i) St. Mary's Road (PR 200) bridge,
- ii) CPR Emerson bridge,
- iii) PTH 59 (south) bridge,
- iv) CNR Sprague bridge,
- v) PTH 1 bridge,
- vi) Greater Winnipeg Water District (GWWD) bridge,
- vii) PTH 15 bridge,
- viii) CNR Redditt bridge,
- ix) CPR Keewatin bridge,
- x) PTH 59 (north) bridge,
- xi) CPR Lac Du Bonnet bridge,
- xii) CNR Pine Falls bridge, and
- xiii) PTH 44 bridge.

The approximate location of these crossings is given in figure 1.2.

1.2 Objectives

Since its construction the Red River floodway has proven to be very valuable in providing flood protection for the City of Winnipeg. The importance of having a good understanding of the capabilities and limitations of the system cannot be underestimated.

In 1997 the floodway was put to the test: diverting a total of 65,000 cfs; a significant increase from its previous highest peak discharge of 39,800 cfs in 1979. There is the very real possibility that a flood event of a larger magnitude can occur. For example the flood of 1826, the largest flood on record, had an approximate flow at the confluence of the Assiniboine and Red Rivers of approximately 225,000 cfs. Given that the largest flood on record is substantially larger than the 1997 flood, which under natural conditions would have been 162,000 cfs, this raises serious questions about the ability of the existing infrastructure to deal with a flood larger than 1997. Therefore the objectives of this thesis are:

i) to develop a calibrated two-dimensional finite element model of the Red River Floodway,

ii) to investigate the maximum capacity of the floodway channel,

iii) to identify the location and magnitude of spills associated with flows larger than the maximum capacity,

iv) to investigate possible means to increase the channel capacity.

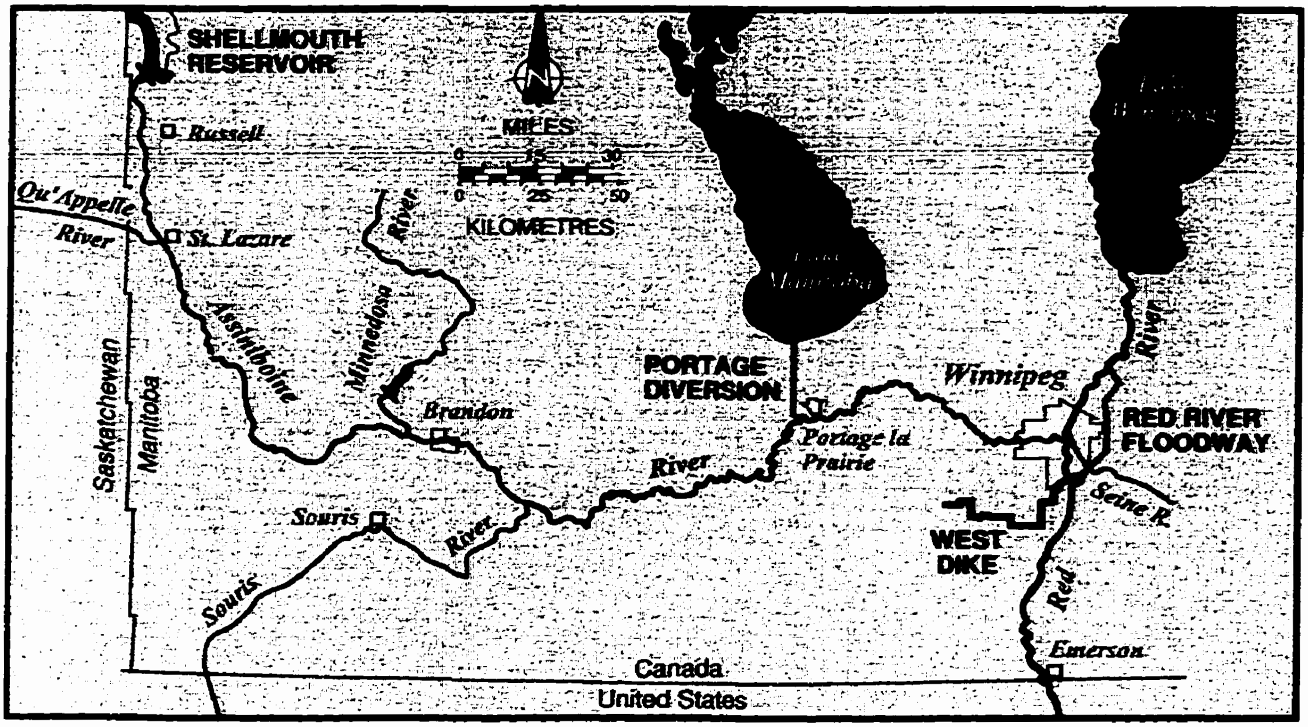
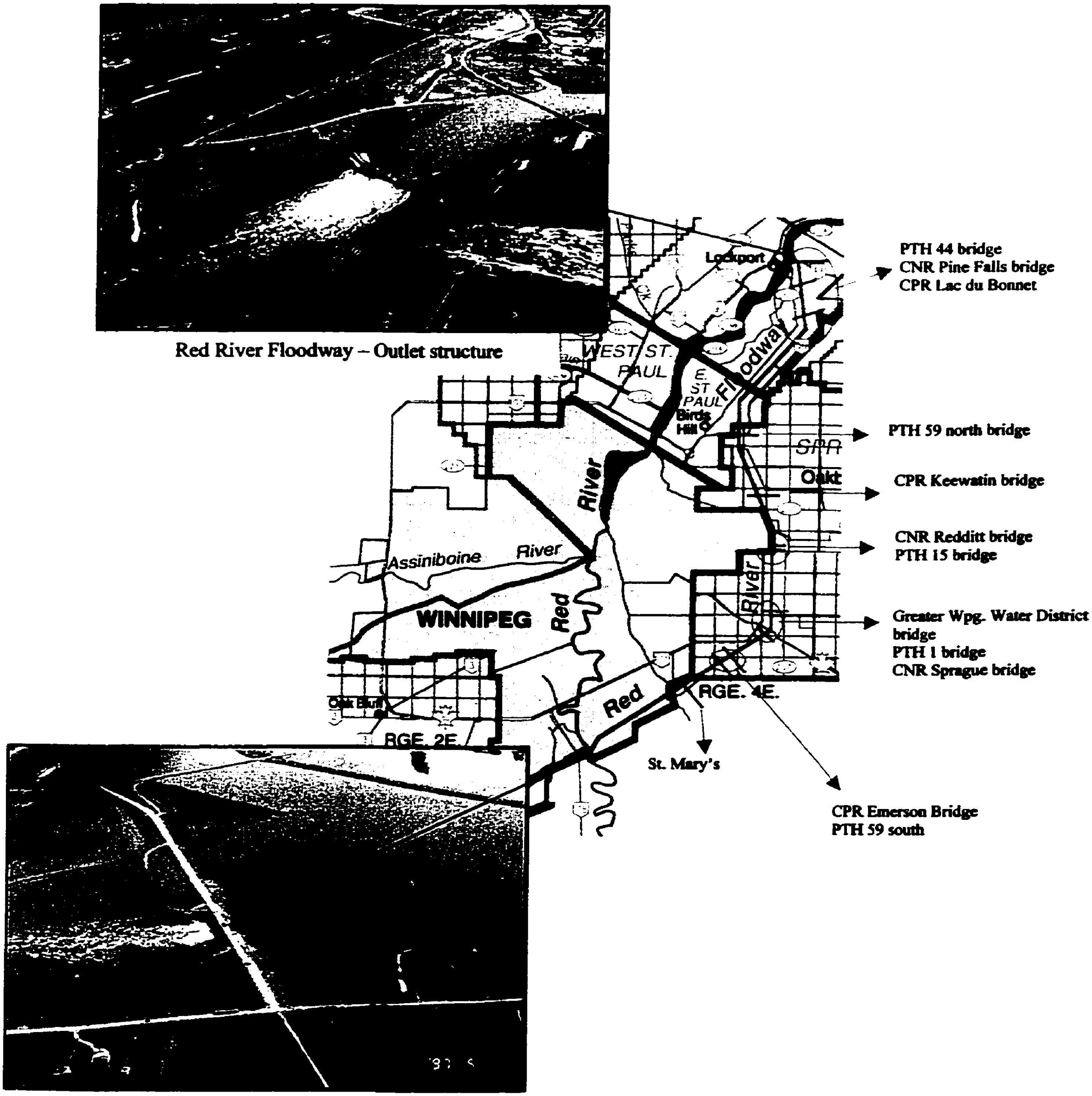


Figure 1.1 City of Winnipeg Flood Control Works (Manitoba Water Resources Branch).



Red River Floodway – Outlet structure

Red River Floodway – Inlet structure

Figure 1.2 The Red River Floodway.

CHAPTER 2

Numerical Models

The hydraulic analysis of the Red River floodway was carried out using two computer modeling programs: HEC-RAS and SMS (FESWMS). Although quite different in their capabilities to simulate flow in an open channel, both programs are interactively and user friendly. The following sections in this chapter review the theoretical background underlying each program, summarizing briefly their characteristics, capabilities, and limitations.

2.1 One-dimensional Modeling: HEC-RAS

HEC-RAS is a one dimensional analysis tool for steady state open channel flow. The program, *H*ydrologic *E*ngineering *C*enter's *R*iver *A*nalysis *S*ystem, HEC-RAS, was developed at the US Army Corps of Engineers Hydrologic Engineering Center and comprises a graphical user interface, data storage and management capabilities, and hydraulic analysis components (US Army Corps, 1997). HEC-RAS performs, under steady state conditions, water surface profile computations in all three flow regimes: subcritical, supercritical and mixed.

2.1.1 Applied Equations

The computational component of the system performs water surface profile calculations based on the Standard Step Method. This method is a trial and error procedure, which uses the energy equation between two cross sections, a distance d apart, to solve for the water surface elevation at the upstream end of the reach.

The energy equation between two cross sections of a given channel is described by the following equation. A graphical representation of equation 2.1 is given in figure 2.1.

$$Z_2 + H_2 + \frac{\alpha_2 V_2^2}{2g} = Z_1 + H_1 + \frac{\alpha_1 V_1^2}{2g} + h_e, \tag{2.1}$$

where Z refers to the elevation of the bottom of the channel relative to some datum, H is the water depth, g is the gravitational acceleration, V is the average velocity to which a correction factor, α, is applied. The latter (*i.e.*, α) is also referred to as a velocity distribution coefficient. h_e is the total energy head loss. The subscripts 1 and 2 refer to the downstream and upstream sections, respectively.

HEC-RAS evaluates head losses in terms of losses due to friction, as well as expansion and contraction of the flow in between cross sections. The total head loss is given by

$$h_e = \underbrace{LS_f}_{\text{Term 1}} + c\underbrace{\left|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right|}_{\text{Term 2}} \tag{2.2}$$

In the above equation, L refers to the discharge weighted reach length, S_f is the friction slope and c is the expansion (c_e)/contraction (c_c) loss coefficient.

Frictional losses (term 1 in equation 2.2) are evaluated using an average of the friction slope of the two cross sections. The friction slope is obtained from Manning's equation, *i.e.*,

$$S_f = \sqrt{\frac{Qn}{1.489AR^{2/3}}}, \tag{2.3}$$

where Q is the total discharge, n is the roughness coefficient (Manning's n), A refers to the cross sectional area, and R is the hydraulic radius. The value 1.489 is a factor used for the application of Manning's equation in imperial units.

The program evaluates the absolute difference in velocity heads between the upstream and downstream cross sections in order to apply a contraction or expansion coefficient. A positive difference in velocity head (term 2 in equation 2.2) would indicate a greater velocity at the upstream section. This in turn would indicate a flow expansion. A negative difference in velocity head would indicate a greater velocity at the downstream section at which point the program assumes a flow contraction. Typical values for the expansion (c_e) and contraction (c_c) coefficients are presented in table 2.1.

Table 2.1 Expansion/contraction coefficients under subcritical conditions (US Army Corps, 1997)

Conditions	c_c	c_e
Gradual transition	0.1	0.3
Typical bridge cross section	0.3	0.5
Abrupt transitions	0.6	0.8

HYDRAULIC SIMULATION AT BRIDGES

HEC-RAS has the capability to model two types of flow: low flows and high flows, as well as any combination that may occur in between such as pressure flow, pressure flow and weir flow.

The HEC-RAS reference manual defines low flows when "the water surface elevation is below the highest point on the low chord of the bridge opening". Conversely, high flows are defined as flows when the water surface elevation is greater than the maximum low chord of the bridge deck. Bridges are modeled using either the Energy equation, the momentum balance method, the Yarnell equation, the FHWA WSPRO method, or the pressure and weir flow method. The latter method is applicable to high flow conditions.

Under high flow conditions, the discharge at a bridge structure is evaluated by the following equation

$$Q = CA\sqrt{2gh}\,, \tag{2.4}$$

Where Q is the total flow over the weir, A is the area under the bridge opening, h is defined as the difference between the energy gradient elevation upstream and the water surface elevation downstream, and C is the coefficient of discharge.

2.2 Two-dimensional Modeling: FESWMS

The two-dimensional computer package used in the analysis of the floodway channel is SMS: *S*urface-water *M*odeling *S*ystem v5.08. SMS is a pre-&-post processor for surface water modeling and analysis (SMS Reference Manual, 1997) developed by the Engineering Computer Graphics Laboratory at Brigham Young University in cooperation with the U.S. Army Corps of Engineers, the Waterways Experiment Station (WES), and the U.S. Federal Highway Administration (FHWA). The SMS system is comprised of several hydrodynamic modeling programs. As a pre-processor, SMS allows for the construction of the mesh to be modeled. As a post-processing tool, SMS can be used to generate plots of the solution files, *e.g.*, color-shaded contour plots for water surface elevations, velocity vector plots, etc.

The interfaces currently supported by SMS are RMA2/RMA4, which are part of the TABS numerical modeling package; SED2D-WES, a sediment transport numerical model; WSPRO, a one-dimensional water surface profile computational program, and FESWMS a hydrodynamic modeling program which supports both supercritical and subcritical flows. The latter was used to carry out the hydrodynamic analysis of the Red River Floodway.

FESWMS is an abbreviation for *F*inite *E*lement *S*urface *W*ater *M*odeling *S*ystem, which was developed by Dr. Dave Froelich, for the U.S. Federal Highway Administration. The program uses finite elements to model surface water bodies where the vertical accelerations are very small in comparison to the horizontal movement (FESWMS User's Manual, 1997). The analysis engine of the FESWMS package is the FLO2DH program.

SMS provides the modeler with the necessary tools to create the mesh to be analyzed, to incorporate flow control structures such as weirs, culverts and to import the solution files for a graphical view of the results. The following sections review some of the theoretical concepts on which the FESWMS program is based. Data requirements and some general guidelines used in the construction of the mesh are also presented.

2.2.1 The FLO2DH Module – General Overview

The depth averaged flow module, FLO2DH, solves the set of equations describing two-dimensional surface water flow in a horizontal plane. The module processes the input data through three major functions before generating an output. These major functions are:

a) reading of all input data,

b) determining a steady state solution (steady state simulation),

c) determining a time dependent solution (dynamic simulation).

This process is shown in figure 2.2.

The program checks all input data (geometric data, initial boundary conditions, element property data) to ensure there is compatibility with the array dimensions. The depth averaged flow equations are then solved, for either the steady or time-dependent case. Continuity norm computations are also carried out. Mass flux comparison between cross sections can be used as a way to check whether the law of conservation of mass is being satisfied. The information regarding the characteristics/properties of the mesh to be modeled is contained in the input data files (element and node numbers, node connectivity, boundary data, etc).

The program solves the set of differential equations by using the Galerkin finite element method. The solution is approximated by using mixed interpolation. Because the approximations do not satisfy exactly the original set of equations the method of Weighted Residuals is applied. The Galerkin finite element method assumes the weighting functions to be the same as the interpolation functions. A description of the Finite Element Method is provided in Appendix A.

2.2.2 Applied Equations

DEPTH AVERAGED FLOW EQUATIONS

The equations that describe the two-dimensional surface water flow are based on the vertically integrated conservation of mass and momentum equations, with the assumption that the vertical motion (velocity and acceleration) is negligible. These three equations are generally referred to as the "shallow water equations."

The depth-averaged flow equations are (FESWMS User's Manual, 1997)

$$\frac{\partial(Hu)}{\partial t}+\frac{\partial}{\partial x}\left[\beta_{uu}Huu+(\cos\alpha_x\cos\alpha_z)^2\frac{1}{2}gH^2\right]+\frac{\partial}{\partial y}(\beta_{uv}Huv)+\cos\alpha_x gH\frac{\partial Z}{\partial x}-\Omega Hv+\frac{1}{\rho}\left[\tau_{bx}-\tau_{sx}-\frac{\partial(H\tau_{xx})}{\partial x}-\frac{\partial(H\tau_{xy})}{\partial y}\right]=0 \tag{2.5}$$

$$\frac{\partial(Hv)}{\partial t}+\frac{\partial}{\partial x}(\beta_{uv}Hvu)+\frac{\partial}{\partial y}\left[(\beta_{vv}Hvv)+(\cos\alpha_y\cos\alpha_z)^2\frac{1}{2}gH^2\right]+\cos\alpha_y gH\frac{\partial Z}{\partial y}+\Omega Hu+\frac{1}{\rho}\left[\tau_{by}-\tau_{sy}-\frac{\partial(H\tau_{xy})}{\partial x}-\frac{\partial(H\tau_{yy})}{\partial y}\right]=0 \tag{2.6}$$

$$\frac{\partial H}{\partial t}+\frac{\partial(Hu)}{\partial x}+\frac{\partial(Hv)}{\partial y}=q\,, \tag{2.7}$$

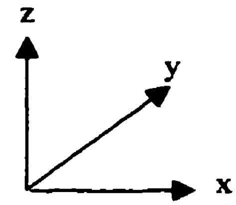
where ρ is the water mass density, q refers to unit source (inflow) or unit sink (outflow), H is the water depth, u and v are the depth averaged velocity components acting in the x and y direction, respectively. Z is the bed elevation, the symbol Ω refers to the Coriolis effect, β is a Momentum flux correction coefficient, τ is the shear stress components due to turbulence, with the subscript b referring to the bed shear stress component and s to the surface shear stress component. The α terms are defined as

α_x: arctan [$(\partial Z_b)/(\partial x)$]

α_y: arctan [$(\partial Z_b)/(\partial y)$]

$$\alpha_z\text{: } \arccos\left(1-\cos^2\alpha_x-\cos^2\alpha_y\right)$$

x,y and z are the cartesian coordinates with x and y lying in a horizontal plane and z pointing vertically upward.



Equations (2.5) and (2.6) are the vertically integrated momentum equations in the x and y direction, respectively. The momentum equations are expressed in terms of inertial and pressure forces, shear stresses due to turbulence, surface and bottom stresses, as well as the coriolis effect. Equation (2.7) is known as the vertically integrated mass transport equation or the continuity equation. It states that a change in the volume of water within a small 'imaginary' control volume must be balanced by the net change in flow rate.

WEIR FLOW EQUATION

The FESWMS User's Manual notes that the assumption of negligible vertical motion cannot be applied when weir-like structures (roadway embankments) are present. Such structures may result in large vertical velocities that cannot be ignored and therefore a more accurate solution can be obtained by applying a 1-D empirical equation.

The equation used to calculate flow over a weir segment is presented as

$$Q_w = K_w(Z_e^h - Z_c)^{1.5}, \tag{2.8}$$

where Q_w is the flow over weir, K_w is the weir coefficient, Z_e^h is the elevation of the energy head at the upstream node and Z_c refers to the crest elevation of weir segment.

The weir coefficient K_w is expressed in terms of

$$K_w = C_s C_w L_w \sqrt{g}\ . \tag{2.8a}$$

In the above equation C_s refers to the weir submergence coefficient, C_w is a discharge coefficient for free weir flow, and L_w length of the weir segment. Typical values for the weir coefficients are given in the FESWMS User's Manual (1997).

Bridge Piers

The effect of bridges during low flow conditions (*i.e.*, water level below the low bridge chord) is taken into account by incorporating bridge piers into the model. The FLO2DH module adjusts the friction coefficient of any element containing bridge piers by adding a drag force. The added piers also result in additional shear stresses, which are taken into account by modifying the element's bed shear stress. The drag force and modified shear stresses are calculated as follows (FESWMS User's Manual, 1997).

Drag force

$$F_d = \frac{1}{2} C_d \rho v^2 A_p . \tag{2.9}$$

Modified shear stresses

$$\tau_{bx}^{'} = \tau_{bx} + \frac{A_p}{2A_e} C_d \rho u \sqrt{u^2 + v^2} \tag{2.10a}$$

$$\tau_{by}^{'} = \tau_{by} + \frac{A_p}{2A_e} C_d \rho v \sqrt{u^2 + v^2} . \tag{2.10b}$$

C_d is the pier drag coefficient, A_p is the below-water cross sectional area of the pier projected normal to the direction of the approaching steamflow, and A_e is the area of the element in which the pier is located.

2.2.3 Creating the mesh

The topography of the water surface body to be modeled needs to be represented as a two dimensional finite element mesh. The mesh consists of nodes, from which elements are generated.

Each node in the mesh is described by an *xyz* coordinate system. The *x* and *y* values position the node with respect to some point of reference, while the *z* value indicates the ground elevation at that particular node.

Creation of elements can be done either automatically by the program using the triangulation option or manually by having the modeler create the elements by connecting nodes. SMS supports both linear and quadratic elements. Examples of linear and quadratic elements are shown in figure 2.3.

The selection of the size, shape, and type (linear vs. quadratic) of elements to be used depends on how much detail needs to be represented and the degree of accuracy required. For example, quadratic elements require more computational effort, but provide more accurate solutions than linear elements. Through experience the modeler learns to create the elements within a mesh that will best describe the area of study without sacrificing accuracy and computational efficiency. Stability of the model can be affected by the geometry of the elements (*viz.*, aspect ratio, internal angles). When preparing the layout of the network the size of the elements becomes important. This will be dictated by the aspect ratio being used. The aspect ratio is defined as the ratio of the longest element dimension to the shortest element dimension (FESWMS User's Manual, 1997). Although the elements making up a mesh may have different aspect ratios, the FESWMS manual recommends keeping it below 12.5. This restricts the internal angles to be kept between 5^0 and 120^0. The above can be applied to both triangular and rectangular elements.

The quality of the mesh is assessed by a number of criteria related to the geometry of the elements. These include proper aspect ratio, gradual area change between adjacent elements, wet/dry edges, smooth boundaries, and ambiguous gradients. The SMS User's Manual (1997), RMA2 section, provides a good description of the factors affecting the mesh quality. The program has the capability to check for ill-formed elements based on these criteria. The quality of any model, can be improved by increasing the mesh density around the problem. Also uniformity among elements may result in an easier constructed mesh. However, this may prove to be impractical and inefficient in terms of the computational requirements.

Figure 2.4 shows a section of the floodway channel as depicted by a two-dimensional mesh. The mesh is a combination of triangular and rectangular quadratic elements (shaded in red). Boundary conditions are applied along a node string (green nodes) across a cross section.

It has been emphasized that the stability of the model is affected by the quality of the mesh. The judgement of an experienced modeler as to the selection of the right elements to provide a mesh conforming to all of the mesh quality criteria is recognized as a valuable tool to provide a stable model.

2.2.4 Assigning Model Parameters

The FESWMS program allows the modeler to identify the conditions under which the network is being modeled. Some of these include type of flow (subcritical, supercritical, steady state or dynamic), Manning's n, and boundary conditions.

The assigned boundary conditions can consist of a discharge or water level at either the upstream or downstream ends of the channel. Material types are identified in terms of bed friction coefficients and eddy viscosity parameters. Bed friction coefficients can be expressed in terms of the Chezy's coefficient or Manning's n. The eddy viscosity coefficients represent the turbulent stresses acting on a fluid element due to the exchange flow. These coefficients were first defined by Boussinesq as the constant of proportionality in his assumption that the turbulent stresses are directly proportional to the mean velocity gradient (Froehlich, 1987). The concept of the eddy viscosity coefficients is similar to the viscosity μ in laminar flow. While the latter is a function of the fluid, the eddy viscosity coefficients depend not only on the fluid but on its mean velocity as well.

The program is capable of taking into account wind conditions and the coriolis effect. For the wind conditions a direction and speed are required. The wind speed may be constant or transient, depending on the type of solution being run (*i.e.*, steady state or dynamic). The coriolis parameter accounts for the effect of the earth's rotation on the movement of water. The above two conditions were not considered in the analysis of the Red River Floodway model and are only mentioned to provide a more complete description of the FESWMS program.

After all the conditions have been assigned and all parameter values entered, the program can run a check to make sure that all data are consistent and that the model to be run is free of errors.

2.2.5 Running the model

Obtaining the final solution of a network may not be achieved during the first simulation. The FESWMS manual suggests that initial conditions (starting values) need to be specified at the beginning of a run. For the first run, the model requires an initial constant water surface elevation to be assigned. If the desired solution is not achieved the modeler proceeds through an iterative process where the results from a previous run can be used as initial conditions for a second run. It has also been found that sometimes it is necessary to start with boundary conditions that are slightly different from the true ones. The boundary conditions can then be changed gradually until reaching the desired conditions. This procedure can be helpful particularly with large meshes (*i.e.*, with large numbers of nodes and elements) that do not converge readily on the first run.

During any given run, elements within the mesh may become dry (*i.e.*, the nodal elevation is greater than the water surface elevation specified). The turning on/off of elements is performed by the automatic boundary adjustment option. Through this option the program compares the nodal elevations of every node forming an element to the water surface elevation in order to decide whether the element is wet or dry. Dry elements are turned off and are not included in the computations of the network.

Convergence problems are largely due to geometric inconsistencies, but can also be present when there is a large difference between the initial conditions and the assigned boundary conditions. For the latter, convergence problems can be eliminated by using intermediate solutions, *i.e.*, finding those conditions which would provide a stable model and proceeding from there on. Elimination of geometric inconsistencies requires a thorough revision of the mesh, but can also include selective mesh refinement.

Convergence of the model depends greatly on the layout of the network, on how accurately the network represents actual physical conditions, as well as on the specified initial and boundary conditions.

The results of a simulation consist of velocity, water surface elevation, and water depth at every node point in the network. The graphical interface of SMS allows the results to be imported for a visual analysis of the results. Velocities and water surface elevations can be shown as contour plots. The 2D mesh and the results can also be exported into other programs, such as Paint, with graphical capabilities for further editing for presentation purposes.

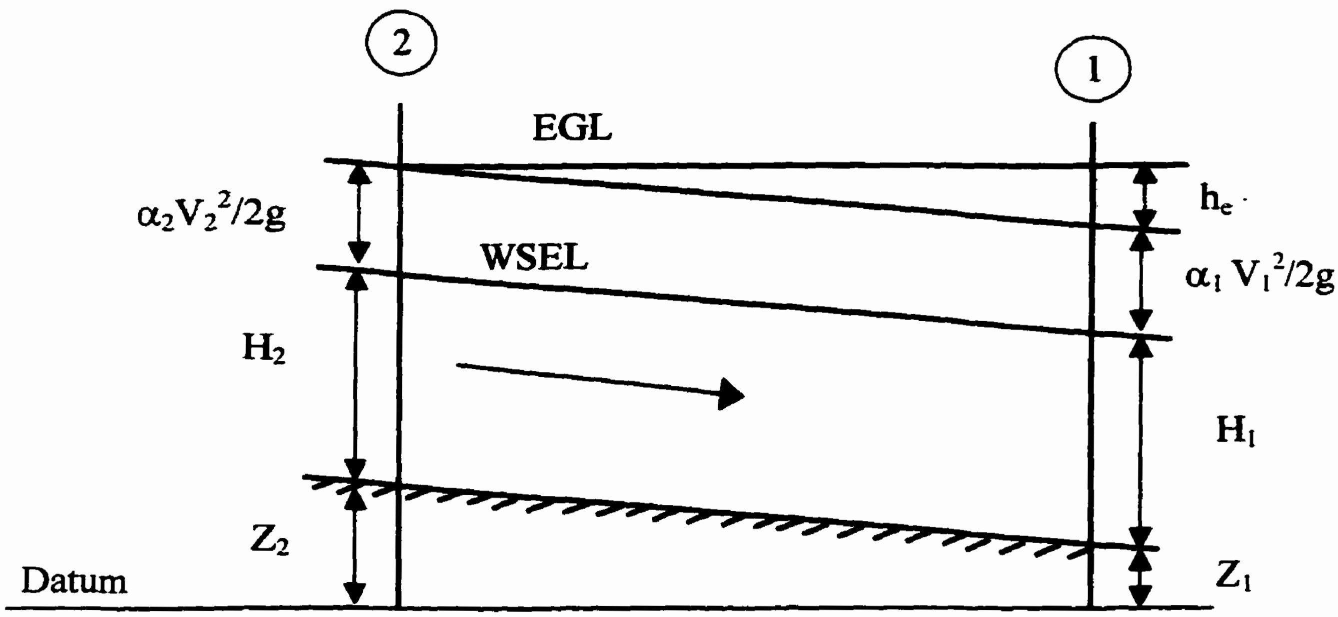


Figure 2.1 Schematic of the energy equation (Hoggan, 1989). Note that the subscripts 1 and 2 refer to the downstream and upstream ends of the section, respectively.

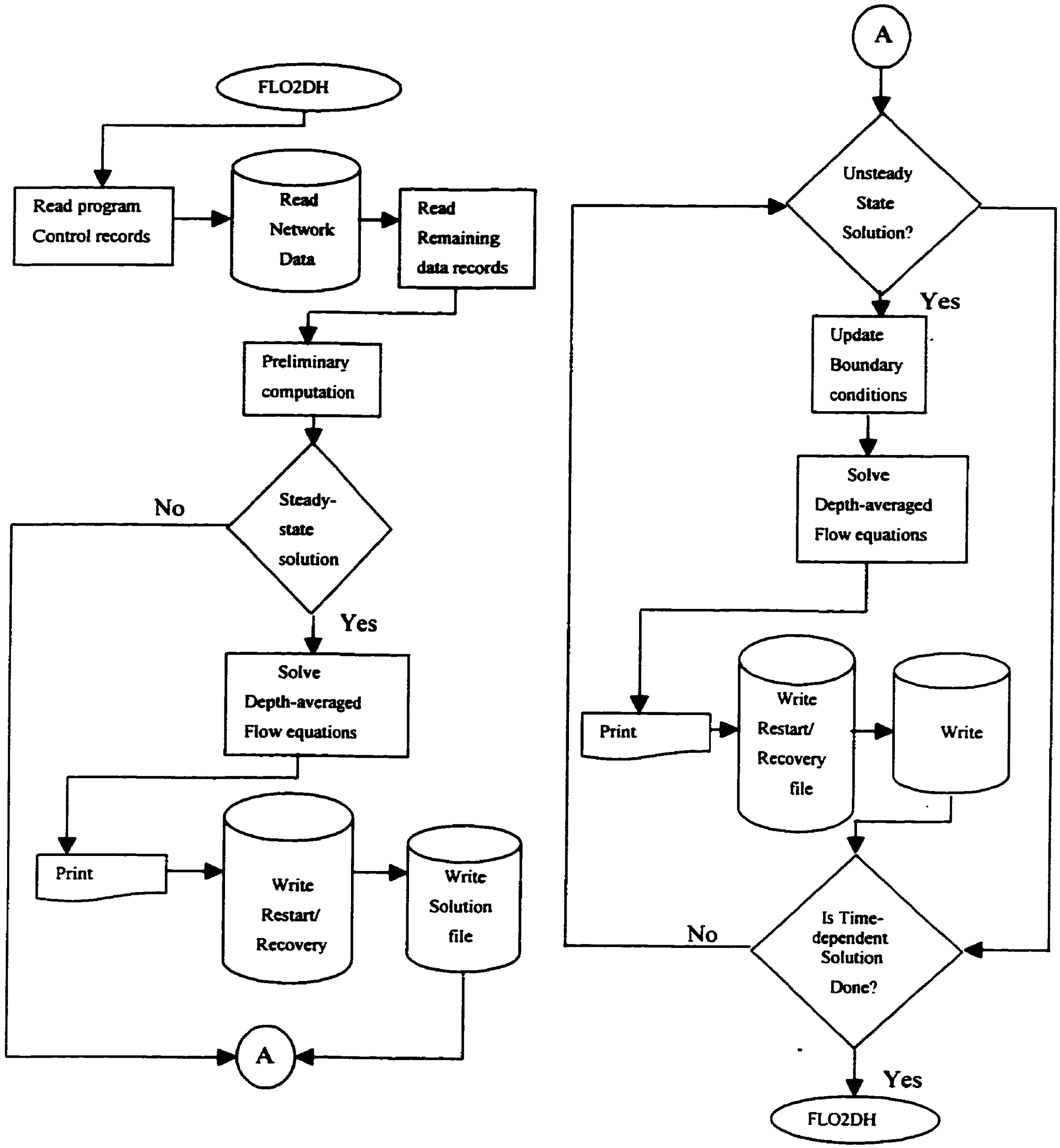


Figure 2.2 Schematic diagram of the FLO2DH module (FESWMS User's Manual, 1997).

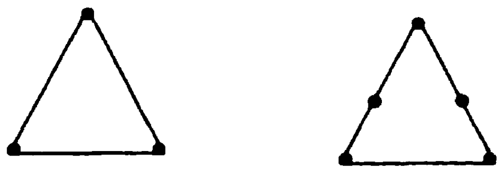


Figure 2.3 Three noded (linear) and six noded (quadratic) triangular elements.

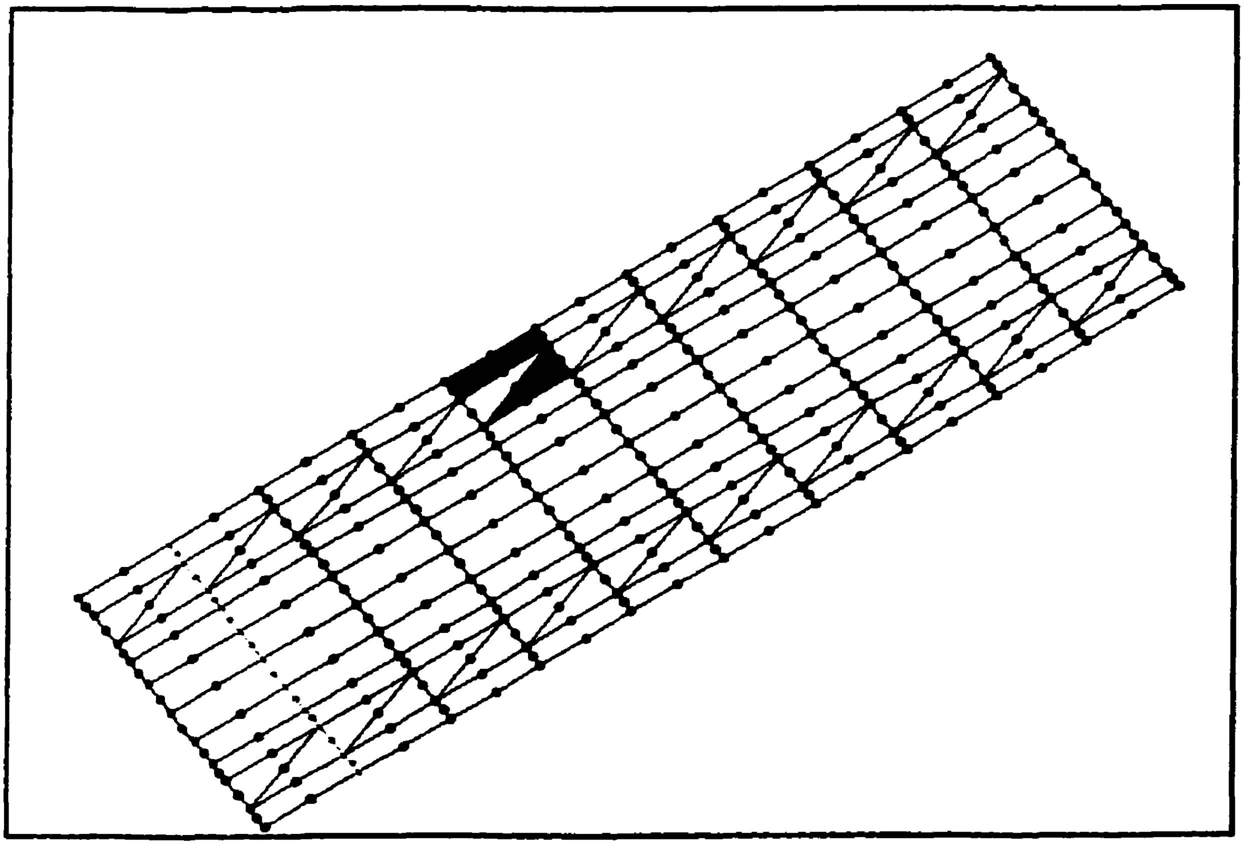


Figure 2.4 A sample portion of the two-dimensional Red River Floodway mesh.

CHAPTER 3

Model Formulation

3.1 Initial Constraints

The computer model of the Red River Floodway was set up in the one-dimensional (HEC-RAS) and two-dimensional (FESWMS) programs. The two dimensional model (FESWMS) imposed some limitations on the modeling of bridges. The version used to conduct the modeling for this research does not support pressure flow conditions. The modeling of small bridges can be accomplished, however, by defining weirs and culverts at the bridge cross section. The weirs, defining the bridge deck, can be used to simulate water going over the bridge, while culverts can be used to convey water under the bridge.

The model of the floodway channel incorporates 13 bridges. It was found that due to the size of the bridge openings a series of large culverts would be needed at each crossing. The use of culverts and weirs introduced an unconvergable instability into the model. Since FESWMS conducts the hydraulic analysis of weirs and culverts by using 1D empirical equations, it offers no advantage as to the simulation of bridge structures over HEC-RAS. Consequently, it was then decided to model the bridge structures in HEC-RAS and transfer the solution back into FESWMS to continue the two dimensional modeling of the channel.

The modeling of the floodway channel in the two dimensional program remained important for the simulation of high flows, where a high flow refers to a discharge of approximately 100,000 cfs or greater, and the possibility of spillage over the dykes would be present.

The Red River floodway model was simulated in HEC-RAS and FESWMS under steady state, subcritical flow conditions. Modeling of channels under steady state conditions is a justified and fair assumption since most Red River flood flows change quite gradually and slowly. Both programs were used interactively throughout the entire modeling process.

3.2 Data acquisition

The first step in any model formulation is data collection. For the Red River Floodway model data was obtained from the Manitoba Natural Resources Department, Water Resources Branch. The Department provided diagrams showing typical floodway cross sections, topographic details of the study area, details regarding bridge structures, as well as water level measurements recorded during the 1997 flood. The latter were used as the basis of comparison for the computed water levels during the calibration process.

East and west dyke profiles were provided by the Department of Highways. Top of dyke elevations were recorded using GPS (*G*lobal *P*ositioning *S*ystem) by Highway workers during the month of February 1998.

In addition, maps of the St. Norbert area, East of the City of Winnipeg and the LockPort area were also referenced to determine the layout of the floodway and approximate the location of bridges.

3.3 Model description

The Red River floodway model is described in terms of the floodway channel, the east and west dykes and bridge structures. The model consists of the channel itself, not including the junctions with the Red River in the upstream (St. Norbert) and downstream (LockPort) areas. The most upstream section is located at the inlet of the floodway channel, modeling the lip. The most downstream cross section corresponds to a location approximately 500 ft. upstream of the outlet structure.

3.3.1 Cross sectional data

Cross sections were generated based on the three typical cross sections found in the floodway: south of Birds Hill, through Birds Hill, and north of Birds Hill. Figure 3.1 shows the dimensions of a typical cross section found within the floodway channel. The low flow channel (hatched area in figure 3.1) was not included in the model. The dimensions of the low flow channel are very small and therefore its capacity is negligible when compared to the channel's overall carrying capacity. Another consideration for omitting the low flow channel was the positioning of the nodes and subsequent formation of elements in the mesh (2D model). For an accurate representation of the low flow channel four nodes would have been required, as shown in figure 3.2. Based on the spacing of the cross sections the resulting elements would have been too thin and long. Not conforming to the recommended aspect ratio, these elements would have introduced instability into the model.

Topographical data of the floodway cross sections were not available. Information provided by the Water Resources Branch was limited to the three typical cross sections as envisioned in the designing stages of the floodway channel. The topography of the channel cross sections was then created based on the dimensions of the typical cross sections (as shown in figure 3.1).

The bottom elevation of the channel at each cross section was calculated based on the following slopes:

Channel grade slope south of Birds Hill: 0.0000868

Channel grade slope north of Birds Hill: 0.00016

Having established the elevation at the bottom of the channel, the generation of each cross section was based on two additional points with known elevations: prairie elevation and dyke elevation. These were measured off channel and prairie profiles. Figure 3.3 shows the location of the three points with known elevation within a cross section. The cross sections were spaced at an average of 820 ft. A total of 187 cross sections were generated. These 187 cross sections were used to create the model of the floodway channel in both the HEC-RAS and FESWMS programs. Additional cross sections were generated as required in each model through linear interpolation.

3.3.2 Manning's n

Flow resistance is affected by form roughness and grain roughness. The former is associated with bedforms, which in turn are affected by vegetation, channel type and irregularities. In the model of the floodway channel, such roughness has been expressed in terms of Manning's n.

The Manning's n values assigned to the study area were obtained from the following table, the selected range of values used is shaded in gray.

Table 3.1 Values of Manning's n (Chow, 1959)

Type of channel	*Minimum*	*Normal*	*Maximum*
Excavated or dredged			
a. Earth, straight and uniform			
1. Clean, recently completed	0.016	0.018	0.020
2. Clean after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short grass, few weeds	**0.022**	**0.027**	**0.033**

3.3.3 Bridge data

Bridge data were obtained from the design drawings of railway and highway bridges (Manitoba Natural Resources, Water Resources Branch). It consisted of topography of the channel at the bridge cross section, high and low cord elevations and pier details (location, shape and dimensions).

3.3.4 Boundary condition data

Since the channel was being modeled as subcritical flow, the boundary conditions consisted of an inflow or total initial discharge assigned at the most upstream cross section and a water level assigned at the most downstream cross section. In order to determine the water level to be assigned as a downstream boundary condition two factors were considered:

i) the rating curve at the outlet structure, and

ii) the effect of the combined flows from the floodway and Red River on the outlet structure.

RATING CURVE AT THE OUTLET STRUCTURE

When the floodway channel was designed, a series of water surface profiles were developed for flows ranging from 5,000 to 100,000 cfs. Using the given water levels at the outlet structure for this range of flows a rating curve was developed. The rating curve at the outlet is shown on figure 3.4. The relationship shown is described adequately by a straight line. The adequacy of the fitted line is reflected in the coefficient of determination, R^2. In a linear regression analysis this coefficient is a measure of the variability of the data being examined. An $R^2 = 1$ indicates a perfect fit, *i.e.*, a 100% correlation of the data. Conversely, $R^2 = 0$ indicates a greater dispersion of the data and a poor representation of their variability. The linear fit is further confirmed by the field measurement made during the 1997 flood.

EFFECT OF COMBINED FLOWS FROM THE FLOODWAY CHANNEL AND RED RIVER ON THE OUTLET STRUCTURE

It was speculated that at higher flows (floodway flows greater than or equal to the maximum design capacity) there might be the possibility of a backwater effect resulting from the confluence of the floodway channel with the Red River. For "submerged" conditions at the outlet structure it would be necessary to determine the significance of such a backwater effect on water levels upstream of the structure. In order to investigate the importance of the tailwater on upstream levels a one-dimensional analysis of the confluence of the floodway channel and the Red River was carried out in HEC-RAS. The sections (of the floodway and Red River) that were modeled are shown in figure 3.5.

The Red River section consisted of several cross sections defining the river channel from approximately the St. Andrew's dam to the Lockport gauging station. The selection of the downstream cross section at the gauging station facilitated the task of determining the water levels at this section. The Lockport gauging station is located at approximately 2.01 km north and downstream of the St. Andrew's dam. It is equipped with a Stevens A-71 recorder, cableway with stand up aluminum cable car and motorized B reel. The Water Resources Branch monitors and collects water levels at this location and provided 1997 data from which a rating curve was developed to allow for the extrapolation of water levels at higher flows. Cross sections used for this section of the river were also obtained from the Water Resources Branch.

The section of the floodway included in this analysis consisted of the most downstream section of the channel (approximately 1,200 ft) from immediately downstream of the outlet structure to where the channel joins the river. Cross sectional data for this section were not available and therefore the dimensions assigned to these cross sections were assumed, based on those found at the outlet structure.

The analysis was carried out for a constant Red River discharge of 80,000 cfs (*i.e.*, flow from the City) and floodway discharges varying from 65,000 to 116,000 cfs. Table 3.2 summarizes the computed water levels at points A and B (figure 3.5).

Table 3.2 Backwater runs

$Q_{Red\ River}$ *(cfs)*	$Q_{Floodway}$ *(cfs)*	*WL at A (ft)*	*WL at B (ft)*
77,318.1	65,000	736.7	737.6
80,000	80,000	747.2	747.8
80,000	100,000	751.1	751.6
80,000	110,000	753.1	753.6
80,000	116,000	754.3	754.7

It should be noted that the HEC-RAS model for this section of the Red River and floodway channel was not calibrated. However, the Water Resources Branch has indicated that during the 1997 flood the tailwater level at the outlet structure was approximately 7-8 ft above the weir crest. The highest peak discharge recorded at the gauging station in 1997 was 142,318.1 cfs with a water level at 735.43 ft. The backwater .computations for these recorded values resulted in a water level immediately downstream of the outlet structure of 737.6 ft. The crest of the outlet structure sits at an elevation of 730.0 ft. The computed water surface elevation, from HEC-RAS, would then indicate a depth of submergence of approxi.nately 7.6 ft at the outlet structure. A value well within the observed range given by the Water Resources Branch. Although lack of data prevented the HEC-RAS model of the Red River and Floodway sections, as shown in figure 3.5, from being subjected to a more thorough and detailed calibration process, the above results gave a positive indication of the performance of the model relative to the observed field data.

The resulting water levels immediately downstream of the outlet structure (point B) for the Red River and various floodway flow combinations were compared to the water levels obtained from the rating curve at the outlet structure (figure 3.4). From this analysis it was determined that the tailwater level would be higher than the weir crest at the outlet structure. Any effects the tailwater level may have on water levels upstream of the structure have been assumed to be negligible.

The literature shows that extensive tests on weirs indicate that submerged conditions (tailwater levels higher than the weir crest) affect the pressure distribution of the jet (Booy, 1993). This leads to modifications of the weir discharge coefficient. Correction of the weir discharge coefficient due to submergence is not an easy task. It requires reliable field data regarding the effect of the plunging jet on the downstream water level. The results from HEC-RAS have shown that tailwater levels would be higher than the weir crest of the outlet structure (for the range of flows analyzed), therefore indicating a potential tailwater effect on upstream water levels. The extent to which this affects upstream water levels has not been investigated.

Properly modeling such effects would be reflected on the modification of the discharge coefficient of the weir equation at the outlet structure. The limited availability of data at the outlet structure, specially at the higher flows, resulted in arriving at the rating curve shown in figure 3.4 using values given by the theoretical water surface profiles calculated during the 1960's for the design of the floodway channel. This rating curve does not take into account the relationship between the upstream and tailwater levels. Awareness of the existence of such conditions is imperative for the appropriate and successful modeling of any structure, which can only be accomplished with reliable and accurate field data. In the absence of such, an attempt is made to provide the best approximations with the available resources and good engineering judgement.

3.4 One-dimensional model (HEC-RAS)

3.4.1 Cross sectional data

Cross sectional data were entered following the program's requirements. In HEC-RAS the geometry of each cross section is identified in terms of x-y coordinates: x identifies a station number and the y value refers to the ground elevation at the x station.

The coefficients used for the calculation of energy losses are in accordance to table 2.1, for gradual transitions and typical bridge cross-sections. The floodway channel was assigned a roughness coefficient of n = 0.026.

3.4.2 Bridge data

Bridges were modeled using the energy method, for low flow conditions, and the pressure and weir flow method, for high flow conditions. For the latter method, the coefficient of discharge used is C = 0.8 for fully submerged pressure flow. When the weir becomes highly submerged the program calculates the upstream water surface by using the energy equation (standard step method).

3.5 Two-dimensional model (FESMWS)

3.5.1 Cross sectional data

The two-dimensional mesh consisted of cross sections described by 11 nodes, as shown in figure 3.6. The two dimensional mesh is made up of a combination of rectangular and triangular quadratic elements. The size of the elements was kept more or less constant within the channel, except near the bridge cross sections where the mesh was refined. The entire floodway channel, depicted by the two dimensional mesh, consisted of 12,289 nodes and 4,694 elements.

3.5.2 Bridge data

The limitation of the FESWMS program for the modeling of the bridge structures takes place during high flow conditions, *i.e.*, when water levels come in contact with the bridge deck leading to pressure and/or weir flow. Therefore, the interactive use of the HEC-RAS and FESWMS programs for the modeling of the floodway channel takes place only during high flows.

As previously stated, FESWMS allows for the incorporation of bridge piers into the model, therefore accounting for the added resistance introduced by the presence of the piers. The two dimensional model includes the piers at all bridges. The piers where entered in terms of an XY system to assign them a location within the mesh. Most of the piers of the Red River Floodway bridges have triangular or semi-circular ends. The FESWMS program requires that a drag coefficient be entered for all piers. The drag coefficient relates to the shape of the pier and takes into consideration flow separation and the force of the flow around the piers. A table listing typical drag coefficients for the various pier shapes can be found in the HEC-RAS Hydraulic Reference Manual v.2.0 (1997). Several runs were made in which the drag coefficient, C_d, was changed (range: 1.20-2.0). In general it was found that computed water levels increased (as expected) with a higher C_d. However, the magnitude of the change in water level as a result of a higher C_d value was not that significant (in the order of 1/10 of a foot). In trying to replicate the calibration event, the pier drag coefficient (in combination with the assigned Manning's n) that resulted in the best approximations was a $C_d = 1.33$.

3.6 Calibration results

The calibration process is an integral part of any numerical modeling. It tests the ability of the model to reproduce a certain known event, therefore confirming the validity of the input data and any assumptions on which the model is based. The calibration step is defined as "the process of adjusting the dimensions of simplified geometric elements and empirical hydraulic coefficients so that values computed by a model reproduce as closely as possible measured values" (FESWMS User's Manual v.2.0, 1997).

In the HEC-RAS model the empirical coefficient that was varied so as to observe the sensitivity of the model to these changes was value of Manning's n. In the FESWMS model, results were sensitive to changes in Manning's n and to a lesser extent to the piers' drag coefficient. Calibration of a model is usually done through the comparison of computed values (water surface elevations, velocities, etc) to field data. Both models, 1D and 2D, were calibrated by comparing the computed water surface elevations to the measured field data gathered during the 1997 flood.

3.6.1 Calibration event: 1997 flood

The 1997 flood, which has become commonly known as "The Flood of the Century", had an approximate natural[*] peak discharge in Winnipeg of 162,000 cfs. An event with a return period of

[*] Natural: assumes no flood control works in place.

110 years (Manitoba Water Commission, 1998). The 1997 flood was the result of a combination of unfavourable weather conditions: high soil moisture content, heavy snowfalls during the winter and heavy precipitation in the spring, rapid melting of the snow. All of this led to one of the worst flood events on record. A brief review of the Red River historical floods (figure 3.7) ranks the flood of 1997 as being almost equal in magnitude to the 1852 flood, the second largest flood on record.

During the 1997 flood, flows through the City of Winnipeg were maintained to within the river channel capacity by diverting Red River waters into the floodway and by controlling flows down the Assiniboine River through the operation of the Shellmouth Reservoir and the Portage Diversion. It has been recorded that flows through the floodway channel peaked on May 4, 1997 with a total discharge of 65,000 cfs. The Water Resources Branch collected water levels at different locations along the floodway during the 1997 flood. These collected field data have been selected as the base against which the computed water levels from both models can be compared. Table 3.3 shows the recorded field data.

Table 3.3 Water surface elevation at selected locations along floodway (Mb. Natural Resources, Water Resources Branch, 1998)

Station or Location	*High water mark (ft)*
148,450	770.63
147,850	770.28
140,000	769.97
137,500	769.62
130,000	769.18
CPR Emerson	768.96
125,600	768.78
120,000	768.14
110,000	767.18
CNR Sprague	766.39
Trans Can Hwy	766.09
95,840	765.6
PTH 15	764.15
CNR Redditt	764.21
73,580	763.31
CPR Keewatin	762.16
55,000	761.24
30,000	757.73
10,000	754.59
CNR Pine Falls	753.48
PTH 44	752.63
Outlet structure	752.35

3.6.2 Manning's n

As previously stated, a calibration of the models was obtained by adjusting the roughness coefficient of Manning's n so as to reproduce the measured field data. Manning's n values of n = 0.024, n = 0.026 and n = 0.028 were applied to the HEC-RAS model. The accuracy of the model was assessed by the magnitude of the error as given by the difference between the field and computed data. The best model performance was obtained with a Manning's n roughness coefficient of n = 0.026, with the largest discrepancy in water level being observed at the CPR Keewatin bridge, where the computed water level was 3.8" higher than that measured (figure 3.8).

Similarly, the values of the roughness coefficients were adjusted in the 2D FESWMS model so as to reproduce the 1997 field data. Manning's n values of n = 0.0215, n = 0.023 and n = 0.025 were applied to the model. The best approximations to the 1997 data were obtained by applying a roughness coefficient of n = 0.0215. When comparing the computed values (2D model) to field measurements it was found that the model's performance ranged approximately within $\pm$ 0.5 ft of the field measurements, with the lower water levels being computed at the upstream section of the floodway model, from CPR Emerson to the inlet. Calibration results are presented in figure 3.9.

An interesting observation to be made is in the difference in roughness coefficients used to calibrate each model. The HEC-RAS program required a higher value than that required by FESWMS. The Manning's n value applied to the 2D model is found at the low end of the specified range for excavated channels (refer to table 3.1). A review of the literature on 1D vs. 2D modeling of rivers indicates that a difference in roughness coefficient from one model to the other is quite common. The 2D model accommodates more accurately two-dimensional spatial variations found within a channel. And therefore it incorporates other roughness factors, such as eddy viscosity, independently from Manning's n. The 1D model on the other hand lumps all of the factors contributing to flow resistance into one weighted Manning's n, requiring a higher value. The fact that the floodway channel is a straight, uniform and clean (gently grassed) channel indicates that the selection of a low Manning's n (within the specified range, Table 3.1) is not that unreasonable.

In addition to the calibration event an additional run, corresponding to the design event, was made. The computed water levels from both models were then compared to the design water surface profile provided by the Water Resources Branch. Because the design event (Q = 60,000 cfs) is quite comparable to the calibration event (Q = 65,000 cfs) the performance of both models was expected to be similar to what was observed during calibration.

It is not known for certain whether the water surface profile for the design discharge takes into account the effect of bridges and therefore, the design event was run in both models for two cases:

i) no bridges in place, and

ii) bridges in place.

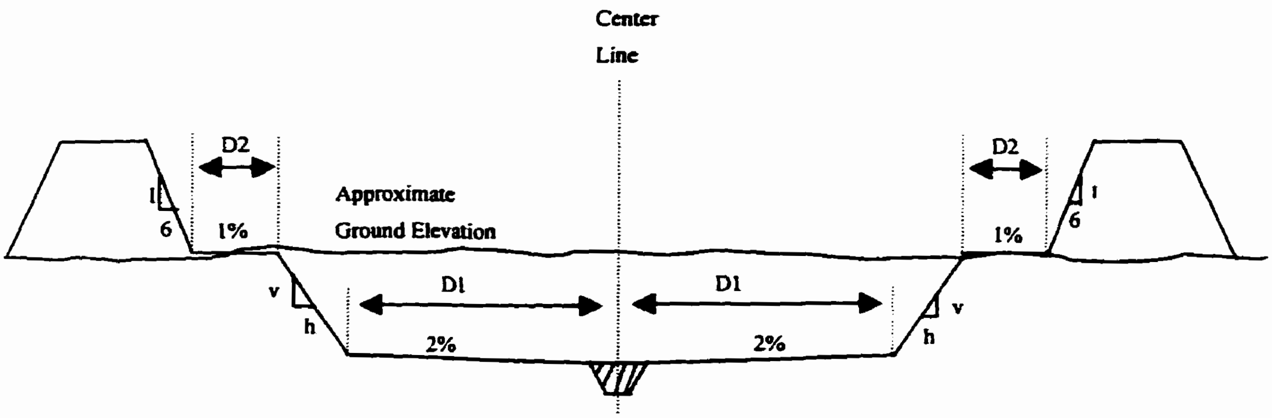
The first case refers to the modeling of the floodway channel with a discharge of 60,000 cfs, without any bridge structures, not even bridge piers. The second condition refers to the incorporation of the bridge structures into both models. For the 2D FESWMS model this translates into modeling of the floodway channel with bridge piers incorporated into the model. The results for both runs are presented in figures 3.10 and 3.11.

Figure 3.10 shows the results of both HEC-RAS and the FESWMS models (no bridges) plotted against the design water surface profile. Results from the HEC-RAS model are on average 0.40 ft lower than the design values. The 2D model provides still lower water levels, with the computed water levels being an average of 1.10 ft lower than the design values.

The performance of the models, as measured by the magnitude of the error between computed and design water levels, is improved when the effect of the bridges is taken into account. These results are shown in figure 3.11. An adequate performance of the HEC-RAS model is observed, with the computed water levels closely matching the design values (an observed average discrepancy between computed and design data of 0.18 ft). Although results from the FESWMS model are greatly improved, an average discrepancy of 0.32 ft, the model still exhibits the same behaviour observed for the calibration event.

An important observation from the results of both models is the significance of the effects of piers in each model. The presence of piers in the HEC-RAS model results in an increase of 0.24 ft in the water level at the inlet. The 2D FESWMS model reports a more significant effect of the bridge piers on flow, with almost 1.0 ft increase in the water level at the inlet.

In general, a two dimensional model is more accurate than a one-dimensional model. However, its accuracy is highly dependent on the accuracy of the topographic and hydraulic data being used. This makes the 2D model more sensitive to any existing discrepancies between the model and the real channel. The cross sectional data being used in the floodway model are not an as built representation of the channel. The generation of cross sectional data was based on the available topographic details and design drawings prepared in the 1960's. The tendency of the 2D floodway model to slightly under-predict water levels at the upstream section of the channel (CPR Emerson – Inlet) may be due to small errors in the constructed cross sectional data in this area.



Dimensions:

	D1 (ft)	*D2 (ft)*	*v:h*
North of B.Hill	270	150	1:6
Through B. Hill	Interpolation	150	1:3
South of B.Hill	190	150	1:6

Figure 3.1 A Typical floodway cross section (Mb. Natural Resources, Water Resources Branch, 1998).

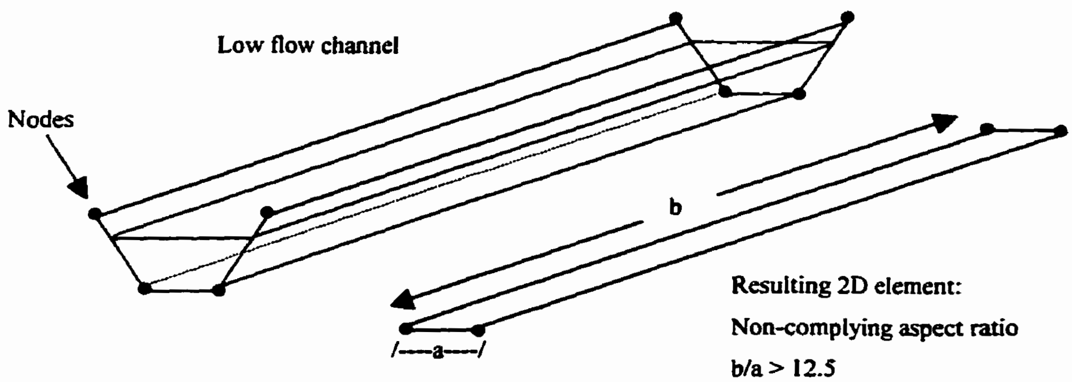


Figure 3.2 2D mesh of low flow channel.

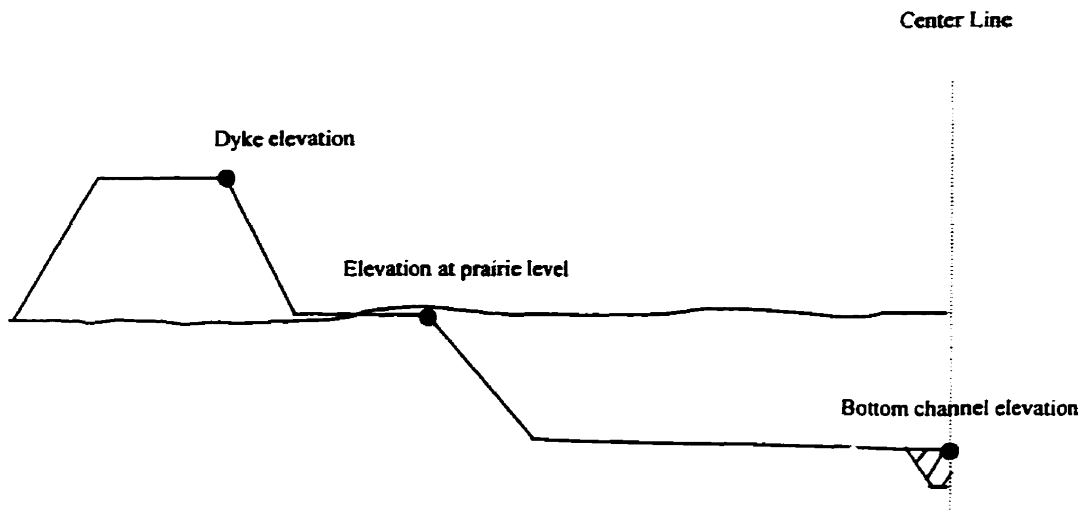


Figure 3.3 Known elevations at floodway cross section.

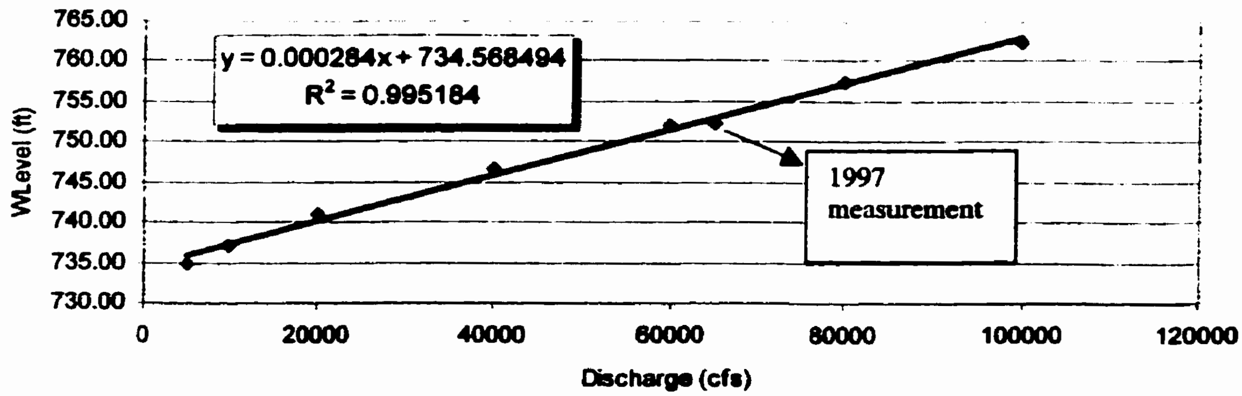


Figure 3.4 Rating curve at outlet structure.

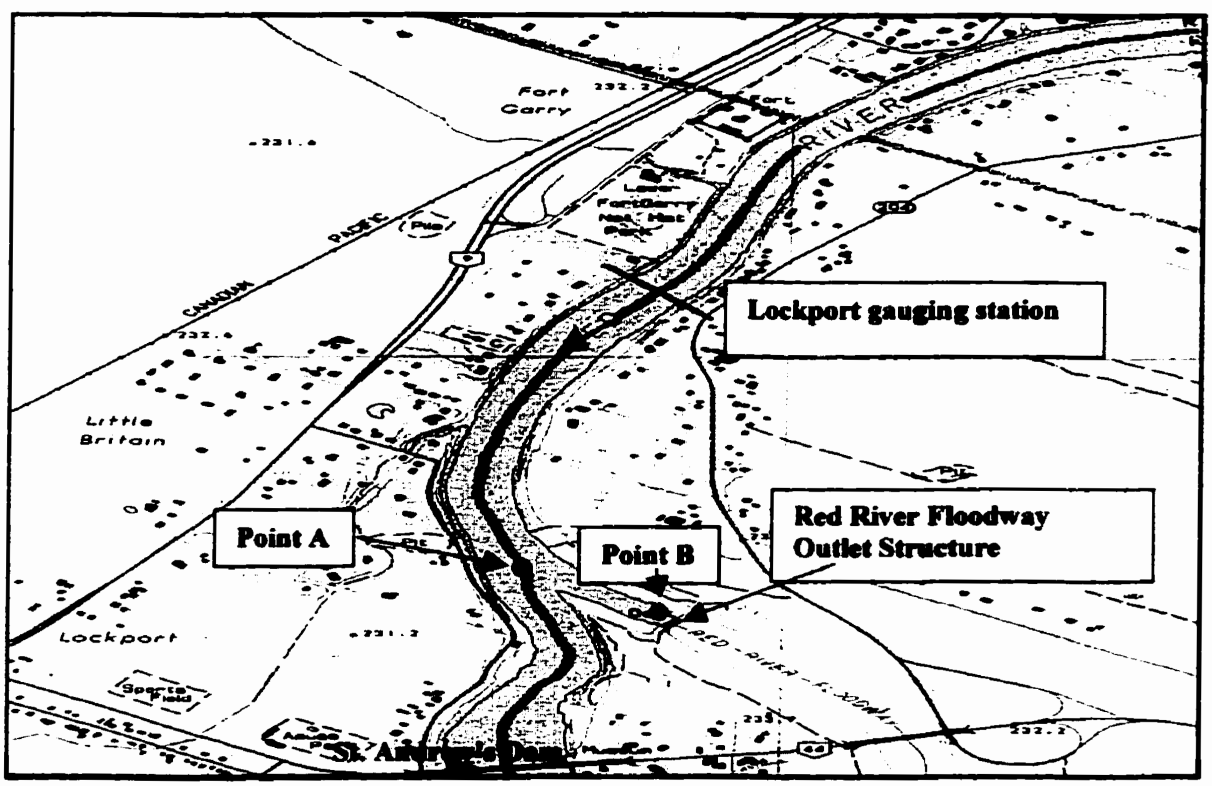


Figure 3.5 Confluence of the Red River and Floodway channel.

a)

b)

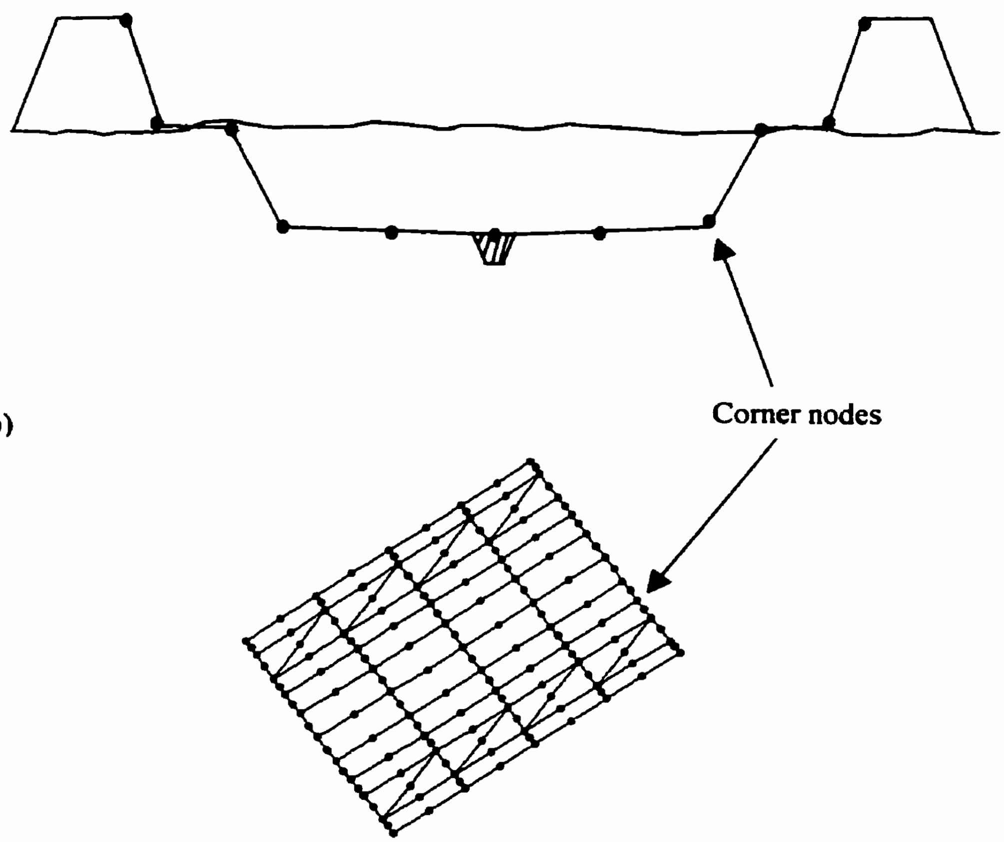


Figure 3.6 Two-dimensional floodway cross section. a) Generated cross section, and b) Generated 2D mesh.

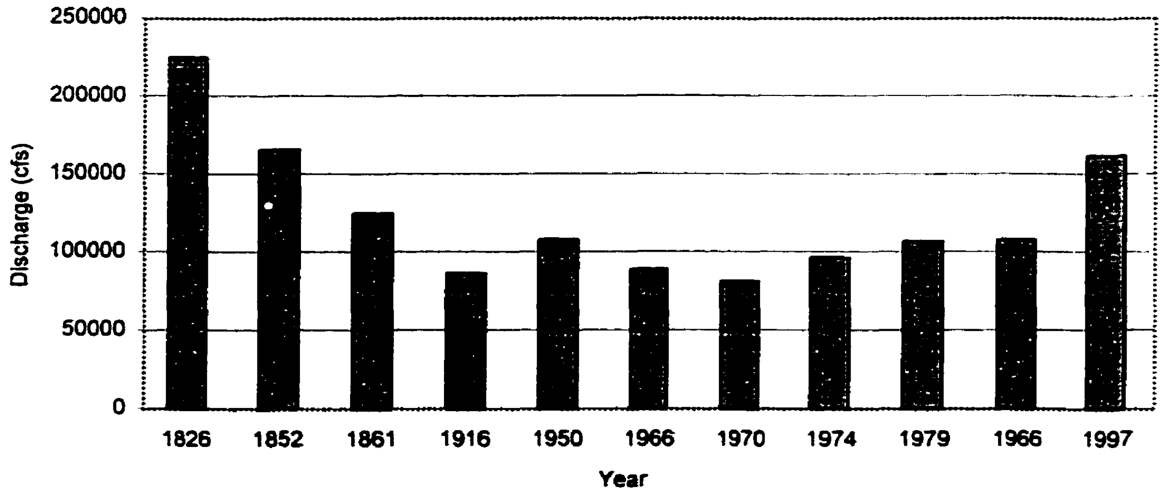


Figure 3.7 Red River historical flood events in excess of 60,000 cfs.

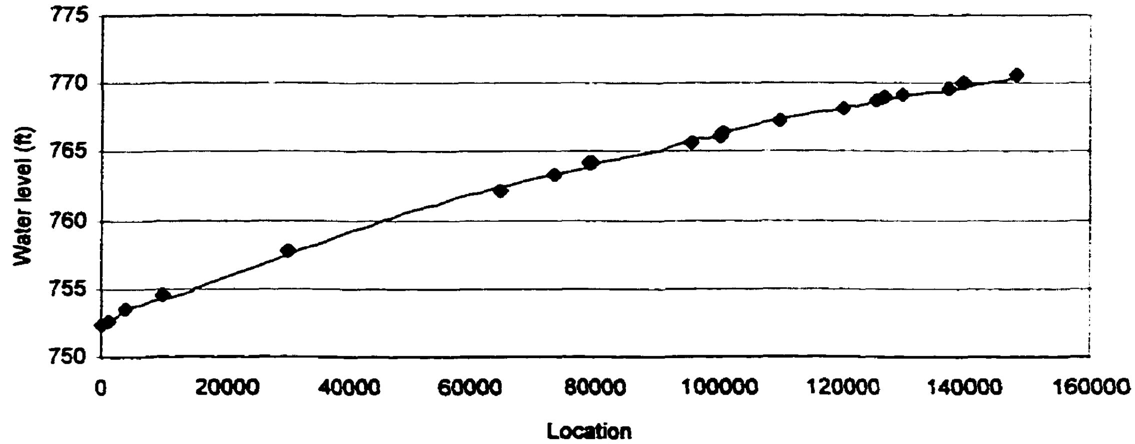


Figure 3.8 HEC-RAS model calibration.

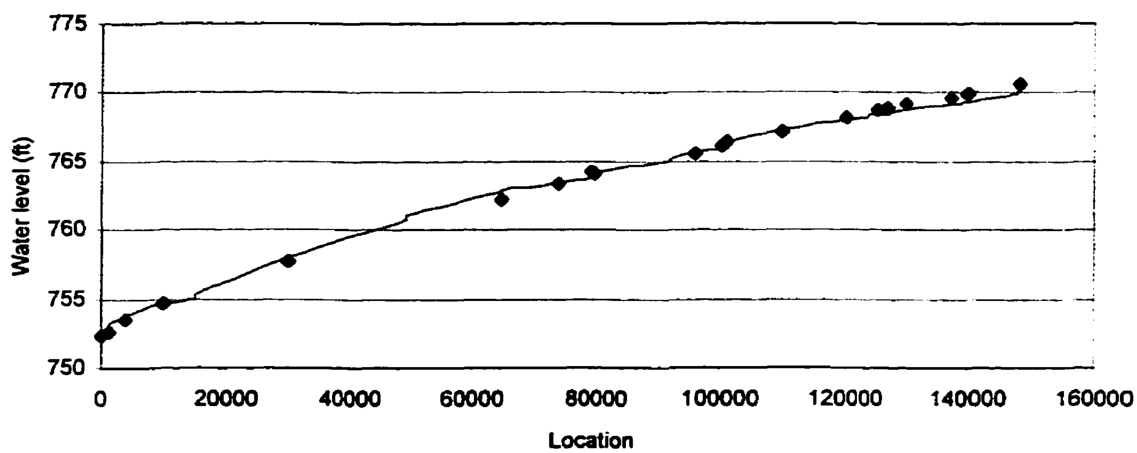


Figure 3.9 FESWMS model calibration.

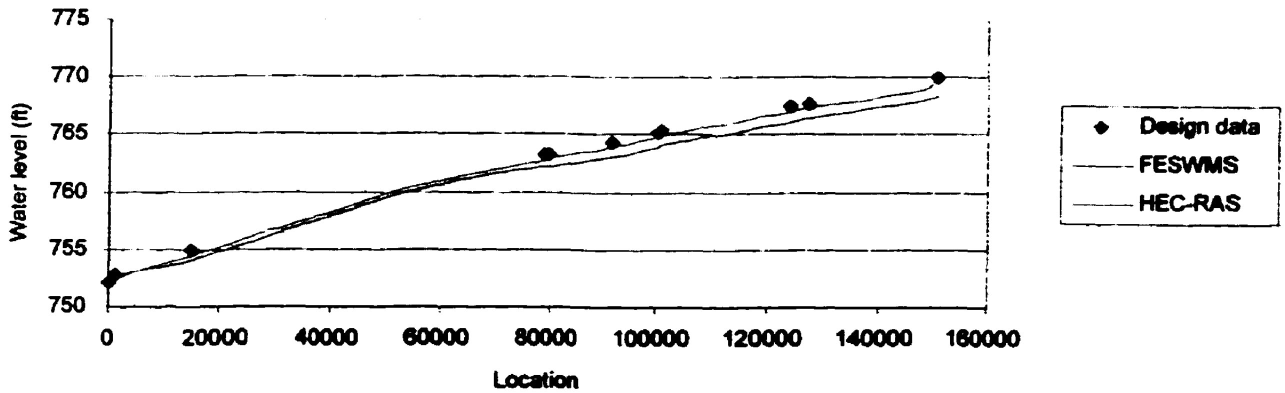


Figure 3.10 Design discharge (no bridges).

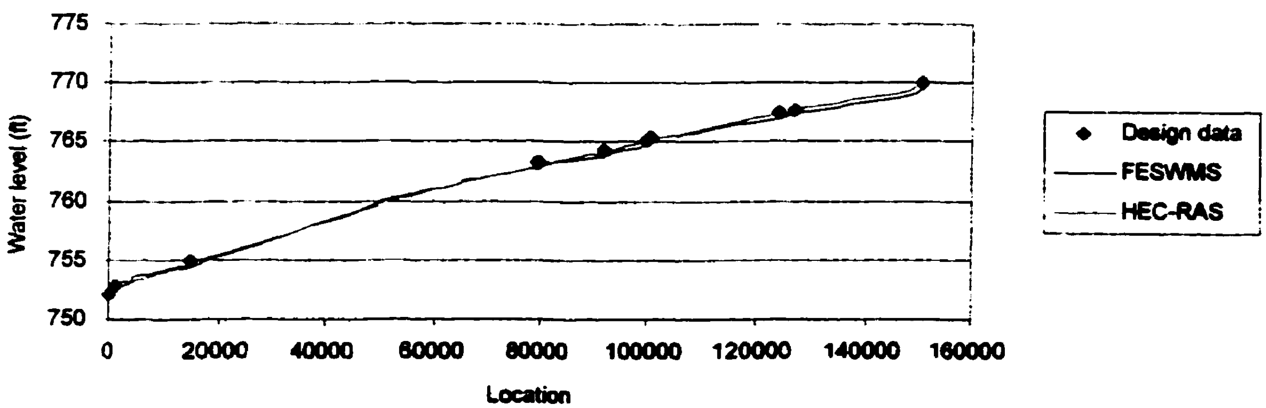


Figure 3.11 Design discharge (with bridges).

CHAPTER 4

Modeling

4.1 Introduction

One of the objectives of this thesis was to use the model of the floodway channel for the simulation of flood flows, more specifically flows corresponding to an event similar in magnitude to that experienced in the spring of 1826. The following sections present the results of modeling the floodway channel for discharges greater than its design maximum capacity. It was considered that modeling of the maximum design capacity of the floodway channel (100,000 cfs) was necessary in order to determine whether or not there exists a significant difference between the maximum design capacity and the maximum current capacity of the channel.

The worst documented flood event in the Red River valley occurred in 1826. A flood of this magnitude exceeds the maximum flood for which the flood control works were designed (169,000 cfs). This raises the following questions:

1) How would the existing infrastructure (flood control works) handle such an event?,
2) Would the existing infrastructure provide reliable/dependable flood protection to the City of Winnipeg?, and
3) If a flood event similar to that experienced in 1826 were to happen again, would the City of Winnipeg be prepared?

The modeling of an 1826 flood provides a closer look at these questions. This chapter presents the assumptions/conditions applied for the modeling of such an event, as well as a discussion on the results obtained.

4.2 Modeling of high flows

During the modeling of high flows, *i.e.*, flows exceeding the design capacity of the floodway channel, it is necessary to simulate spilling over the dykes. Identification and simulation of spill sections was important not only for determining the deficiency of the dykes along the floodway, but also for determining the appropriate boundary conditions for the model. The methodology used to develop a set of boundary conditions that would account for spills is presented in this section.

One of the difficulties encountered for the simulation of flows greater than or equal to the bankfull capacity of the floodway was the modeling of the overtopping of the dykes. The one dimensionality of the HEC-RAS program does not adequately account for the modeling of water spilling (sideways) out of a defined channel. The program assumes that the flow is perpendicular to the cross sections within the channel, and extends "vertical walls" at the edges of a cross section so as to contain the flow. This is shown in figure 4.1. The foundation of the one-dimensional program lies on the assumption that the total energy head is the same for all points in a cross section.

A two-dimensional program provides a more realistic approach for the modeling of spill conditions. The conditions at an overtopped section along a dyke can be modeled as flow over a broad crested weir. The overtopped section is modeled by defining single-noded weirs along the edge of the model domain, where the spilling occurs. The program makes use of the weir equation (equation 2.8) in order to calculate the flow being spilled. The single-noded weir is defined by an upstream node only and assumes that the flow going over it does not return to the model. This weir flow is an outflow and the program continues all computations, downstream of the spill section, taking this into account. The flexibility of the program to model spills allowed for the boundary conditions applied to the model to be modified accordingly. Consider the example, illustrated in figure 4.2. The figure shows an arbitrary reach of channel, for which the boundary conditions are an upstream initial inflow (Q_{ini}) and a water level (WL) at the downstream end. The water level, WL, assigned will correspond to the initial discharge, Q_{ini}, as long as the flow is contained within the channel (*i.e.*, $Q_{spill} = 0$). Once the capacity of the channel is exceeded at any point within the reach, the flow being lost needs to be accounted for at the downstream water level. In other words, with some of the initial flow being lost as Q_{spill}, the water level observed at the downstream end of the channel would correspond to a discharge equal to $(Q_{ini} - Q_{spill})$.

The difficulty in applying a relatively simple concept to the two dimensional model of the floodway channel was the iterative procedure leading to the "commensurate" set of boundary conditions, Q_{ini} and WL, that take into account the flow (Q_{spill}) lost due to spills. The size of the 2D mesh for the entire floodway channel made this iterative process time inefficient; the complete processing of the 12,289 nodes making up the mesh was time consuming, often interrupted due to instability introduced by the weirs located along the edges of the model. In order to eliminate instability problems and speed up the processing of the 2D mesh, making more time efficient the process of determining the appropriate boundary conditions, the floodway channel was analyzed in sections.

4.2.1 Methodology

The floodway model was broken into 14 sections, with the breaking points occurring at the bridges. Figure 4.3 shows a schematic of the floodway sections. The relevance of having the "break points" in between sections at the bridges was to facilitate the HEC-RAS – FESWMS interaction. Each section was analyzed individually, assigning as boundary conditions the commensurate set found for a particular discharge. The commensurate set of boundary conditions has been defined as an upstream inflow boundary condition and a downstream water level. The latter modified so as to take into account any spill(s) occurring within that particular section. The commensurate set of boundary conditions was found at every section for a set of flows: 65,000, 100,000, 116,000, 125,000, and 150,000 cfs.

The objective of the analysis was to develop a set of relationships that would define the conditions at each section in terms of water levels and discharges. From the analysis three sets of relationships were obtained for each of the 14 sections:

i) Initial discharge *vs.* water level (at the downstream end of the section),

ii) Initial discharge *vs.* water level (at the upstream end of the section),

iii) Initial discharge *vs.* spill.

Figure 4.4 shows a schematic of the most downstream section of the floodway channel (PTH 44 to outlet). This figure shows the commensurate set of boundary conditions, Q_{ini} and WL_1, assigned at the upstream and downstream ends of the section, respectively. The water level at the upstream end of the section (WL_2) corresponded to a water level just downstream of the bridge structure, as computed in the 2D FESWMS program. This water level was then taken into HEC-RAS where the program performed the hydraulic analysis at the bridge structure, providing a water level upstream of the bridge. The water level as obtained from HEC-RAS was then taken back into the FESWMS program and became the new water level at the downstream end of the next upstream section. The iterative process began again so as to modify the value of the new WL_1 and provide the commensurate set for the next upstream section. This process was applied to all 14 floodway sections, starting with the most downstream section (Outlet - PTH 44) and continuing in a downstream to upstream direction, until reaching the upstream end of the channel (St. Mary's bridge - Inlet). The range of flows investigated at every section provided 5 points from which the relationships for Q_{ini} *vs.* WL_1, Q_{ini} *vs.* WL_2, and Q_{ini} *vs.* Q_{spill} were derived. The supporting data are presented in Appendix B. For all sections the relationships are adequately described by a second degree polynomial, except for the Q_{ini} vs. Q_{spill} relationship which is well described by a linear fit. The benefit of having analyzed each section individually, developing the three sets of relationships is that now the discharges to be considered in this study, and the water levels along the floodway channel can be determined from these relationships.

4.3 Maximum design capacity

The floodway channel was designed to carry 60,000 cfs with a maximum capacity up to 100,000 cfs. For the simulation of 100,000 cfs through the floodway channel, dyke elevations were modified at the bridge crossings. Dyke openings were closed by raising and leveling the dykes with the adjacent elevations immediately upstream and downstream of the opening. The resulting water surface profile is presented in figures 4.5 and 4.6. These figures show the 100,000 cfs water surface profile plotted against the existing east and west dyke profiles. The blue boxed sections along the dykes identify the critical dyke sections where the elevations are below the computed water surface. These sections are near the entrance of the floodway channel and at PTH 59 south, along both the east and west dykes. Also along the west dyke, the section downstream of CNR Sprague and PTH 1 (red oval, figure 4.5) the water surface comes very close to the dyke crest, providing an average of 0.88 ft of freeboard.

The analysis from the 2D FESWMS model indicated a total of 4,512 cfs flowing over the dykes at these sections with a resulting water level at the inlet of approximately 777.40 ft. Downstream of the spill sections, the floodway channel carries the remaining flow of roughly 95,500 cfs without any further spills. The sections where the spills occur are identified as sections having the lowest dyke elevations (relative to the adjacent dyke elevations). It is considered that in a significant flood event where the floodway channel would be subjected to flows on the order of 100,000 cfs, these "critical" sections would be identified a priori and the dykes would be raised so as to avoid any spilling. Raising the dyke elevations at the identified critical sections in order to confine a 100,000 (cfs) flow within the channel results in a water surface elevation increase of 1.15 ft. at the inlet.

4.4 The 1826 flood

With a combined natural peak discharge of 225,000 cfs and water levels of 36.5 ft at James Avenue, the 1826 flood is the worst flood on record. Such an event has a return period of approximately 450 years (Water Resources Branch, 1999).

The spring of 1826 experienced the right combination of all the ideal factors leading to such a catastrophic flood event:

- Oversaturated soil conditions from a previous wet autumn
- Severe winter conditions with heavy snowfalls
- Sudden thaw in a late spring with heavy precipitation (rainfall)

It has been documented that conditions were worsened by ice jams (Andrews, J. 1993). An 8.85 ft rise in the Red River was experienced at Fort Garry over a 24 hour period.

4.4.1 Flow distribution

It is important to realize that the estimated peak discharge for the 1826 flood (225,000 cfs) is a combined discharge. This implies that both the Red River and Assiniboine River flows must be taken into account. In order to determine the contribution from each river, reference was made to figure 4.7. This figure shows the relationship between flows on the Red River downstream of the Assininboine, and the contribution of the Assiniboine River to those peaks. This relationship was approximated to be linear, with a defined range within which the Assiniboine River contribution probably lies 90% of the time (red dashed lines).

The plot shows the 1950 peak discharge. This event was accompanied by "unusually low" Assiniboine River flows, resulting in the Red River carrying over 90% of the total discharge. The "unusually low" term is relative to the plotted records prior to 1950 where there appears to be a more evenly flow distribution from both rivers. An additional point, the 1997 natural peak discharge has been added to the graph. Similar to the 1950 event, Assiniboine River flows were rather low during the 1997 flood, accounting for approximately 12% of the total discharge. The plotted 1950 and 1997 records suggest a possible variation of the combined peak discharge – Assiniboine River contribution relationship at higher flows from that shown in figure 4.7. However, due to a paucity of data to support this, the Assiniboine-Red River discharge relationship has been assumed to be as presented in figure 4.7. Considering the range within which the Assiniboine River Contribution probably lies 90% of the time, the Assiniboine River flows for a combined peak discharge of 225,000 cfs should be in between 39,500 cfs and 61,000 cfs. Based on this range, the flow distribution between the Assiniboine and Red Rivers is as shown on figure 4.8.

The floodway channel offers the only means of decreasing the magnitude of the Red River flow entering the City. The worst case scenario for the floodway is considered to be when the Red River contributes most of the flow. The flow distribution shown in figure 4.8a was therefore adopted for modeling an 1826 event; the Red River discharge accounts for over 80% of the combined peak discharge.

It has been assumed that during an 1826 flood event the Portage Diversion and Shellmouth Reservoir would be operating at their maximum capacity. The flow to be stored at the Shellmouth Reservoir and diverted through the Red River Floodway and Portage Diversion was determined based on a maximum capacity of the Red River channel through the City of Winnipeg. Flow limitations through the City were imposed based on the maximum protection provided by the primary dyking system (77,000-80,000 cfs).

The flow distribution is as shown on figure 4.9. This figure assumes that the total flow down the Assiniboine River (39,500 cfs) would be decreased by 32,000 cfs by having the Shellmouth Reservoir and Portage Diversion operating at their maximum capacity. This would leave 7,500 cfs entering the City. The assumption on the Red River is that flows allowed through the inlet structure would be such as to comply with the City's maximum capacity. Under such conditions the Red River discharge downstream of the inlet structure would correspond to 69,500 cfs. The Red River floodway would then be used to divert any excess flows from the Red River, in this case the resulting discharge is 116,000 cfs.

For an 1826 flood event, under the flow conditions described above, it has been determined that the Red River floodway would be subjected to a peak discharge of 116,000 cfs. A flow larger than its maximum design capacity.

4.4.2 Floodway operating rules

An important point to be observed is that the condition applied to figure 4.9 regarding the maximum flow allowed through the City of Winnipeg (77,000 cfs), is not in accordance with the original floodway operating rules established in the late 1960's. The operation schedule for the Red River Floodway is presented below (Manitoba Natural Resources Dept., 1998).

Operation Schedule

Flow conditions	*Method of operation*
Routine Operation:	
Up to 169,000 cfs	*The control structure will be utilized to maintain natural water levels at the inlet.*
Emergency Operation:	
169,000 cfs to 189,000 cfs	*The control structure will be operated to maintain an elevation of 751.5 at Redwood bridge. Water levels will be raised above natural levels at the inlet accordingly to a maximum of 775.8 ft.*
189,000 cfs to 199,000 cfs	*Raising of those portions of the primary dyking system which correspond to elevation 26.5 ft City datum, by 4 ft, will be undertaken at 190,000. When flows reach 199,000 cfs the water levels at Redwood bridge will be 755.5 ft with the water level at the inlet maintained at 775.8 ft. Construction of the dyking system must therefore be completed before a flow of 199,000 cfs is*

	reached. In the event that construction difficulties delay raising the dykes the control structure will be operated to maintain an elevation of 751.5 ft at Redwood bridge but the inlet elevation will not be allowed to exceed elevation 778.0 ft. These conditions will apply until dykes are raised.
199,000 cfs to 217,000 cfs	*The control structure will be operated to maintain the elevation at Redwood bridge at 755.5 ft, one foot below the emergency dyke level. Water levels at the inlet will be raised, as required, to the maximum elevation of 778.0 ft.*
Above 217,000 cfs	*The water levels at the inlet will not be allowed to exceed the maximum level of 778.0 ft.*

The objective of operating the floodway for flows below the design event is to regulate the flow through the City ensuring flood protection to those downstream of the inlet while maintaining the same water levels upstream of the inlet that would exist under natural conditions. The sensibility of this rule is appreciated in the provision of maximum flood protection for the City without affecting negatively the area upstream of the floodway.

During emergency operations, *i.e.*, flows greater than the design event, the rules dictate that water levels at the inlet would be allowed to rise, but not exceed an elevation of 778.0 ft. At this point more water would have to be permitted to flow through the City, with the condition of having raised the primary line of defense by 4 ft. The sensibility of this is debatable, since the protection of an entire City (with nearly 3/4 million people) against flows of such magnitude is solely based on our ability to raise the primary dyking system.

Having just experienced the flood of the century, an event lesser in magnitude than that being modeled, and after reviewing some of the numerous lessons learned by the different departments/committees involved in the flood fighting effort there is the possibility that the raising of the primary line of defense (a total of 110 km of dyke through the City) may not be accomplished on a successful and timely fashion. The successful raising of the primary dyking system, under emergency conditions, would depend on a number of factors some of which we may not have control over. For example, such a task would require:

i) readily available and updated drawings of the existing primary diking system, as well as forecasted Red River water levels;
ii) the coordinated effort of both paid workers and volunteers in the raising of reliable, well constructed dykes;
iii) the supply and availability of construction material (borrow areas, sandbags) taking into account the unforseeables: weather, soil conditions – wet weather or frozen ground would affect negatively the timely completion of the dykes;
iv) extra material required for the control of leaks/possible breaches; and
v) most importantly the availability of manpower, not only during the construction of the dykes, but also for the continuous monitoring of the structures.

Another important issue to be raised is the adequate provision of freeboard. The rules state that water levels in the city would be maintained a foot below emergency dyke level (Refer to floodway operating rules, for flows 199,000 cfs to 217,000 cfs). This condition then does not take into consideration possible wind and wave action. With the presence of wind, there is a potential for the dykes to be overtopped. The quantity of water stored within the river channel would result in the rapid erosion of the overtopped section(s) leading to disastrous dyke breach(es).

The following are some of the facts reported by the City of Winnipeg (McNeil and McBride, 1997) after the 1997 flood:

- total sandbag production of 6,608,000 sandbags;
- raising of both the primary and secondary diking systems at several locations was necessary;
- over 9,000 people were evacuated (St. Norbert, Kingston Cres, Scotia Street);
- total flood related costs were given at $47 million.

All of the above were efforts required for a flood event lower than the maximum design event of 169,000 cfs. The efforts required for an 1826 flood event would certainly be of a greater magnitude and in the case of failure, the losses would be far greater.

The approach taken in this study has given priority to the protection of the City of Winnipeg by limiting Red River flows to manageable levels (*i.e.*, maximum capacity of 77,000 – 80,000 cfs as provided by the existing primary dyking system), leaving the excess flow to be diverted through the floodway channel. The reason for limiting the Red River flows through the City is that it is believed that facing a flood event of an 1826 magnitude, under emergency conditions there would be insufficient time to raise the primary line of defense by 4 ft. The floodway channel would then be forced to take the excess flow.

4.4.3 Results

The water surface profile corresponding to a discharge of 116,000 cfs has been derived by approximating water levels within the floodway from the set of relationships developed at each section of the channel (section 4.1.1). These water levels reflect any changes in discharge that might have occurred due to spills. The resulting water surface profile is presented in figures 4.10 and 4.11. These figures show the water surface profile being plotted against the existing dyke profiles. This comparison of water surface and dyke elevations allows for the identification of the spill sections, *i.e.*, sections where the water surface elevation is above the dyke crest. These sections with their approximate spill have been listed in table 4.1.

Table 4.1 Location and magnitude of spills

	Q_{spill} (cfs)	
Floodway section	*West dyke*	*East dyke*
Inlet – St. Mary's	68.6	5,756.8*
CPR Emerson – PTH 59 (s)		562.4
PTH 59 (s) – CNR Sprague	11,511.7	283.07

*Spills along the east dyke within this section of the floodway occur at the entrance of the channel (refer to figure 4.12). In the 2D model the computed water surface elevations along this section of the dyke are compared to the elevation of the dyke. The model determines spilling (*i.e.*, water flowing out of the channel) at this particular section because the elevation of the dyke crest is lower than the computed water level. It is suspected, however, that the direction of the flow may be reversed should the section of the Red River, upstream of the floodway entrance, be considered.

For a discharge of 116,000 cfs, it has been estimated that the combined spill over both the east and west dykes is approximately 18,182 cfs, with the west dyke accounting for almost 64% of the total spill. The approximate location of these spills within each of the sections (floodway sections listed in Table 4.1) is shown in figures 4.12 through 4.14. The velocity distribution within the channel is more or less uniform, with the highest velocities found at the center of the channel. The magnitude of these velocities varies from 5.0 to 6.0 ft/sec, approximately. However velocities as high as 7-10 ft/sec were computed at the PTH 59 south spill section.

It is considered that letting the floodway channel overflow its dykes at the previously identified locations is a very unlikely choice of action, especially when the potential for dyke erosion and breach is quite high. Such a case can only lead to larger flows spilling into the City with great economic loss and flood damages. Therefore, the next case examined was the confinement of the total dishcarge of 116,000 cfs within the floodway channel. For this several low spots along both dykes were raised just high enough so as to prevent spilling. The most critical sections can be identified from figures 4.15 and 4.16. These figures present a comparison of the existing conditions and the dyke elevations required in order to confine the flow within the channel. The boxed areas (sections 1 through 4, figures 4.15 and 4.16) are considered to be the most critical in terms of the 'depth' by which they have to be raised and the length of the section.

> *Section 1*: (West dyke) Roughly from St. Mary's Road to CPR Emerson, includes a distance of approximately 3.87 miles. This section was raised an average of almost 2.0 ft.
>
> *Section 2*: (West dyke) Includes a distance of 1.27 miles, approximately. This section can be found about 1 mile upstream of CNR Sprague to some 400 ft downstream of PTH 1. Raised an average of 3.5 ft.
>
> *Section 3*: (West dyke) Roughly from CNR Pine Falls to the end of the channel. This distance, 1.25 miles approximately, was raised an average of 3.17 ft.
>
> *Section 4*: (East dyke) This section includes the most upstream end of the floodway channel, from the entrance to CPR Emerson. An approximate distance of 4.7 miles was raised an average of 1.5 ft.

Raising the elevation of the dykes prevents any spills, however, higher water levels are observed throughout the channel. Of particular importance is the water level observed at the floodway entrance. The containment of the flow causes a 2.76 ft increase in the water level at the floodway inlet, resulting in a water surface elevation at this point of 781.52 ft. Figures 4.17 and 4.18 present the resulting water surface profile.

It can be observed that the water surface comes 'too close' to the crest of the dykes at several points. In other words, even after having raised several sections, the freeboard provided at these locations is not adequate. Table 4.2 summarizes the average freeboard provided at sections 1 through 4 (as identified in figures 4.15 and 4.16).

Table 4.2 Average freeboard at raised dyke sections

	Average	*Freeboard (ft)*
Sections	*West dyke*	*East dyke*
Section 1	0.5	
Section 2	1.0	
Section 3	0.57	
Section 4		0.60

While the objective has been achieved, *i.e.*, to contain the flow within the channel and avoid any spills, it must be kept in mind that the provision of adequate freeboard is a safety factor that needs to be taken into account. This safety factor protects against conditions such as strong winds or the rapid increase in water levels due to a coincident rainfall event.

BRIDGES

With the resulting water surface profile (as shown in figures 4.17 and 4.18) some of the bridge structures crossing the floodway channel are subjected to pressure flow, while others are completely submerged. Figure 4.19 shows the bridge conditions found in the floodway channel with a contained discharge of 116,000 cfs. Bridges are numbered as follows:

1. PTH 44
2. CNR Pine Falls
3. CPR Lac du Bonnet
4. PTH 59 (n)
5. CPR Keewatin
6. CNR Redditt
7. PTH 15
8. GWWD
9. PTH 1
10. CNR Sprague
11. PTH 59 (s)
12. CPR Emerson
13. St. Mary's Road

A total of 8, out of the 13 bridges crossing the floodway channel would be completely submerged. With the depth of submergence varying from 0.48 ft (at CNR Sprague) to 5.7 ft (at PTH 44).
Bridges subjected to pressure flow have the potential to further obstruct flow, reducing the channel capacity and increased water levels caused by a sudden blockage or jam due to an accumulation of material and debris often carried by flood waters.
An important point to be considered is the water surface elevation at the entrance of the channel. As previously stated, an elevation of 781.52 ft was computed at the inlet. This elevation becomes significant when considering the elevation of the dykes in the surrounding area: Turnbull drive dykes, the East-West dyke. The latter is an extension of the floodway's west dyke and continues in a south-easterly direction away from Winnipeg. The dyke has an average crest elevation of 781.0 ft near Winnipeg. With a water level of 781.52 ft at the floodway entrance, water levels against the East-West dyke would be above the crest. In 1997 Turnbull drive dykes were raised to an elevation of 773.5 ft (Manitoba Water Commission, 1998). An elevation well below the computed water level.

Also, at the inlet structure, the floor of the machine room is found at 780.0 ft. Having the floor of the machine room under water introduces problems for the operation of the control structure.

The removal of bridges is a possible means of lowering the water levels at the inlet. "Blowing up" the bridges is not a recommended alternative. The 13 structures crossing the channel are of a considerable size. Blowing out the bridges could block the flow, increasing the upstream water levels and possibly causing an even worse situation. The alternative of removing only the bridge decks has been investigated. It has been suggested that the bridge deck could be removed by sections (*i.e.*, spans) leaving the piers in place. Such an approach would be possible, however, the effort and costs incurred would be high. The process of disassembling the bridge deck involves cutting of the deck in between the girders. This would allow the removal of the entire girder section with the concrete deck on top as a whole unit. While the bridge sections would be removed intact, it is very unlikely they would be placed back on top of the piers once water levels within the channel have receded. The removed sections would have to be replaced. The removed sections, however, could be 'recycled' and used as smaller structures providing access to other areas where they would be subjected to less stress and weight.

It is possible that the bridge piers after being subjected to large hydrodynamic forces, both from the flowing water and the material carried in it, might also be damaged. These piers might not provide adequate support to a new bridge deck. Therefore, the removal of bridges, even if it is by sections, would also require the replacement of all bridge supports. This translates into the building of new structures.

The analysis conducted assumed the removal of the bridge deck from some of the bridge structures. The piers, however, remained in place and their effect was taken into account by carrying out the simulation in the FESWMS program.

The results shown in figure 4.19 were used to select the bridges to be removed. From a hydraulic point of view, the option of selectively removing the submerged bridges was considered to be justified since the larger head losses are observed at these bridges. Therefore, the first step in the analysis was to examine the removal of only those structures that would be under water. And as a last option, the removal of all bridges was also considered.

The following simulations were run:

Simulation 1: Removal of the 5 most upstream bridges only. (St. Mary's Road, CPR Emerson, PTH 59 (s), CNR Sprague, PTH 1).

Simulation 2: Removal of all submerged bridges (St. Mary's Road, CPR Emerson, PTH 59 (s), CNR Sprague, PTH 1, CPR Lac du Bonnet, CNR Pine Falls and PTH 44).

Simulation 3: Removal of all bridges

For simulations 1 and 2 the remaining bridges (*i.e.*, those subjected to pressure flow only) were left in place, thus the analysis at these bridge structures was conducted in HEC-RAS. For the bridges that were removed, it has been assumed that the bridge piers are still in place. Taking this into account the analysis has being carried out in the 2D FESWMS program. The results are presented in the figures 4.20 to 4.22.

The results of simulations 1, 2 and 3 are shown in figures 4.20, 4.21 and 4.22. The computed water surface profile from each simulation is compared to that resulting in a water level of 781.52 ft at the inlet (Refer to water surface profile for a confined flow, figures 4.17 and 4.18).

The removal of the 5 most upstream bridges (simulation 1) lowers the water level at the inlet from an elevation of 781.52 ft to 779.85 ft. A decrease in water level of 1.67 ft. It was found that the additional removal of the remaining submerged bridges, the three most downstream structures (simulation 2), does not have a significant effect in reducing the water level at the entrance of the channel. Simulation 2 results in an upstream water level of 779.67 ft, only 3" lower than the water level achieved during simulation 1. This is an indication that the most upstream bridges seem to be the most important. While lower water levels are observed throughout the channel during simulation 2, whatever contribution is made by the removal of PTH 44, CNR Pine Falls and CPR Lac du Bonnet (bridges at the downstream end of the channel), the presence of the remaining bridges in the middle section of the channel still results in significant head losses (varying from 0.31 ft to 0.51 ft) which contribute to the increase of upstream water levels.

The removal of all bridges, simulation 3, results in a water surface elevation at the inlet of 779.26 ft, a decrease in water level of almost 2.3 ft. It is considered that simulation 3 results in an improvement over the original water surface elevation of 781.52 ft. However, even with the removal of all bridges an elevation of 779.26 ft at the inlet of the floodway channel is still an uncomfortable water level to deal with in terms of the inadequate freeboard that the dykes would provide.

COMPOSITE CROSS SECTIONS

It is considered that in order for the floodway channel to accommodate flows from an event larger than the one experienced in 1997, its current capacity would need to be increased. There have been several suggestions made as to the different alternatives available for the redesigning of the channel. These have included raising of the bridges, widening of the channel cross section, and the use of composite cross sections. The latter is looked at more closely in this section.

The alternative of using composite cross sections throughout the floodway channel considered the idea of lining some sections of the channel. Lining of the channel using a material with a roughness coefficient smaller than that currently applied would allow a greater discharge to pass through the same cross sectional area. This is as derived from Manning's equation for open channel flow, which establishes an inverse relationship between the discharge and the roughness coefficient of Manning's n: everything else constant, Q increases with a smaller n (equation 2.3). The material investigated is concrete. It was found that typical values for the roughness coefficient of concrete, depending on the type of finish, vary from 0.011 to 0.015. In the analysis a Manning's n value of 0.012 has been used.

The two-dimensional model used in the analysis consisted of the floodway channel only, taking into account the bridge piers. The alternative considered the following options:

Option 1: Lining of the bottom of the channel

Option 2: Lining of the bottom of the channel and east and west berms

Option 3: Lining of the bottom of the channel and side slopes

These options have been illustrated in figure 4.23.

The results are presented in figures 4.24 to 4.26.
The resulting water surface profiles from each one of the options has been compared to the one resulting from bridge simulation 3. Bridge simulation 3 was carried out assuming all bridges had been removed and flow through the channel was only affected by the presence of the bridge piers. It will be recalled that this simulation was performed using the calibrated Manning's n of 0.0215. The results from figures 4.24 to 4.26 indeed show that the use of a smaller friction coefficient results in an increase in capacity. Lower water levels are observed throughout the channel, most importantly at the entrance of the floodway.

The results from all three options show a significant improvement in the channel capacity. Option 1 resulted in a water level of 776.1 ft (at the entrance), a decrease of 3.16 ft from the previous elevation of 779.26 ft obtained from bridge simulation 3. Option 2 yielded a water surface elevation of 775.75 ft, approximately 4" lower than what was achieved with option 1. Comparison of results from bridge simulation 3, option 1 and option 2 indicates that while a completely lined channel would definitely have a greater capacity than if it were natural, a combination of natural-lined sections within the same cross section proves to be advantageous and may be more cost effective. The calculated water surface elevation at the entrance of the channel for option 3 is of 775.51 ft. This option results in water levels (at the entrance) almost 3.75 ft lower than the previous 779.26 ft.

4.5 Discussion

The results presented in the previous sections show that the existing floodway channel is not adequate to accommodate flows corresponding to an 1826 flood event. It was found that with a discharge of 116,000 cfs the floodway channel would spill roughly 15.5% of the total flow, with more than half spilling over to the west side.

The capacity of the floodway channel is not uniform. The most critical section, *i.e.*, that section of the channel with the least capacity, can be found at the upstream end of the floodway; roughly from the entrance to PTH 1. Within this segment of the channel, the west dyke is overtopped at St. Mary's Road and PTH 59-CNR Sprague. This scenario corresponds to a do-nothing alternative, *i.e.*, subjecting the floodway channel in its existing conditions to flows of a magnitude corresponding to an 1826 flood event. This is a very unlikely choice of action.

Supposing Winnipeg were to face an event similar to that experienced in 1826 (and a flow distribution as that assumed in this thesis), raising of the floodway dykes is one emergency type of alternative that would be implemented. Such an alternative would confine the flow within the channel avoiding any spills. However, the confinement of the flow results in higher water levels throughout the channel. The impact of such levels needs to be considered not only at the bridge structures but also at the entrance of the floodway, where a water level rise of 2.76 ft is experienced. The resulting water surface elevation of 781.52 ft at the inlet is of great concern not only because of the backwater effect it would have upstream of the floodway, possibly aggravating conditions in the flooded areas, but also because of the crest elevation of the west dyke, which prevents flooding of the City. Under such conditions, the current elevation of the dykes would provide no freeboard.

The velocity distribution within the floodway channel is quite uniform with magnitudes varying from 5.0 to 7.0 ft/sec. However, at the overtopped sections velocities as high as 10.0 ft/sec were observed. Therefore, erosion at the overtopped sections could be expected. This would inevitably result in a dyke breach. As previously stated the area most susceptible to this is that from St. Mary's Road to PTH 1, along the west dyke. If the floodway dykes (west side) were to be overtopped, water would undoubtedly flood the southernmost part of St. Vital and St. Boniface. The land elevation in this area varies from approximately 754-760 ft north of St. Mary's Road to 771 ft in the RM of Springfield, just south of PTH 1. West of the spill sections, the perimeter highway offers points of high ground, varying from 764 to 777 ft. The perimeter highway can be used as a temporary, secondary west dyke in order to prevent the spilled water from further flowing into the City.

Figure 4.27 shows the area bounded by the perimeter highway to the west, PTH 1 to the north and St. Mary's Road to the south that could be ultimately sacrificed should the dykes at the upstream section of the floodway channel be overtopped. This area, approximately 10 mi^2, would help reduce the high water levels experienced at the inlet. The economic losses in this area, although not as great as flooding the entire City, would still be significant. Not considering the inconvenience and disrupted life caused to the south end residents of these areas. It should be noted that, the south end sewage treatment plant is also found within this area.

A possible way to relieve flows down the floodway channel might be to force a section of the east dyke to breach. The breached section would let water out of the channel, inundating the land east of the floodway, therefore reducing water levels within the channel. However, the location of the breached section is somewhat restricted to the downstream end of the floodway (section considered: downstream of PTH 44), where the topography of the land is the lowest and the spilled water would eventually flow into the Red River. It is considered that providing a breached section at the upstream end of the channel (Inlet to PTH 1) would be of very limited usefulness since the area east of the dyke would already be flooded. Figure 4.28 shows the extent of inundation for the 1826 flood. With a flood of this magnitude, it is expected to have water against the east dyke, outside of the floodway channel, as far north as PTH 1.

The removal of the bridge structures is also an alternative that would contribute some additional channel capacity. It has been found that the 5 most upstream bridges (St. Mary's Road, CPR Emerson, PTH 59 south, CNR Sprague and PTH 1) are the primary bridges that affect most significantly the channel hydraulics. Removal of these bridges lowers the water level at the inlet by 1.67 ft. The findings from simulation 3 (removal of all bridges) shows that the water surface elevation at the inlet could be lowered to 779.26 ft. However, even with all bridges removed, there is less than 2 ft. of freeboard provided between the dyke crest and the water surface.

It is considered that the benefits obtained from removal of the bridge decks would only be temporary and achieved at a very high cost. In 1996, Manitoba Highways spent approximately $2.8 million twinning PTH 59 south over the floodway. Using this as a base amount, a rough value of $3.0M could be placed on each bridge removed. Additional expenses due to the removal of the existing piers and the demand the City would have on the placement of these bridges in the least amount of time are factors that could not be ignored and indeed would result in an increase of the estimated total cost per bridge. Under emergency conditions the time required to complete the removal of a bridge deck should also be considered. The process could be slow depending on availability of equipment and working conditions. The alternative of bridge removal is looked upon as a possible course of action to be taken under emergency conditions that would help keep Winnipeg dry. However, this would have been achieved at a very high cost and not enough confidence on the degree of flood protection provided.

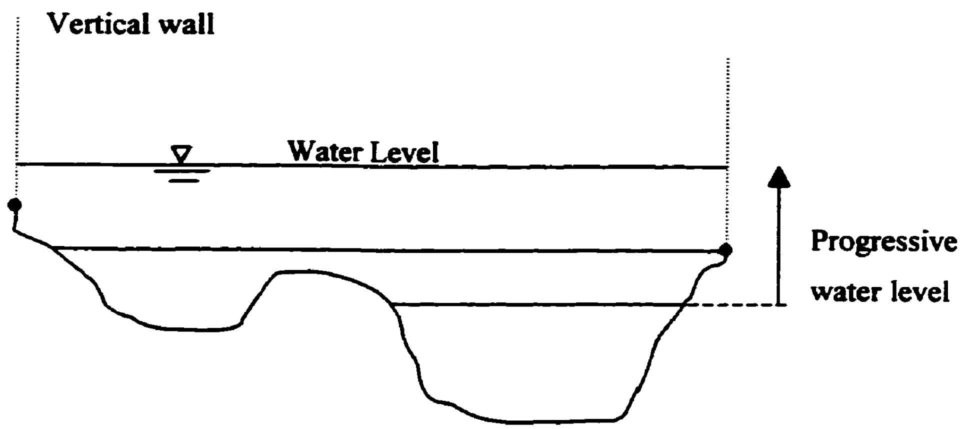


Figure 4.1 HEC-RAS hypothetical river cross section.

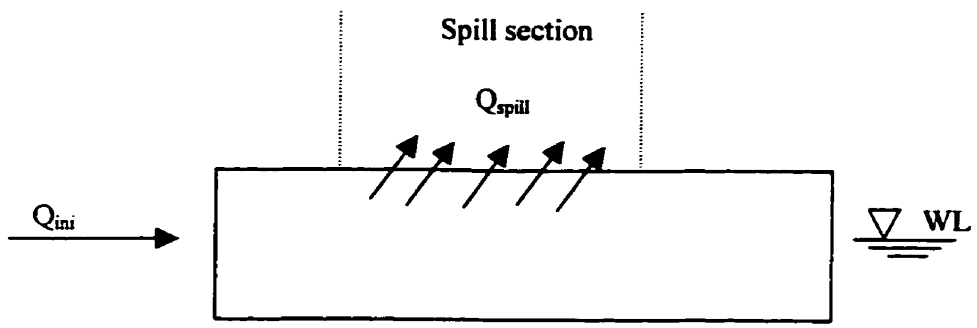


Figure 4.2 Example of a hypothetical channel cross section.

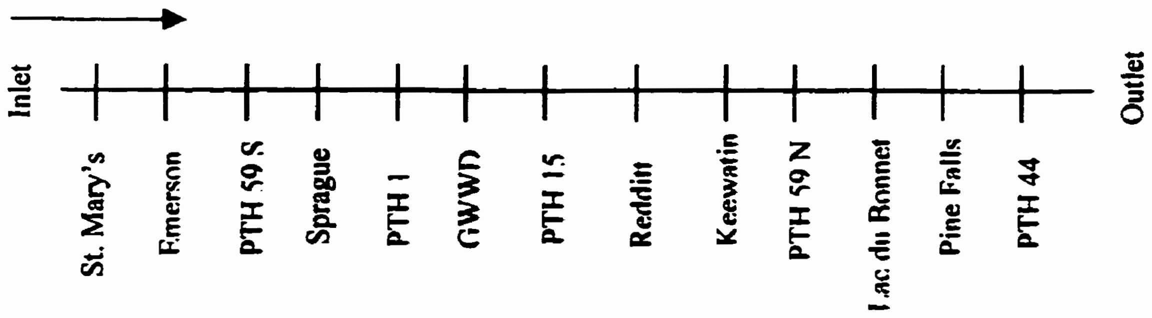


Figure 4.3 Floodway sections.

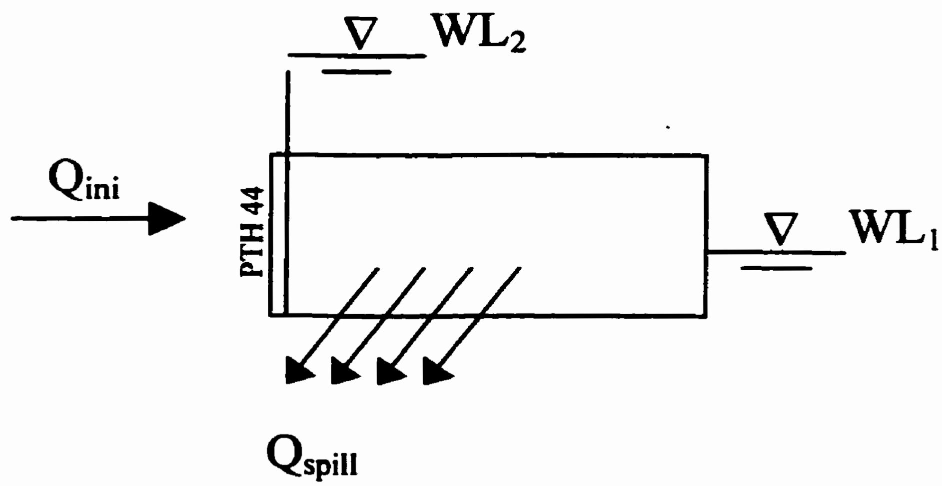


Figure 4.4 Isolated floodway section.

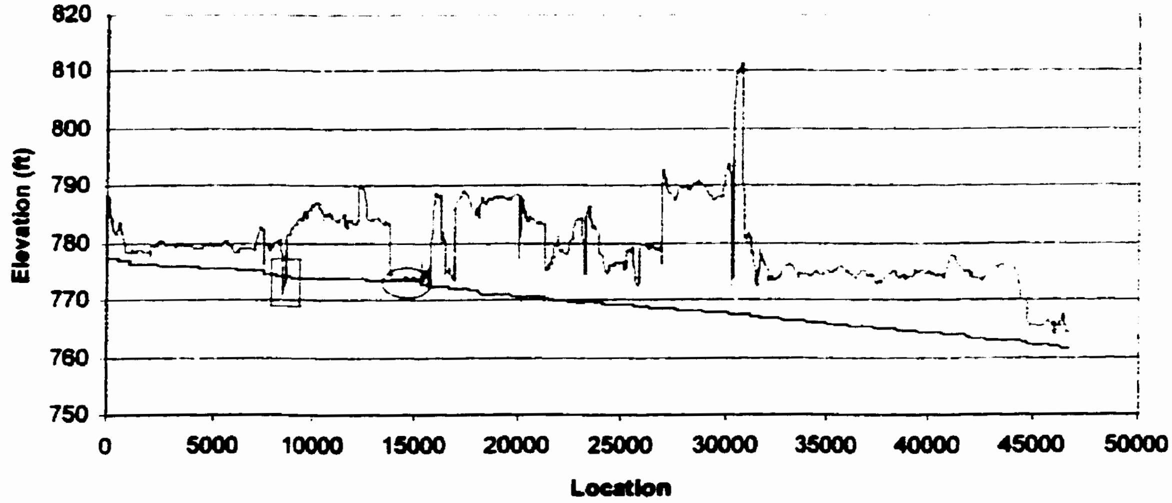


Figure 4.5 West embankment: water surface profile (Q = 100,000 cfs).

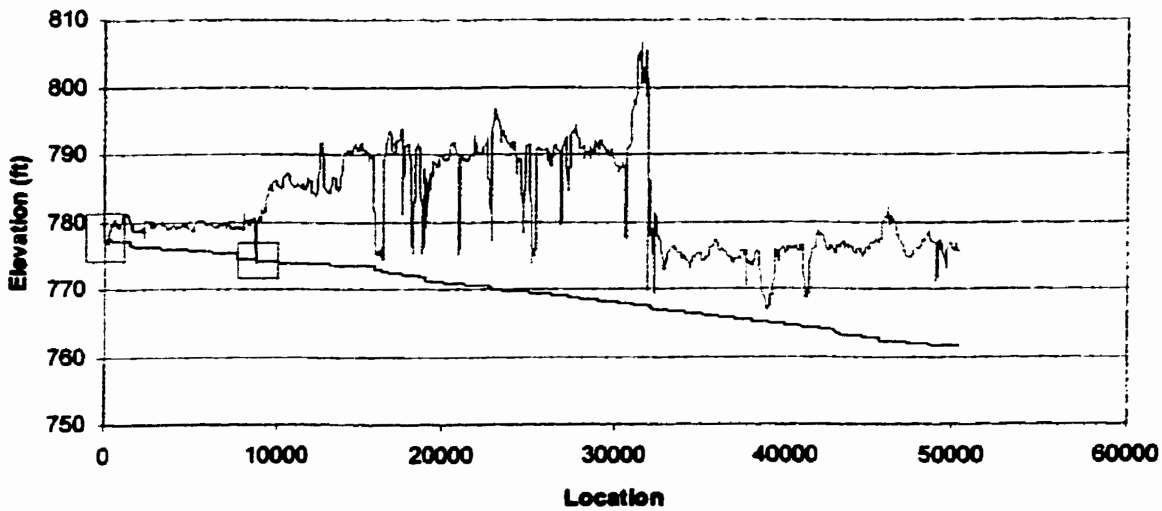


Figure 4.6 East embankment: water surface profile (Q = 100,000 cfs).

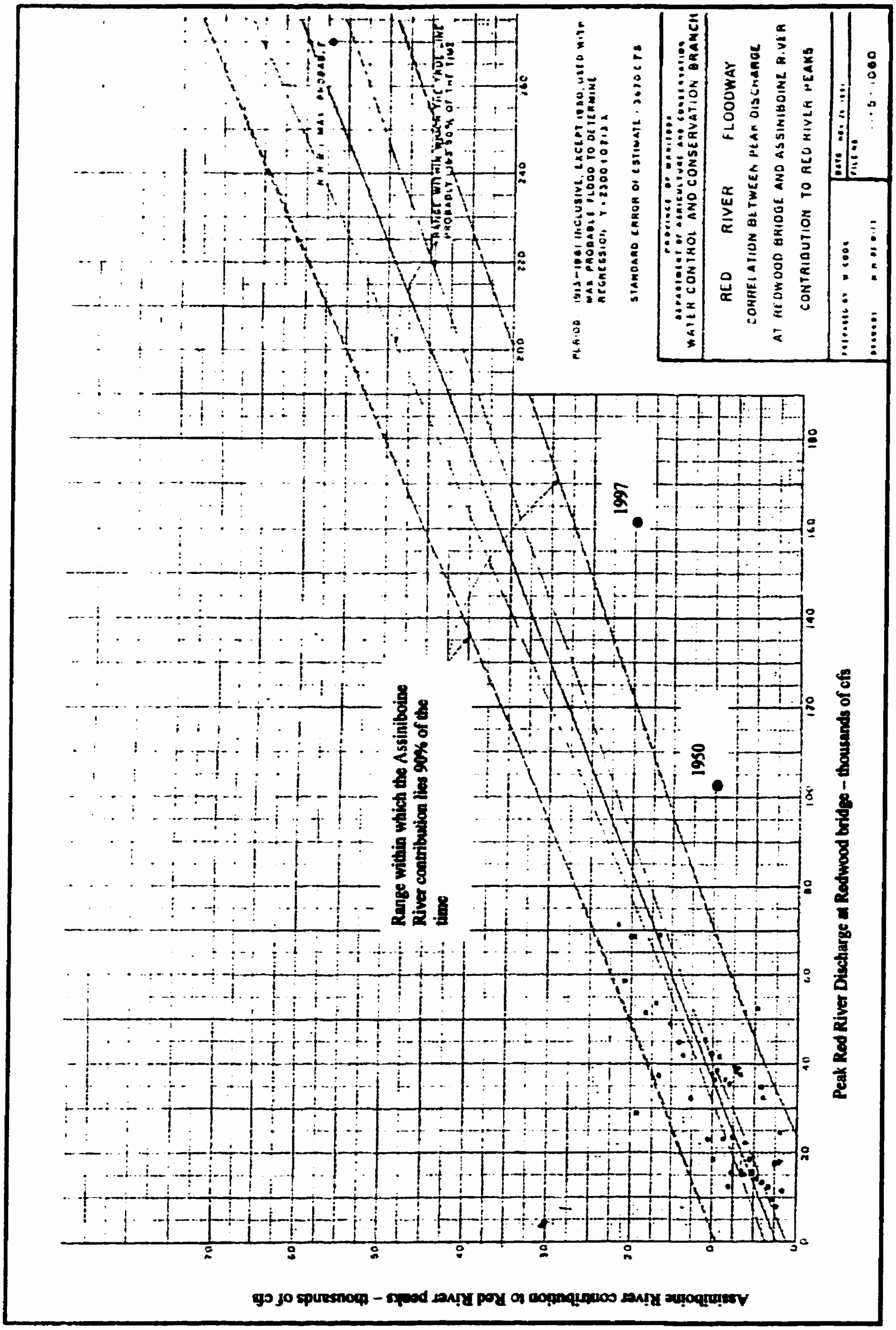


Figure 4.7: Assiniboine River contribution to Red River peaks (Long, 1970)

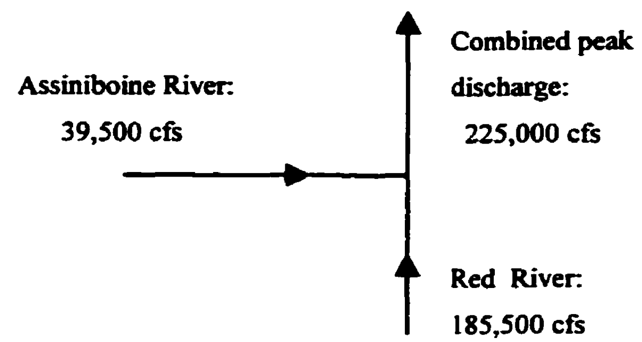


Figure 4.8a: Assiniboine River flows – Lower

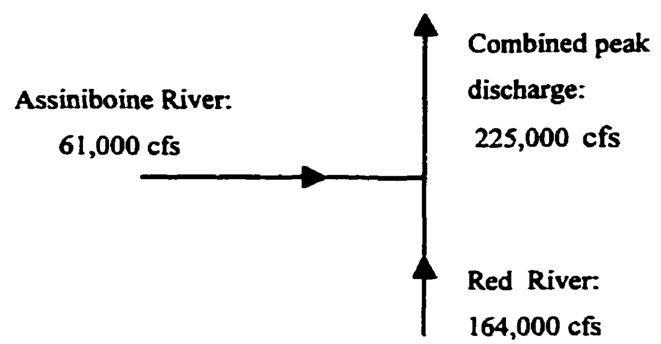


Figure 4.8b: Assiniboine River flows – Upper

Figure 4.8 Potential flow distribution for an 1826 flood event.

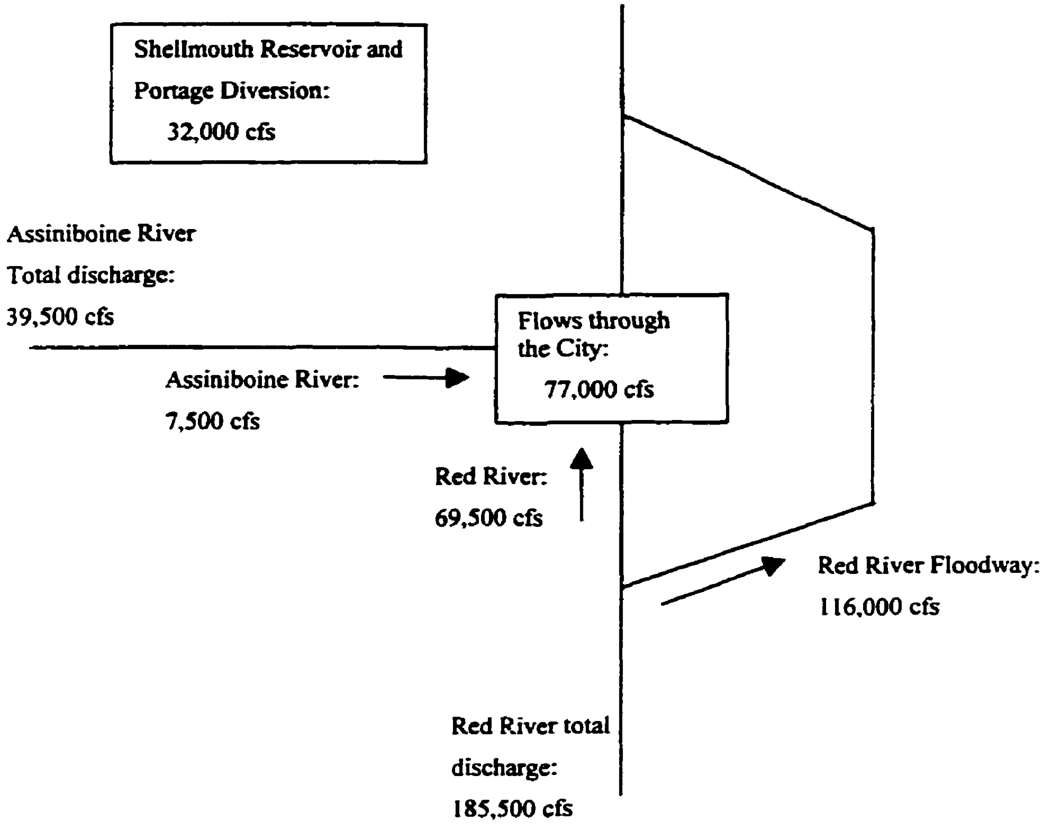


Figure 4.9 Assumed flood control works operation for an 1826 flood event.

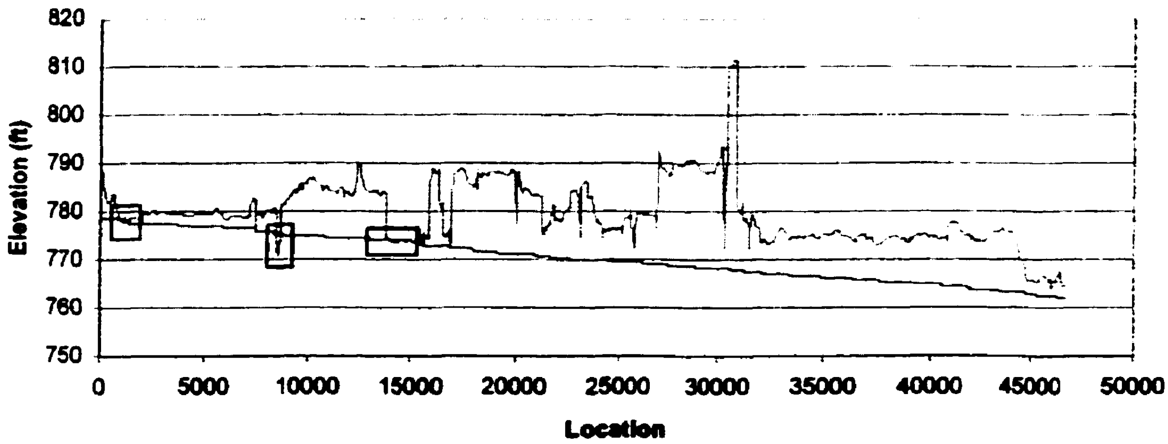


Figure 4.10 West embankment: water surface profile (Q = 116,000 cfs).

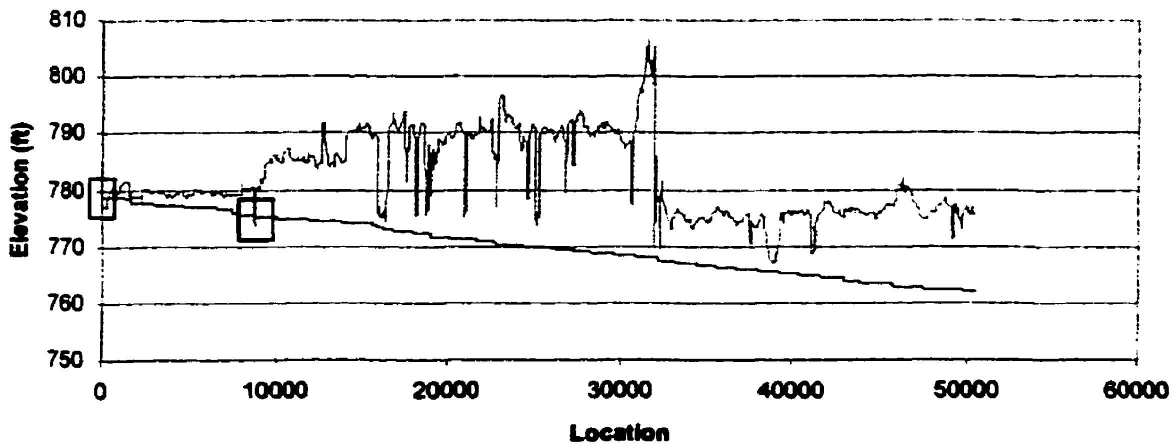


Figure 4.11 East embankment: water surface profile (Q = 116,000 cfs).

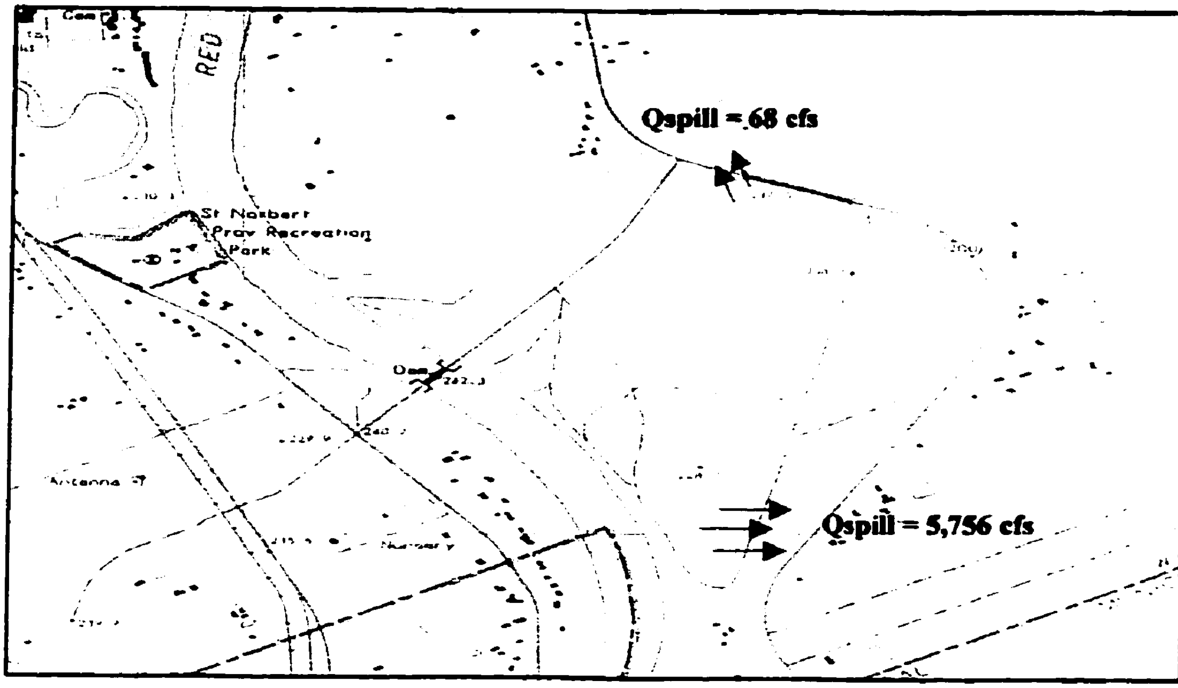


Figure 4.12 Section from Inlet to St. Mary's Road.

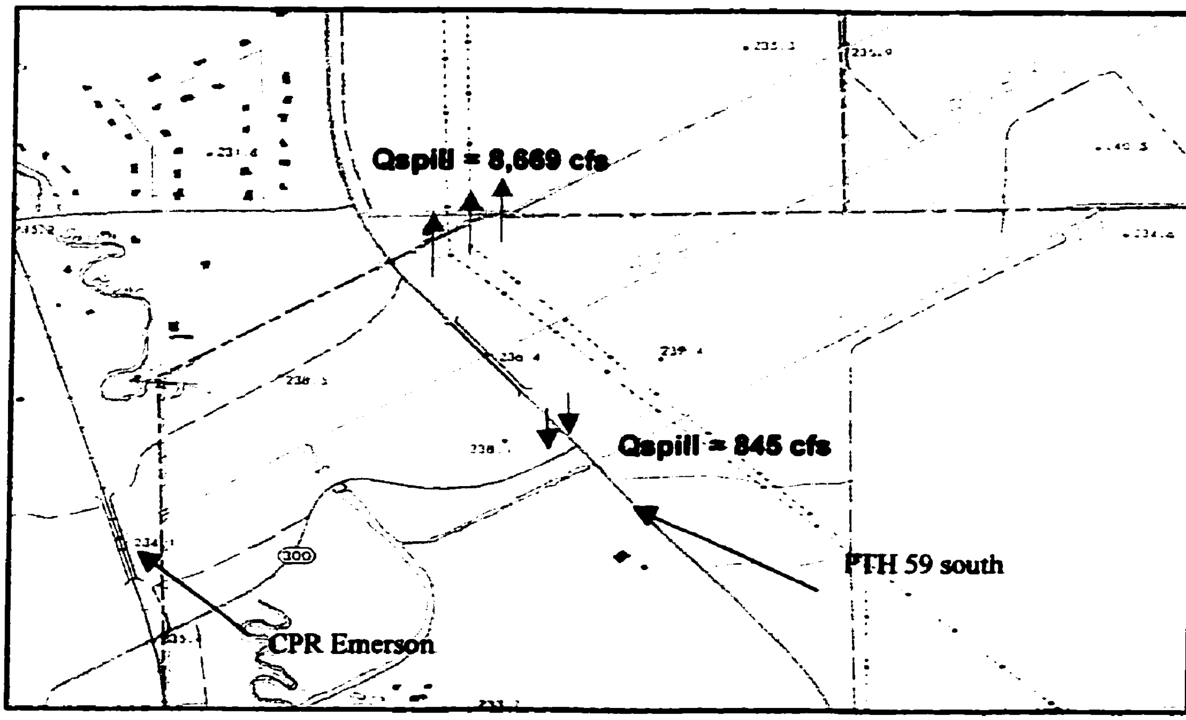


Figure 4.13 Section from CPR Emerson to PTH 59(s).

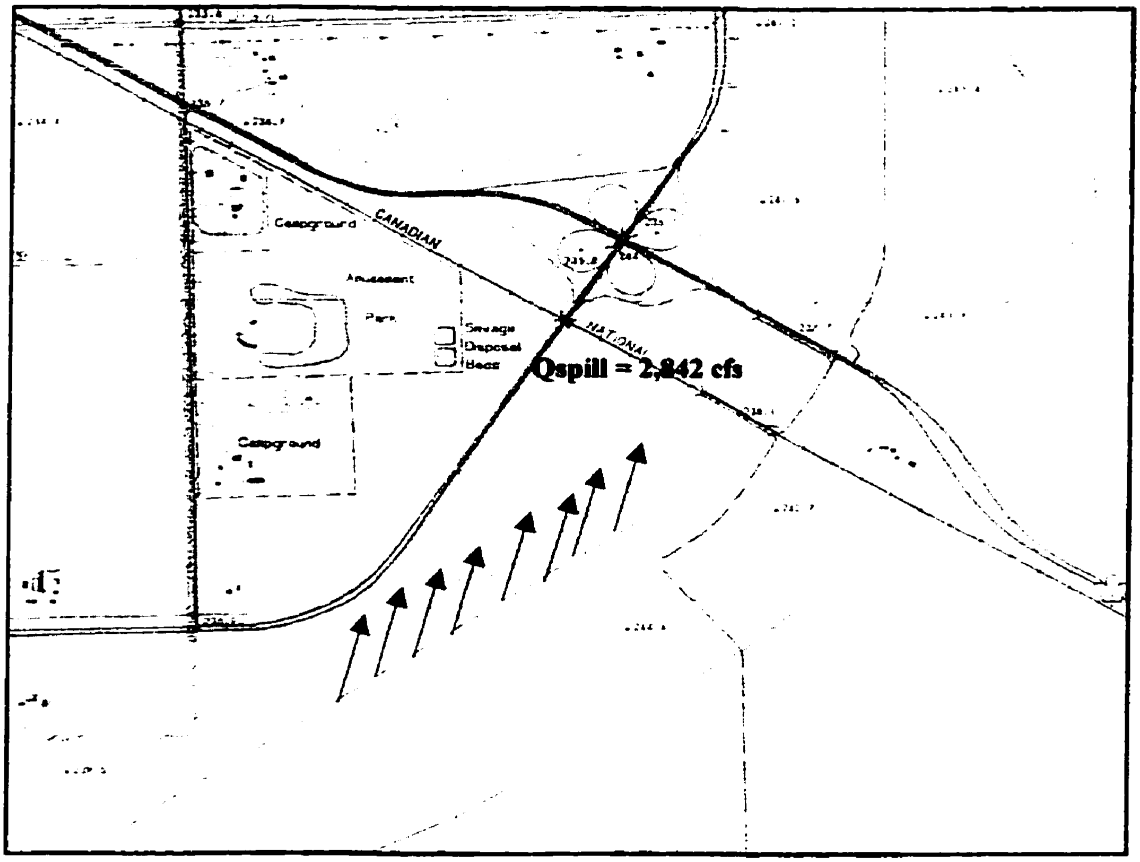


Figure 4.14 Section from PTH 59(s) to CNR Sprague.

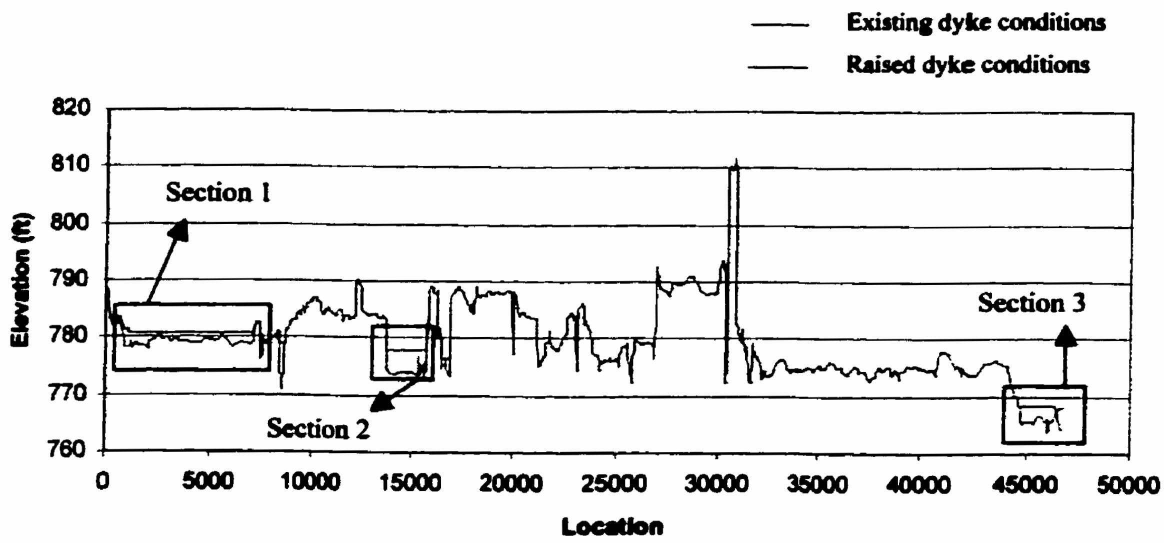


Figure 4.15 West embankment raised conditions.

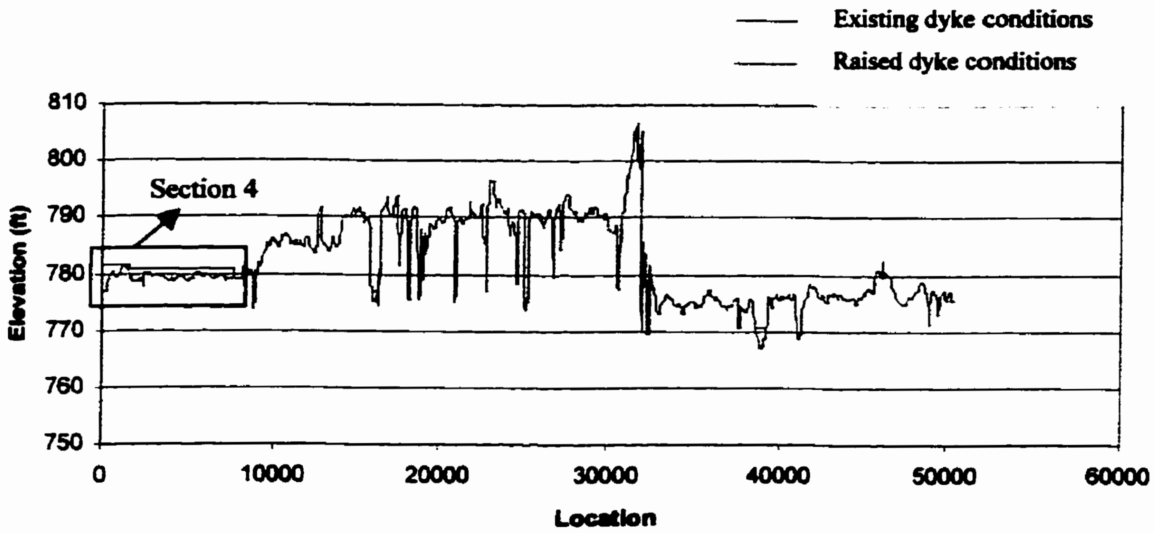


Figure 4.16 East embankment raised conditions.

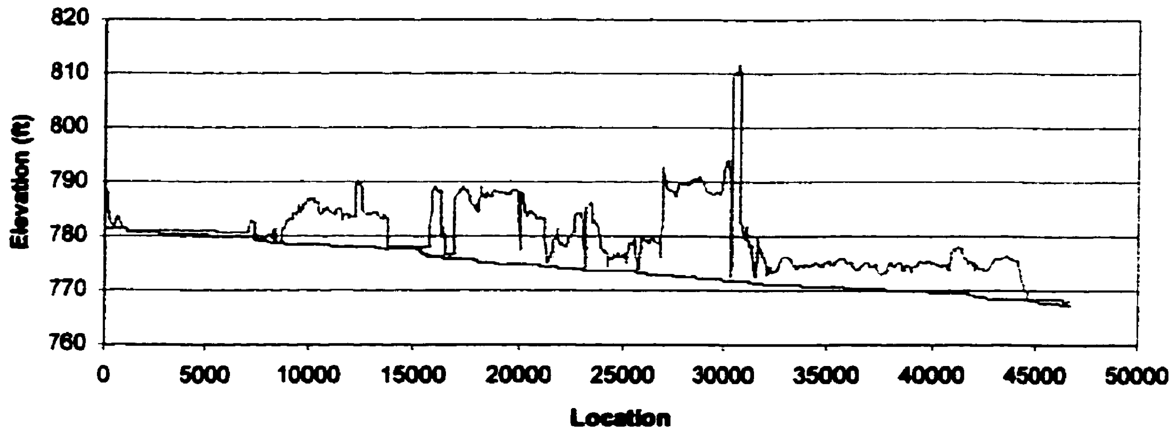


Figure 4.17 West embankment: water surface profile (confined flow).

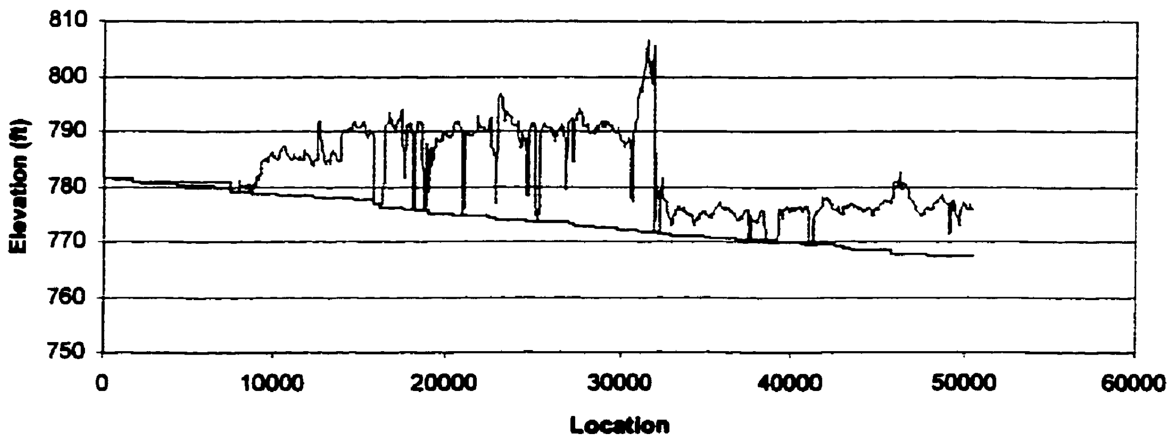


Figure 4.18 East embankment: water surface profile (confined flow).

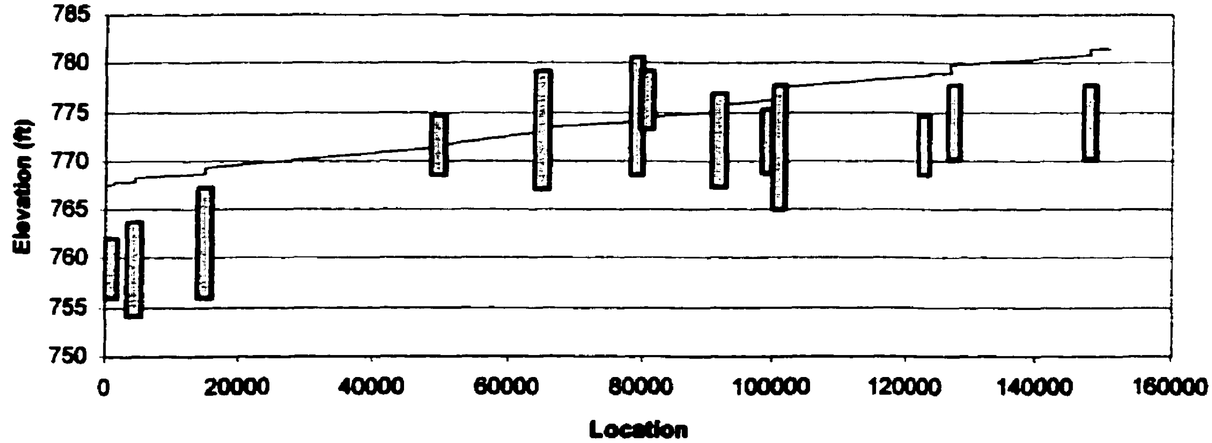


Figure 4.19 Bridge conditions for Q = 116,000 cfs (confined flow).

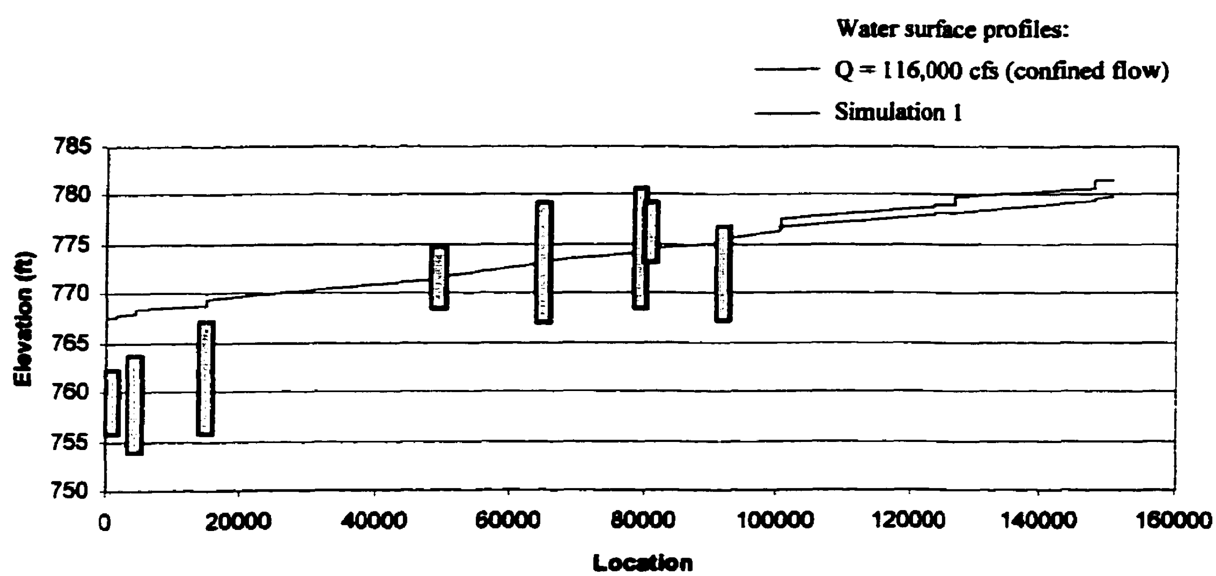


Figure 4.20 Bridge simulation 1: Removal of 5 most upstream bridges (Q = 116,000 cfs).

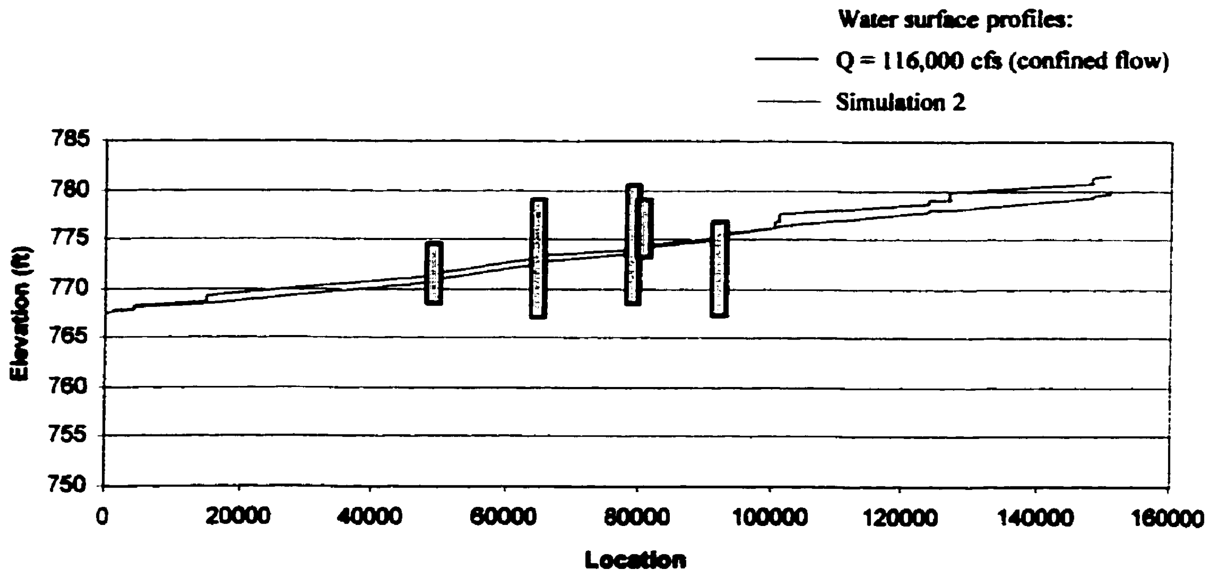


Figure 4.21 Bridge simulation 2: Removal of all submerged bridges (Q = 116,000 cfs).

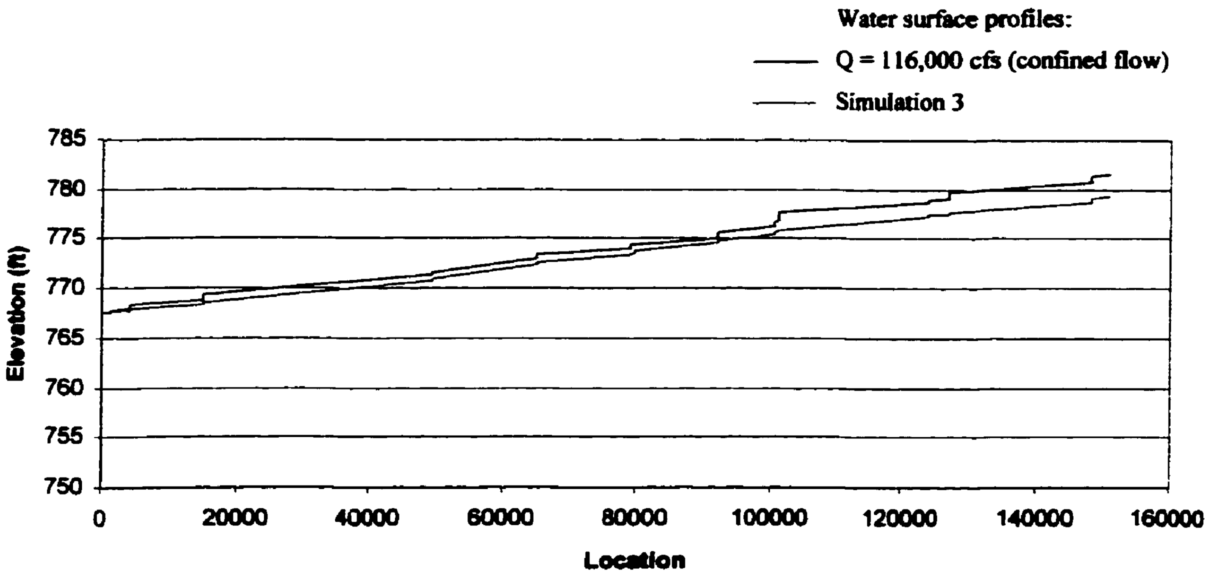


Figure 4.22 Bridge simulation 3: Removal of all bridges (Q = 116,000 cfs).

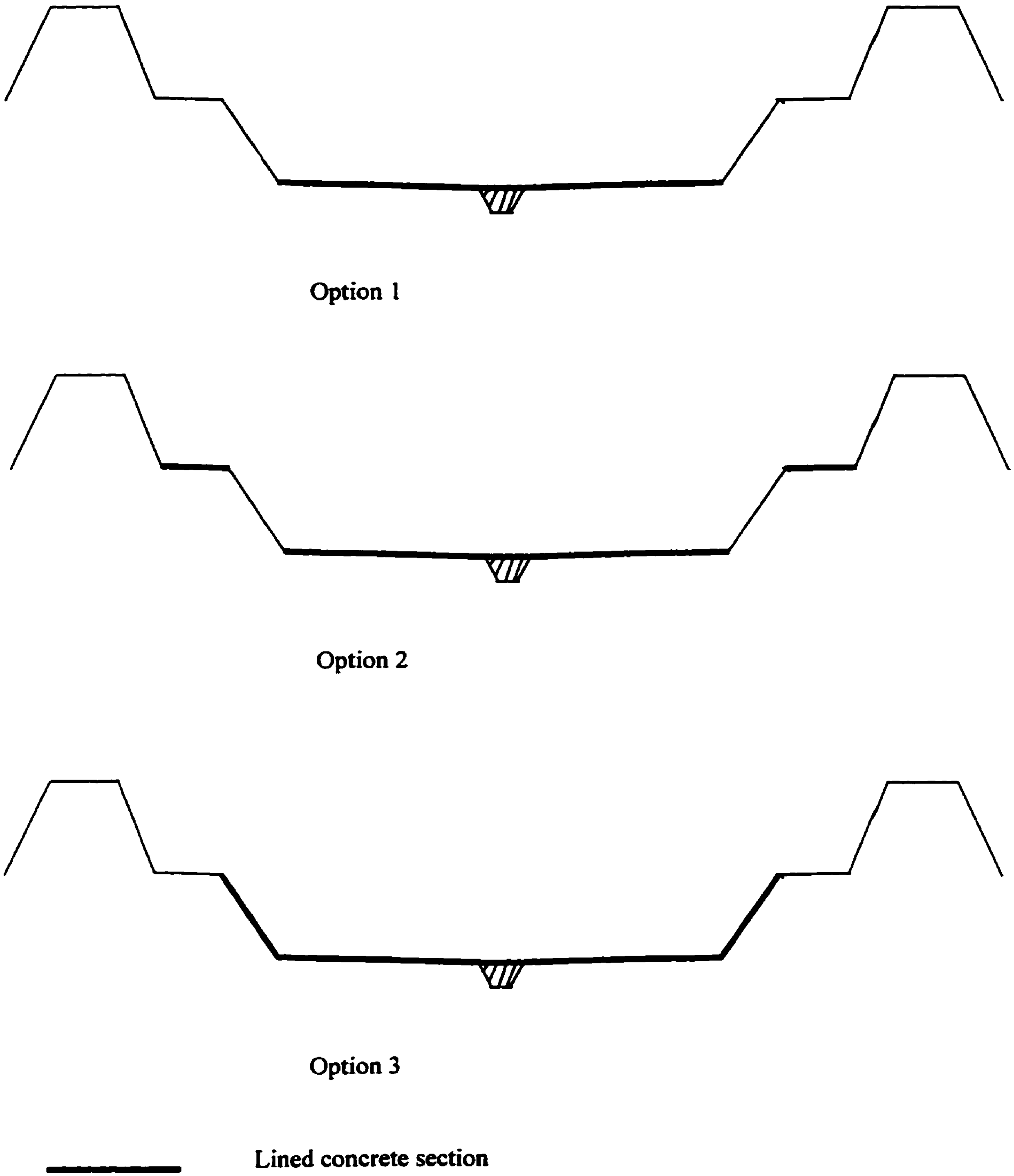


Figure 4.23 Composite cross sections.

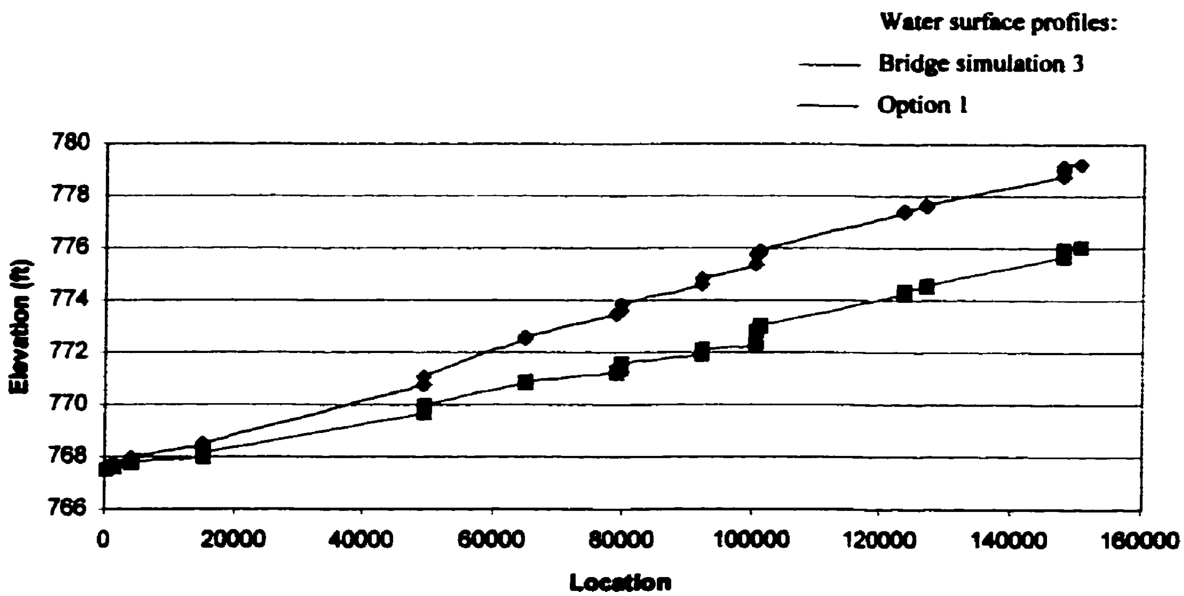


Figure 4.24 Option 1: Lining of the bottom of the channel.

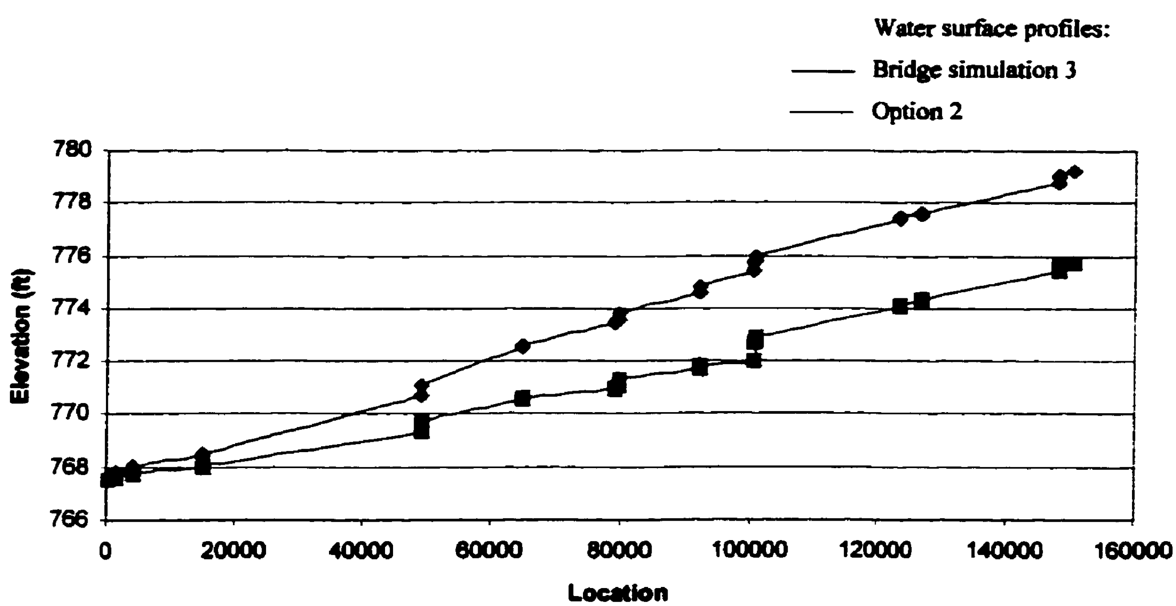


Figure 4.25 Option 2: Lining of the bottom of the channel and berms.

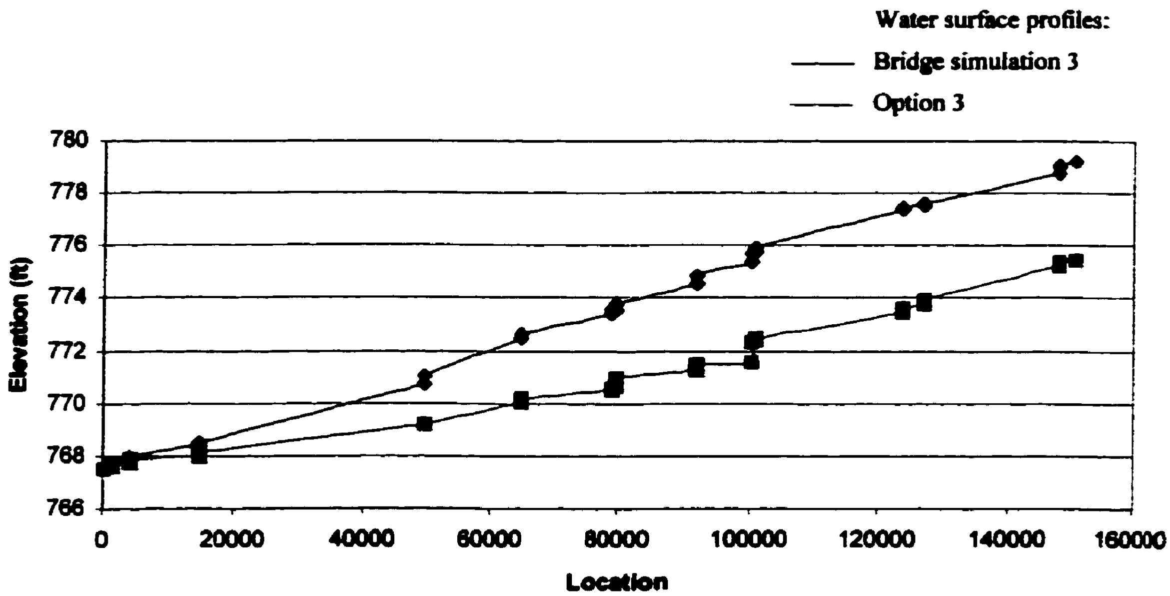


Figure 4.26 Option 3: Lining of the bottom the channel and side slopes.

Figure 4.27 Emergency retaining area. ---- emergency dyke structure.

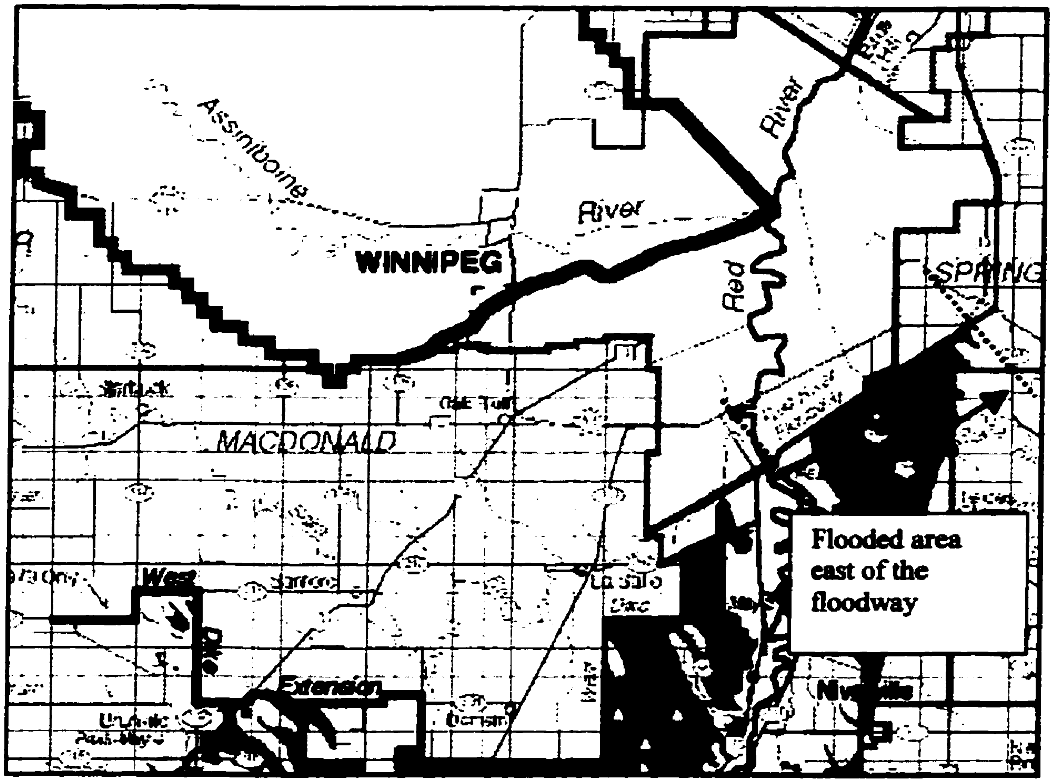


Figure 4.28 1826 flood extent of inundation.

CHAPTER 5

Conclusions

5.1 Introduction

Throughout its history the City of Winnipeg, located at the junction of the Assiniboine and Red River, has been susceptible to flooding. Some of the recorded flood events have had catastrophic results. The worst flood damage of this century was experienced in 1950. This event resulted in 1/8 of the city being under water, flooding more than 10,000 homes. Flood damages were over $75M (1995 dollars). This was the event that triggered the design and construction of the flood control works: Shellmouth Reservoir, Portage Diversion and the Red River Floodway. While the Shellmouth Reservoir and Portage Diversion help to reduce flows down the Assiniboine River, the Floodway is the only component of the system that offers immediate flood relief to the City of Winnipeg from Red River floodwaters. Since its construction (in 1968) the Red River floodway has served its purpose, lowering water levels in the Red River through the City and saving thousands of dollars in flood damage.

The Red River floodway was put to the test in 1997 when the flow diverted through the channel slightly exceeded the design capacity of 60,000 cfs. With a peak discharge of 65,000 cfs being diverted through the floodway, the adequacy of the channel to provide flood protection against floods of a greater magnitude became questionable. Under design conditions, the floodway channel will carry 60,000 cfs up to a maximum of 100,000 cfs. However, after the 1997 flood, it became quite obvious that 100,000 cfs would exceed the capacity of the channel. This raised questions regarding the level of protection offered by the flood control works, especially that provided by the Red River floodway.

The possibility of the City of Winnipeg facing a flood event greater than that experienced in 1997 is not far fetched. Historical records show that some of the higher floods occurred in the 1800's, with the worst event being that of 1826. The peak flow of 225,000 cfs that would result from such an event is well above the capacity of the flood control works. Under such conditions, would the City of Winnipeg be prepared to handle such a flood?, How would Winnipeg face such a challenge? In order to further investigate the above and evaluate the performance of the channel under high flows a two-dimensional finite element numerical model of the Red River floodway was established.

5.2 Summary

The numerical model of the Red River floodway was set up using the two-dimensional hydrodynamic computer package SMS, with the analysis being run using FESWMS. Modeling of the channel was further complimented using the one-dimensional hydraulic analysis program HEC-RAS. Both programs were run under steady-state, subcritical flow conditions. The HEC-RAS and FESWMS programs were used interactively to model flows in excess of the design capacity in the floodway channel. The HEC-RAS program was used to perform the analysis at the bridge structures, while the FESWMS program was used to simulate the flow in the channel between the bridges.

The 2D mesh depicting the floodway channel was formed based on 187 cross sections. In the absence of detailed topographical data of the floodway channel, the floodway cross sections were based on the dimensions of typical floodway cross sections, *viz.*, South of Birds Hill, through Birds Hill and north of Birds Hill. The data was provided by the Water Resources Branch and the Department of Highways.

The FESWMS and HEC-RAS models were calibrated using data collected during the 1997 flood. Both models calibrated quite well. The calibrated models were then used to simulate flow conditions corresponding to the maximum design flood (100,000 cfs) and an 1826 flood event. Results from these runs allowed for the identification of those sections along the dykes were spilling occurred. The magnitude of the spills has also being determined.

The simulation of an 1826 flood was looked at more closely. The flow distribution for an 1826 flood used in the analysis was based on the assumption that the Shellmouth Reservoir and Portage Diversion would operate at their maximum capacity, while Red River flows through the City would be limited to the level of protection offered by the primary dyking system (77,000-80,000 cfs). It should be noted that the above assumption is not in accordance to the floodway operating rules. The rules stipulate the raising of the primary dyking system by 4 ft, allowing more water through the City. The deviation of the approach taken in the analysis of the 1826 flood from the floodway operating rules is considered justifiable based on recent experience from the 1997 flood. Although Winnipeg was successful in its flood fighting effort in 1997, the amount of planning and organization that took place was overwhelming. It is considered that for an 1826 event the risk of relying on emergency preparations would be far too great. It was determined that for an 1826 event a peak discharge of 116,000 cfs would need to be diverted through the floodway channel. This peak discharge was simulated through the channel under several alternatives. The alternatives include consideration of existing conditions of the floodway dykes, raising the crest elevation of the floodway dykes at several locations, removal of bridge structures and use of composite cross sections throughout the channel.

5.3 Conclusions

The interactive analysis of the floodway channel was successful with the use of both the 1D HEC-RAS and 2D FESWMS programs. It was found that the transfer of results from one program to the other was easily accomplished. One of the areas where the programs differed greatly from one another was in the requirement of the input data. The 2D model although more accurate requires more effort in terms of preparing the input data and is more susceptible to instability leading to non-convergence. Although FESWMS has the capability to check the mesh for ill-formed elements, it was considered that it did a very poor job at detecting causes of instability once the program started running. Refining the mesh is not always the solution to mesh instability. While it sometimes may solve the problem, it increases the size of the mesh, therefore requiring more computational effort. The manipulation of the graphical data in the FESWMS program was found to be somewhat limited. The graphical interface although quite helpful for the visualization of results within SMS, has very limited compatibility with other programs. This makes the preparation of the data for presentations purposes a time consuming process.

The HEC-RAS program does not require nearly as much detail as the 2D program. Unfortunately, the one dimensionality of the program does not allow an accurate analysis of what happens when the capacity of the channel has been exceeded and its banks overflow. Also, the program offers a more simplified view of the flow patterns with more complex channel geometry. For example, the velocity distribution around a bend cannot be fully appreciated in HEC-RAS. The same is applicable to variations in water levels within the same cross section.

The FESWMS and HEC-RAS models were calibrated using data collected during the 1997 flood. In general, the HEC-RAS model calibrated better than the FESWMS model, with a maximum discrepancy of 3.8" being observed between the measured and computed data. The calibrated FESWMS model values ranged within $\pm$ 0.5 ft of the field measurements. The greatest discrepancies were observed at the upstream section of the floodway, from CPR Emerson to the inlet. A two-dimensional model is highly dependent on the accuracy of the topographic and hydraulic data being used. It is speculated that an inadequate representation of the cross sectional data at the upstream end of the floodway may be affecting the performance of the model in this area.

The existing conditions of the Red River floodway were analyzed by subjecting the channel to its maximum discharge of 100,000 cfs. The simulations of this discharge through the channel indicated that the existing capacity of the floodway is slightly less. Results showed a total spill of 4,512 cfs, approximately.

The simulation of an 1826 flood event with a peak discharge of 116,000 cfs through the channel was looked at more closely. Several alternatives were explored during the simulation of such an event. Findings from the different runs have been summarize as follows:

- With a peak discharge of 116,000 cfs the floodway dykes are overtopped at several locations, at the most upstream end of the channel:

  Along the east dyke near the entrance of the channel and PTH 59(s), and along the west dyke at St. Mary's Rd., and PTH 59(s) – CNR Sprague
- With a peak discharge of 116,000 cfs, the magnitude of the spill is of approximately 18,182 cfs. Roughly 64% of this amount spills over to the west side
- Raising of the dykes so as to confine the flow within the channel and avoid any spills results in a water surface elevation of 781.52 cfs at the inlet
- Confinement of the flow creates pressure and submerged conditions at the bridge structures. 8 out of the 13 bridges crossing the floodway channel would be completely

submerged. These have been identified as St. Mary's Rd., CPR Emerson, PTH 59(s), CNR Sprague, PTH 1, CPR Lac du Bonnet, CNR Pine Falls and PTH 44.

The high water levels observed throughout the channel are of great concern not only at the bridge structures, but also at the inlet where the crest elevation of the dykes protecting the surrounding area is comparable to the computed water levels, if not lower.

The removal of bridges was looked at as an option to increase the channel capacity. The results showed that with the removal of all bridges water levels at the inlet are lowered by 2.3 ft.

The use of composite cross sections throughout the floodway greatly increases the channel's existing capacity. It was found that reducing the channel's roughness coefficient from 0.0215 (calibrated Manning's n) to 0.012 (use of concrete lining) lowered water levels at the entrance of the channel by more than 3.0 ft. Of the three options examined, the most significant results were obtained from option 3: lining of the bottom of the channel and the side slopes. This option lowered water levels (at the floodway entrance) from 779.26 ft (n = 0.0215) to 775.51 ft (n = 0.012).

5.4 Recommendations for future work

The fact that the accuracy of any numerical model depends on the accuracy of the input data cannot be stressed enough. In general, only an accurate representation of the topography of the area to be studied and other details describing the conditions present can lead to reliable, well-documented results.

It is considered that the floodway channel plays a major and important role in providing flood protection to the City of Winnipeg. A role that has been proven several times in the past and should continue to do so in the future. An up-to-date database of the floodway channel is necessary in order to have recent, realistic data at hand for any numerical analysis of the channel may need to be undertaken. This database would include surveyed cross sections, embankment profiles, bridge structures and hydraulic data.

The collection of accurate hydraulic data provides a history of records on which to confidently base a model that will be used to forecast a future event. The collection of hydraulic data for the floodway channel should include water level measurements, as well as velocity measurements. Strategic points for these measurements should be selected so that the collected data can provide a complete overview of the channel behaviour. The floodway channel has 13 bridge crossings. These structures affect the flow patterns within the channel and therefore should not be ignored, *i.e.*, the collected data should not overlook the impact these structures have on the channel flow.

The usefulness of an active and current database would prove to be unmeasurable, providing the necessary input data required for numerical model studies of the channel. Both the 1D and 2D models used in this thesis could easily be brought up to date, improving greatly the performance of the models, especially at the upstream end of the channel where most of the discrepancies were observed.

SHORT TERM MEASURES

The realization that a flood event greater than that experienced in 1997 can occur at any time is the first step toward evaluating, planning and possibly redesigning of the existing Red River floodway. It is considered that under emergency conditions there is little reaction time and therefore the limited courses of actions that can be taken have a very high risk of being carried out unsuccessfully. However, raising of the floodway dykes is a possible alternative that would help contain the flow within the channel. Containment of the flow would inevitably lead to higher water levels throughout the channel. Consideration should be given to the presence of the bridge structures under such conditions. While the removal of bridges is quite possible a more detail study of this option would be beneficial. A closer look at what this process would involve, considering equipment, cost and time factors; as well as the socio-economic impact this alternative would have on the City is recommended.

LONG TERM MEASURES

The results presented in this thesis have shown that the existing capacity of the Red River floodway is inadequate for a flood event larger than that experienced in 1997. Long term preparedness, although more difficult to establish, is the only means of providing adequate and reliable flood protection. An inspection and assessment report carried out in February, 1997 indicated the need to implement a $2M rehabilitation and upgrading work to the inlet control structure (Manitoba Water Commission, 1998). Although formal maintenance programs for the Winnipeg flood control works should be an integral part of any flood-proofing plans, a re-evaluation of the maximum design flood event and subsequent planning and designing of the existing flood infrastructure to accommodate the new parameters is necessary. Recommendations given by the Manitoba Water Commission in its Interim Report (March, 1998) include the need to establish modeling tools to "determine the overall impact of newly planned infrastructure in the Valley on water elevations and flow patterns". This can be extended to future studies and analysis on the Red River floodway considering various modifications to the existing channel in order to increase its conveyance.

Finally, the involvement and cooperation of the public is considered to be of great importance in order to provide a level of flood protection acceptable to all involved. The public needs to understand the importance and operation of the flood control works, as well as their limitations.

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APPENDIX A

The Finite Element Method

A.1 Introduction

The Finite Element Method (FEM) was developed to address the ever increasing need to solve the non-linear partial differential equations that formed the foundation of many engineering problems. The use of the method can be traced back as far as 1943 to the mathematician R. Courant (Brauer, 1988). Although it was originally developed to be applied to structural analysis problems in the aircraft industry, the use of the finite element method has rapidly extended to other areas such as thermal, electromagnetic and fluid flow analysis.

The FEM is based on the idea of using piecewise continuous functions to approximate the unknown solution for a given domain. This is accomplished by subdividing a large, complex body into several, smaller regions. Each individual region, called an element, is described by a set of equations. The set of equations for all elements can then be assembled into a matrix form to be solved, hence providing a solution for the entire domain. Figure A-1 shows the subdivision of a given domain into a finite number of elements.

The concept is summarized in Chapra & Canale (1988) using the following procedure:

1. Discretization
2. Element equations
3. Assembly
4. Boundary conditions

5. Solution

The discretization step involves the formation of nodes and elements within the area/body under analysis. The elements are the small, homogeneous regions within the larger domain. The nodes are the points at which the elements are connected. The variation of the quantity being modeled is described within each element by an interpolation function. The value of the function is defined at every node. The solution to the problem involves determining these nodal values.

Once the interpolation function has been chosen, the element equations need to be determined. The element equations determine the relationship between the unknowns. There are several methods that can be used to derive the element equations. Among the most common are:

1. the direct method,
2. the variational method, and
3. the weighted-residual methods.

The application of each method depends on the principles governing the problem under study. The FESWMS program solves the set of differential equations by using the Galerkin finite element method. The latter is one of the techniques used in the weighted-residual methods. The following section looks more closely at the weighted-residual method.

A.1.1 The method of weighted residuals

The method of weighted residuals is a technique used to solve linear and non-linear partial differential equations by defining the dependent variable in the partial differential equation in terms of some unknown parameter. The method solves for the unknown parameter so as to provide an approximate solution. The assumed value of the parameter must satisfy the differential equation and the boundary conditions imposed on the problem. The substitution of the assumed value into the partial differential equation does not provide an exact solution, therefore introducing the presence of residuals. The accuracy of the approximation depends on the elimination of the residual.

Froehlich (1987) explains the above method by considering the following partial differential equation

$$\underline{L}U - f = 0, \tag{A.1}$$

where $\underline{L}$ refers to a differential operator, U is a dependent variable and f is a known function. It is assumed that the dependent variable can be defined in terms of an unknown parameter, C, and a set of functions, N, as given by

$$U = \overline{U} = \sum_{i=1}^{n} N_i C_i . \tag{A.2}$$

The partial differential equation can then be expressed in terms of C_i and N_i

$$\underline{L}\left(\sum_{i=1}^{n} N_i C_i\right) - f = \varepsilon, \tag{A.3}$$

where ε is the residual error of the approximate solution.

An approximate solution is obtained by solving for the unknown parameters C_i in such a way so as to keep the residual error ε as small as possible. The method of weighted residuals accomplishes this by expressing a weighted average of the error in terms of weighting functions. The weighted average of the error is required to vanish when integrated over the entire domain

$$\int_r W_i \varepsilon \, dr = 0 \qquad \text{for } i = 1, 2, \ldots\ldots n, \tag{A.4}$$

where r refers to the entire domain and W is a weighting function.

There are several weighted residual methods. They all differ based on the choice of weighting functions. The most commonly used method is the Galerkin's method. In this method the weighting functions are the same as the interpolation functions, *i.e.*,

$$W_i = N_i.$$

Therefore, the approximate solution given by the Galerkin's method is as follows

$$\int_r N_i (\underline{L}\overline{U} - f)\, dr = 0 \qquad \text{for } i = 1, 2, \ldots\ldots n. \tag{A.5}$$

An expression such as equation A.5 is formed for each element in the system. The solution for the entire system is obtained by combining the resulting expressions into a set of algebraic equations that can then be solved.

A.1.2 The Finite Element Method (FEM) vs. The Finite Difference Method (FDM)

As previously stated, the Finite Element Method is a "piecewise approximation" approach in which the solution for the domain under analysis is obtained by combining together the solutions from the individual elements. Very much like the above, the Finite Difference Method is based on the same concept, but with a slight variation. The FDM requires a subdivision of the entire domain into a uniform grid with a finite number of points. The partial differential equation is replaced by finite divided differences and solved based on the imposed boundary conditions. The application of the finite difference method has some severe limitations when the problem involves irregular geometries. Figure A-2 presents the finite difference grid generated on the same domain used in figure A-1.

The use of the FDM on complex and irregular shapes does not provide the flexibility of using non-uniform, non-rectangular elements, therefore restricting the accuracy of the approximated solution. There is also some difficulty in applying boundary conditions to curvilinear boundaries. The FEM has the advantage of being flexible as to the choice of grid to be used (*i.e.*, varied size and shape of elements). It also allows for greater accuracy on the approximation through the use of higher order polynomials. However, the effort required for the code formulation of such a method and the computational costs involved are greater than those required by the FDM.

Weare (1975) discusses the economic advantage of the methods in terms of the unit cost of computation. For the FEM such cost increases with the bandwidth which is related to the number of elements found in the mesh. The literature indicates that the "solution time is roughly proportional to the cube of the number of unknowns, or at least the square of the unknowns" (Brauer, 1988). It will be recalled that the number of unknowns within every element is equal to the number of nodes making up the element. Therefore, the greater the number of elements the greater the computational effort required in order to obtain a solution.

The analysis of two dimensional problems using both FEM and FDM have shown that while the former can address a wider range of problems and provide more accurate solutions, it can get very expensive and time consuming.

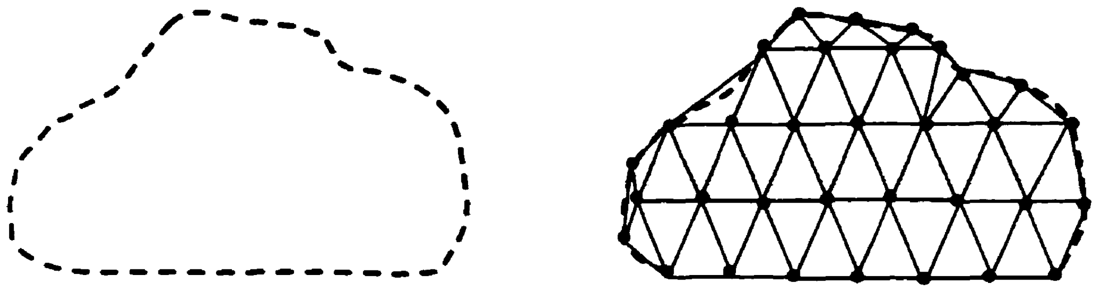


Figure A.1 Finite element grid.

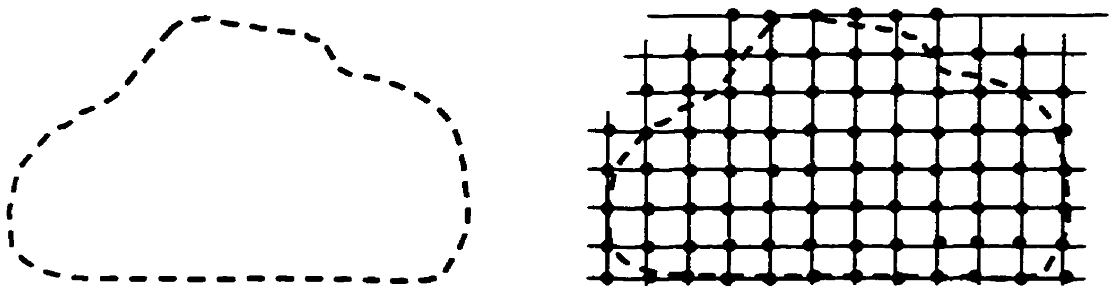


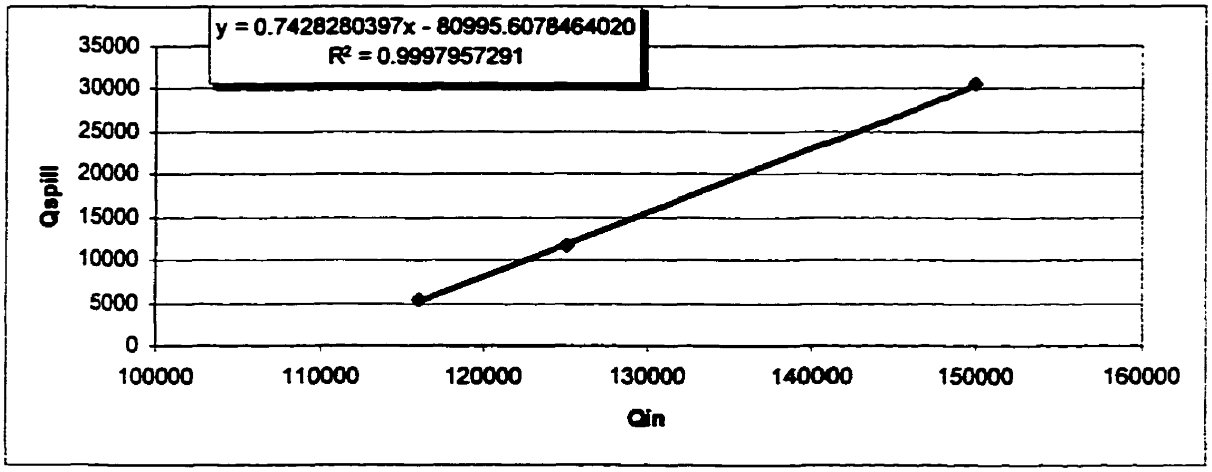
Figure A.2 Finite difference grid.

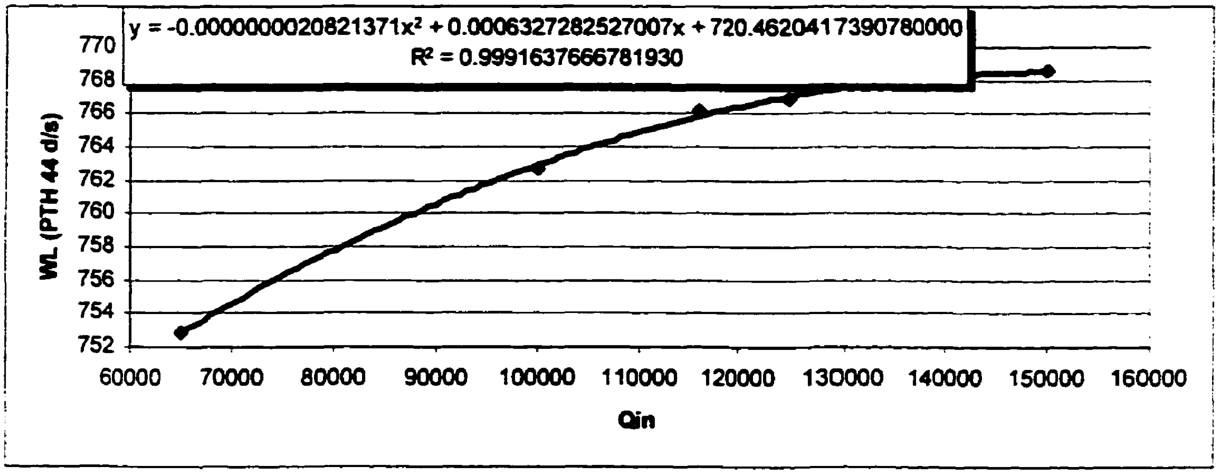
APPENDIX B

Floodway sections: Discharge – water level relationships

Data presented in this appendix refers to the relationships obtained at each of the floodway sections (as presented in Chapter 4.0, section 4.1.1) in order to evaluate the set of commensurate boundary conditions to be applied at each section.

Each section of the floodway channel was run for five different discharges, establishing for each run a relationship between the initial discharge, the water level at the downstream end of the section and the discharge going over the dykes. The range of discharges applied to each section provided five points through which a curve could be fitted in order to define the above relationships. These curves would then be used to approximate the amount of water being lost over the dykes and water levels within a section for the particular initial discharge under review, *i.e.*, 116,000 cfs.





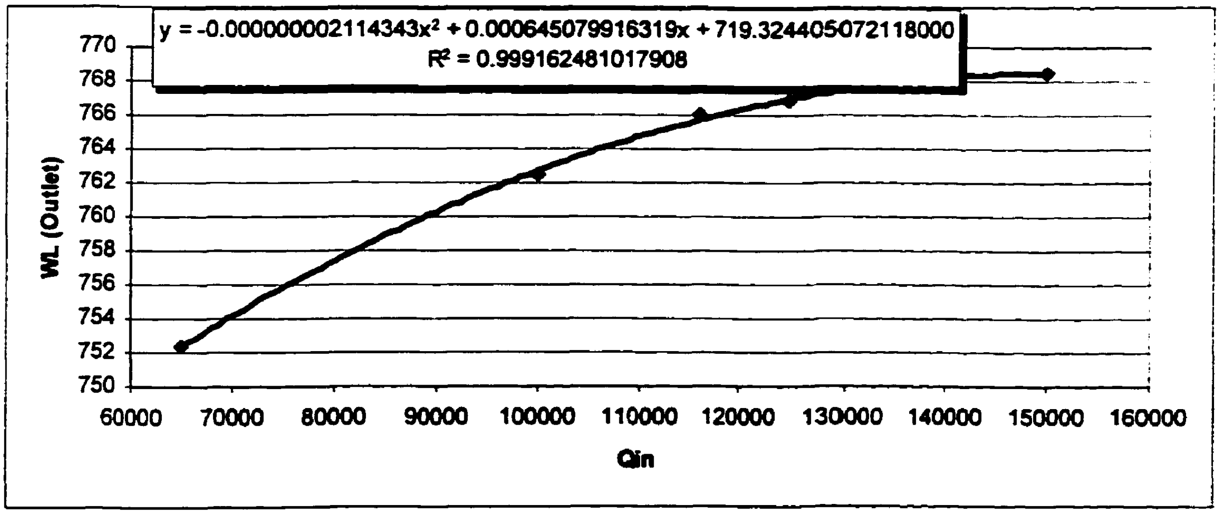


Figure B.1 Relationships for the floodway section from Outlet to PTH 44

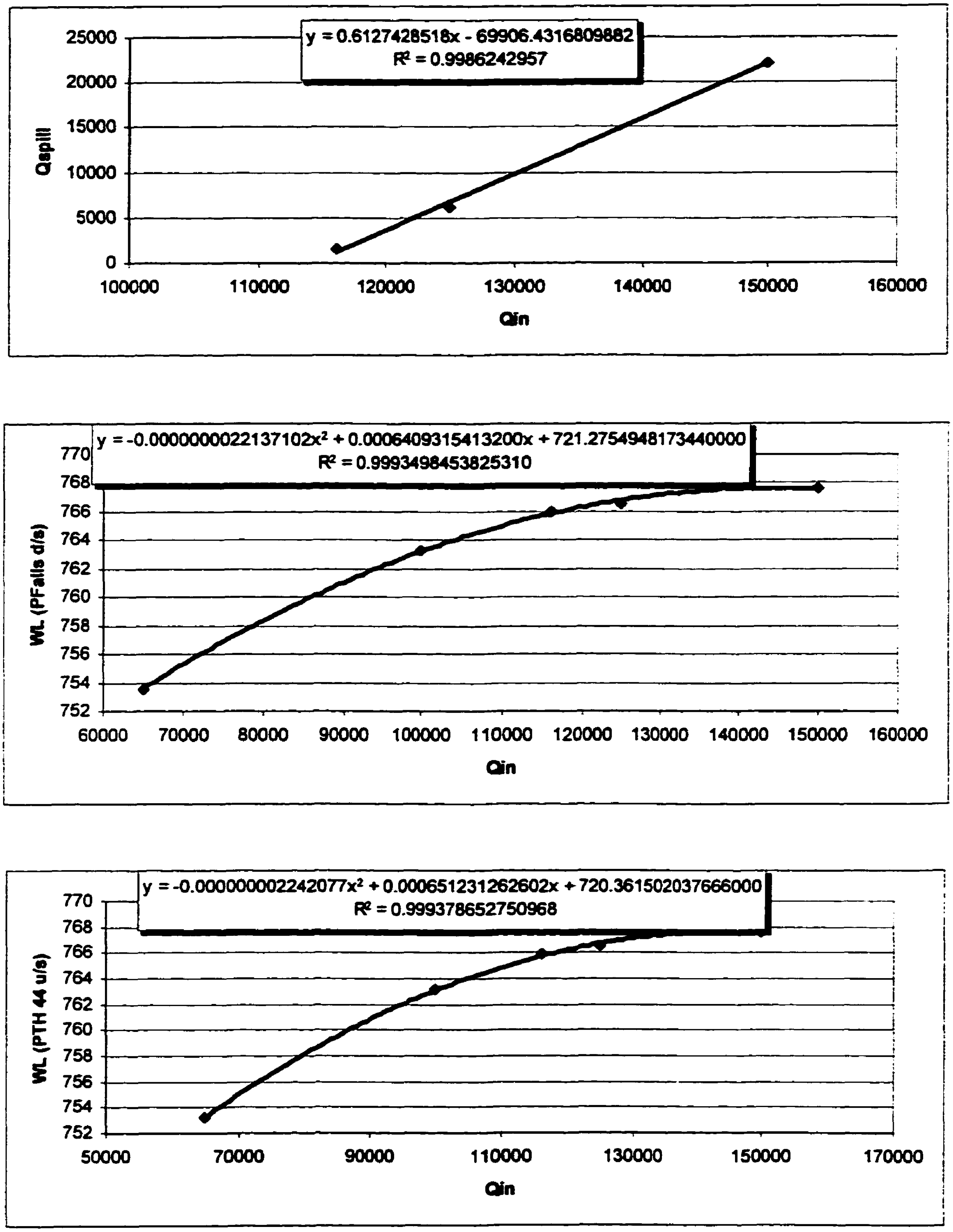
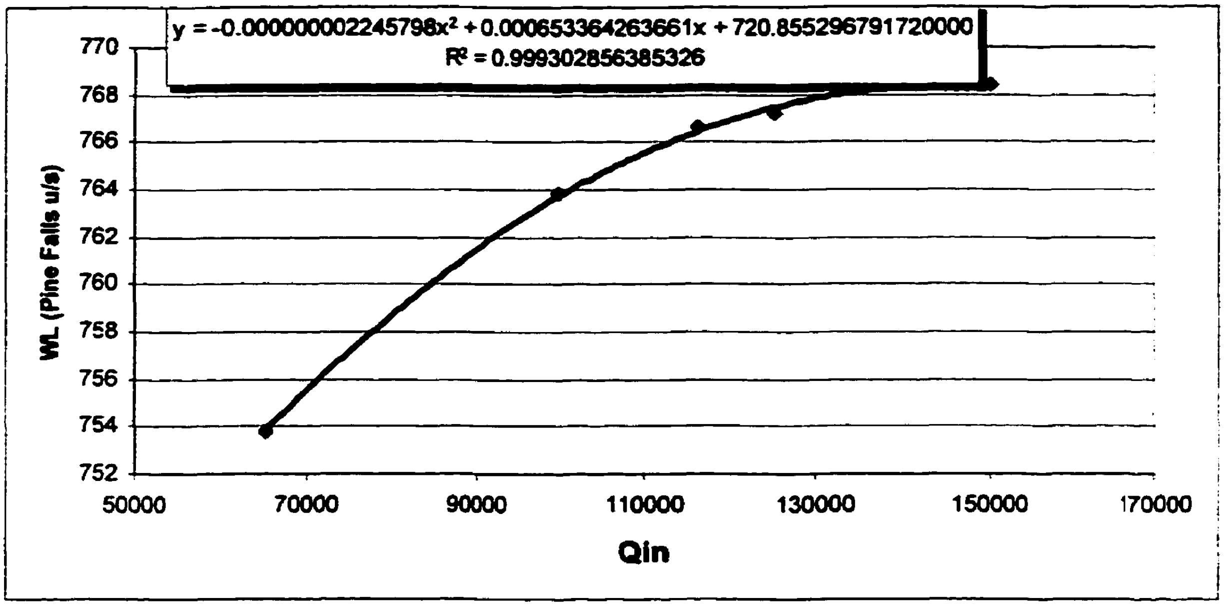


Figure B.2 Relationships for the floodway section from PTH 44 to CNR Pine Falls



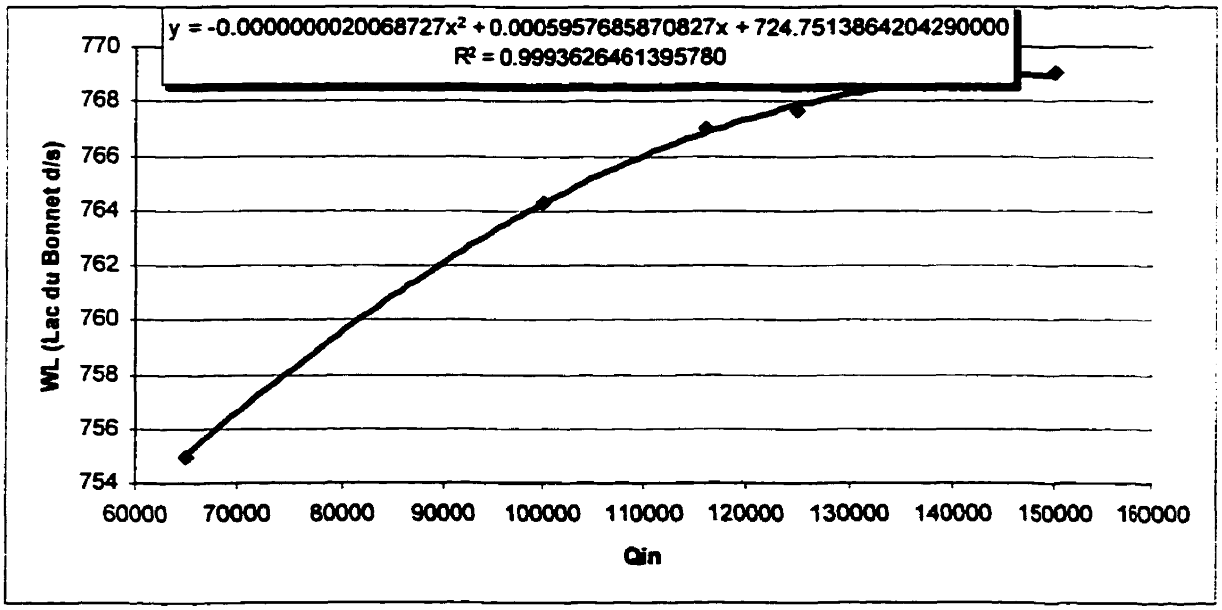


Figure B.3 Relationships for the floodway section from CNR Pine Falls to CPR Lac du Bonnet

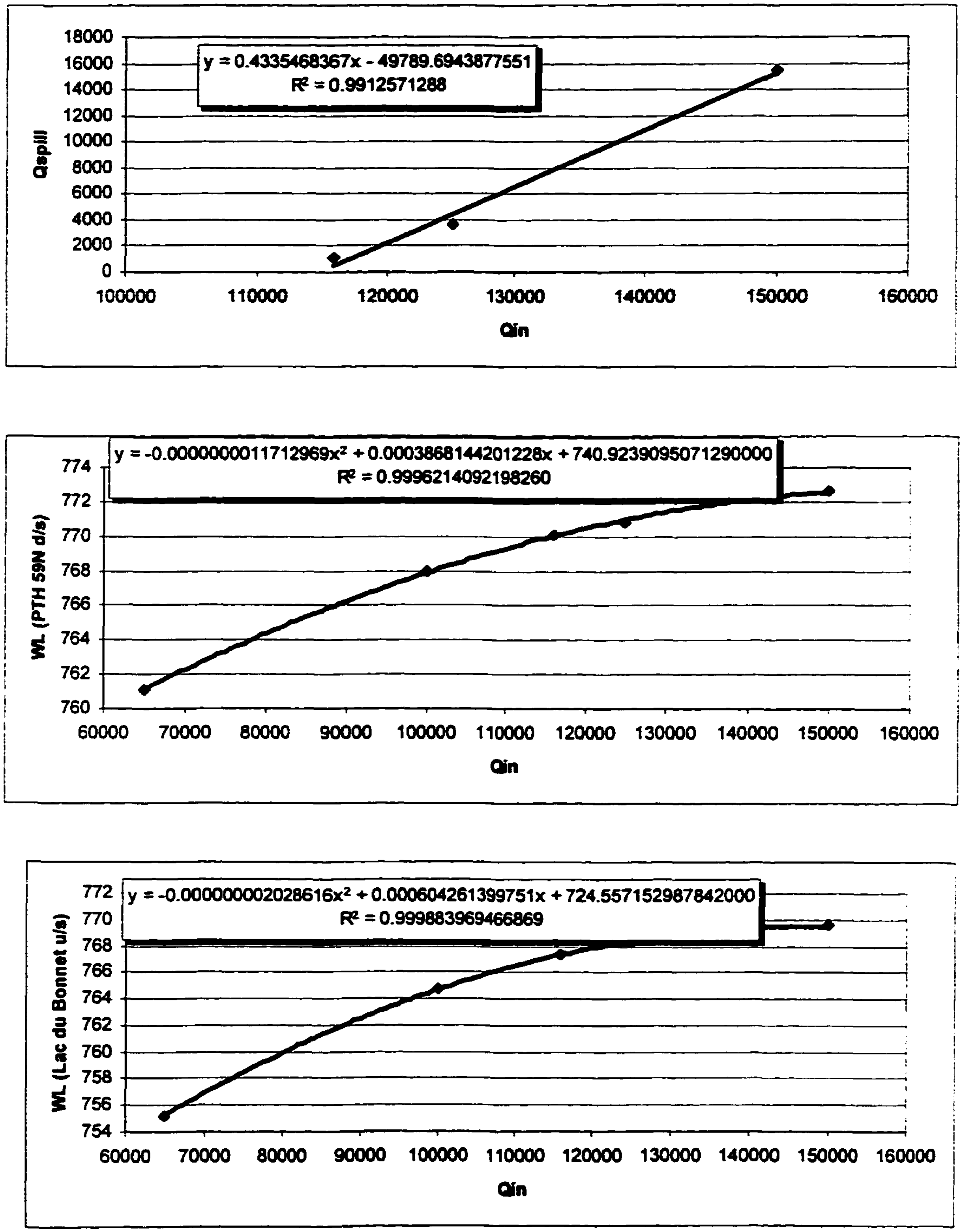
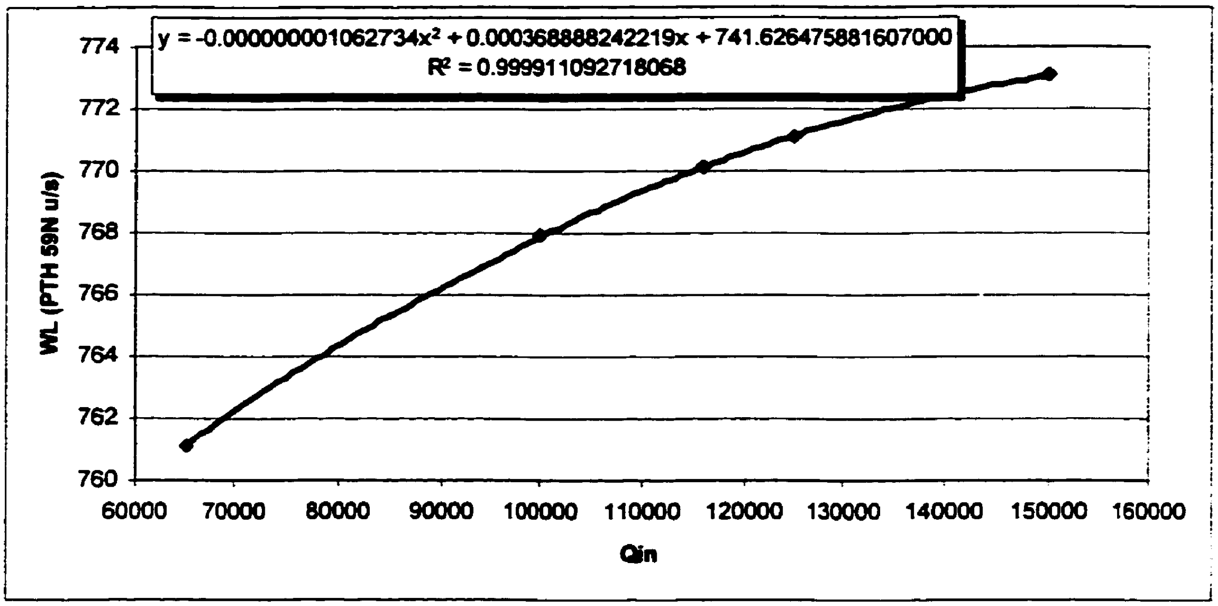


Figure B.4 Relationships for the floodway section from CPR Lac du Bonnet to PTH 59(n)



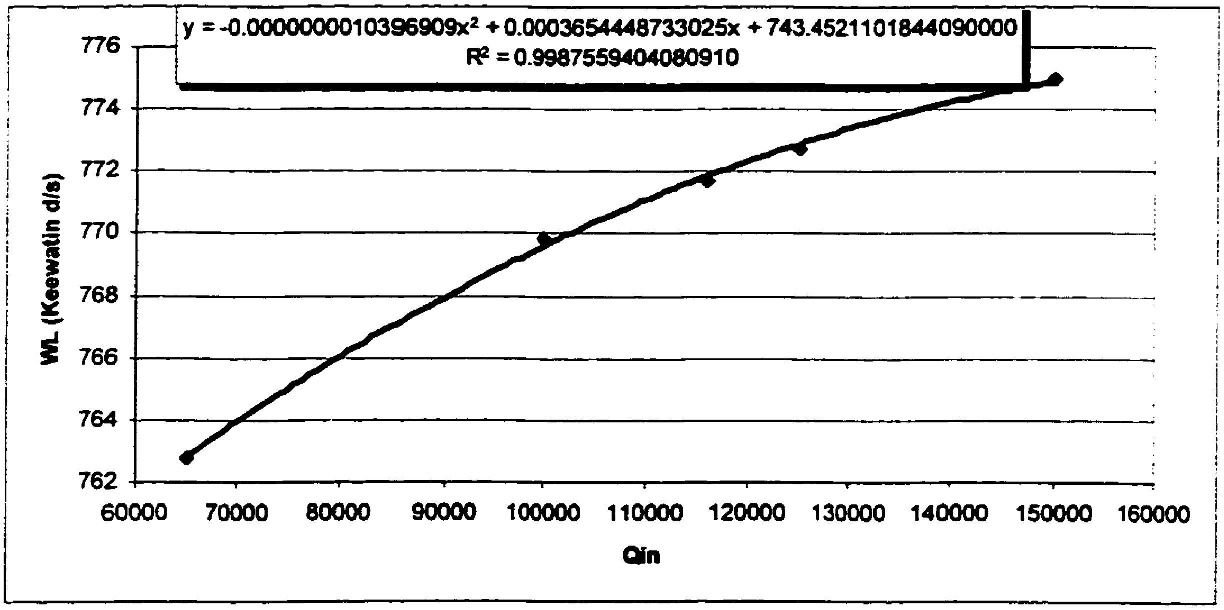
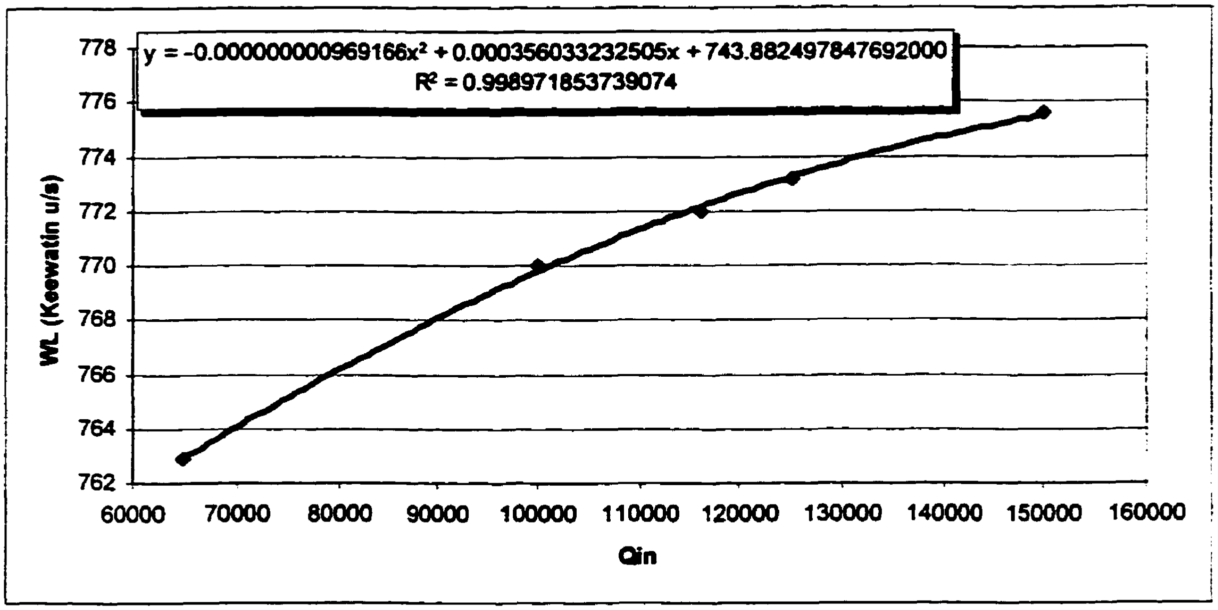


Figure B.5 Relationships for the floodway section from PTH 59(n) to CPR Keewatin



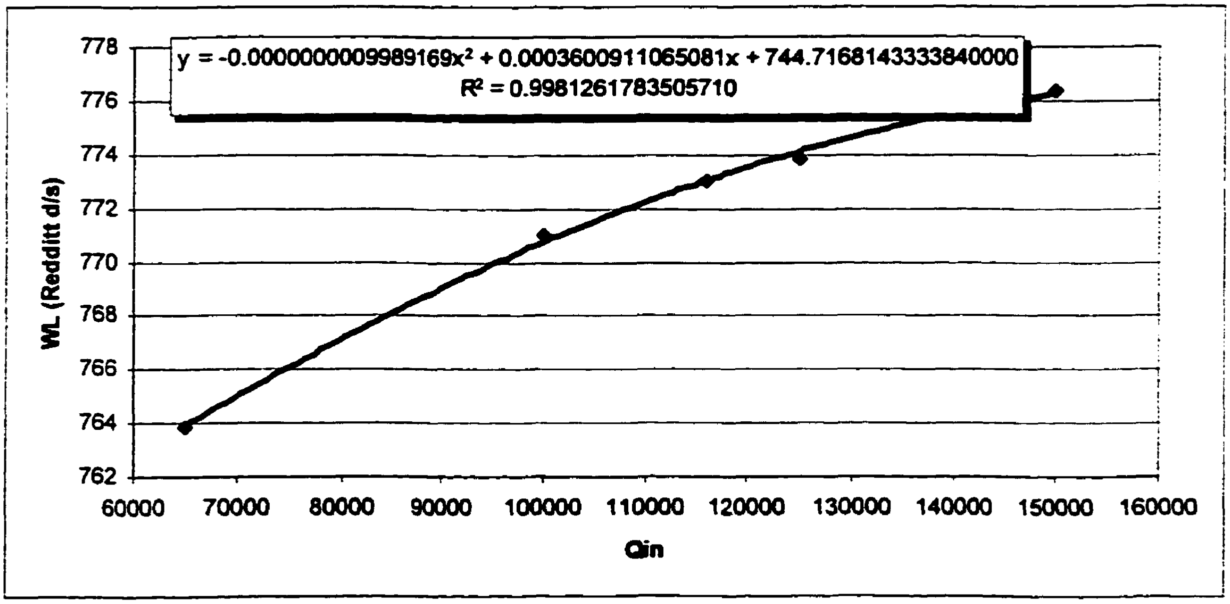
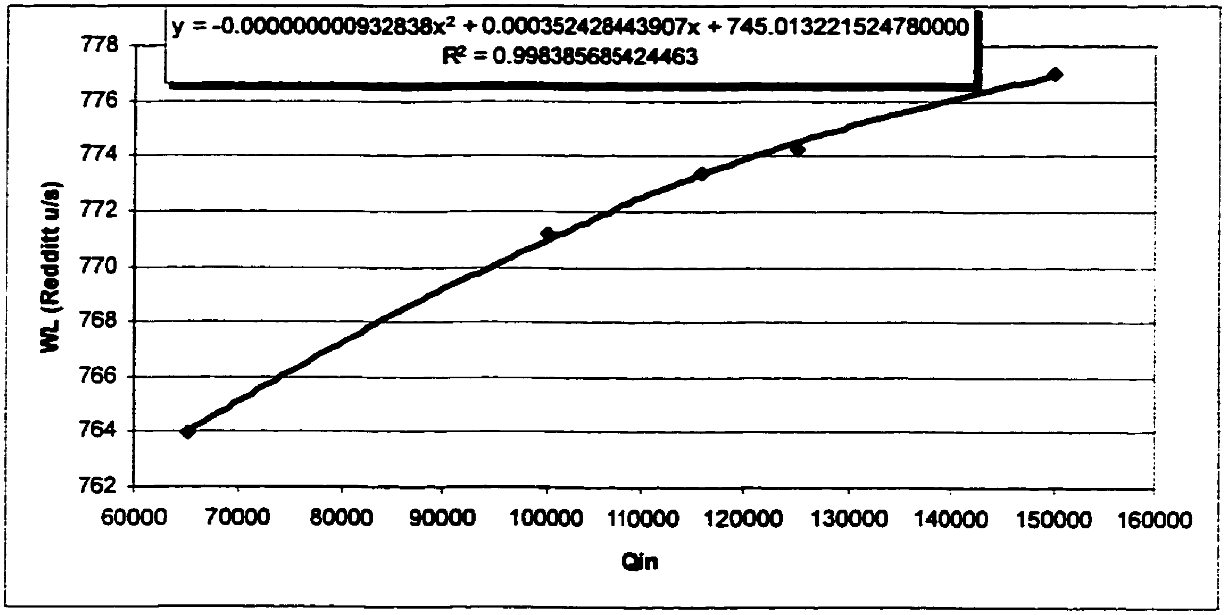


Figure B.6 Relationships for the floodway section from CPR Keewatin to CNR Redditt



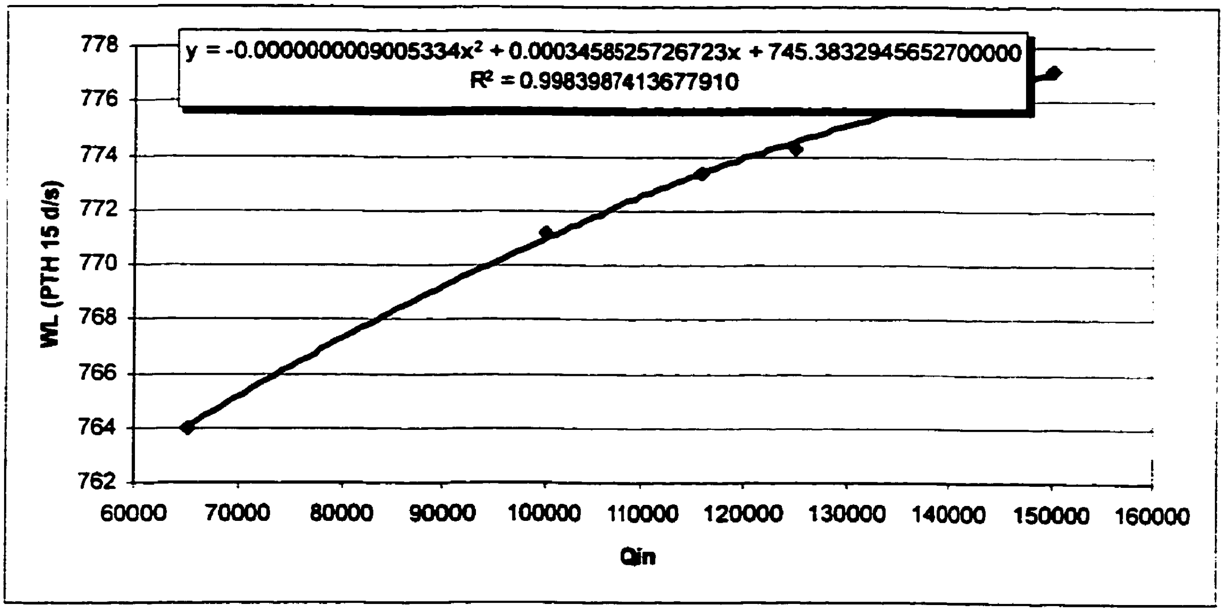
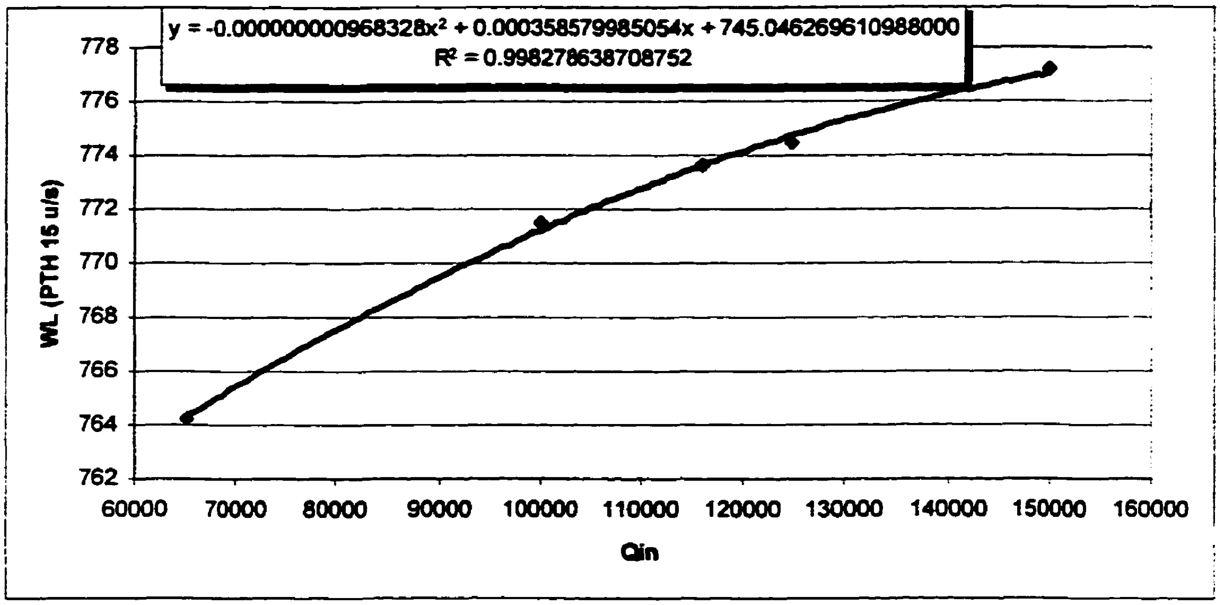


Figure B.7 Relationships for the floodway section from CNR Redditt – PTH 15



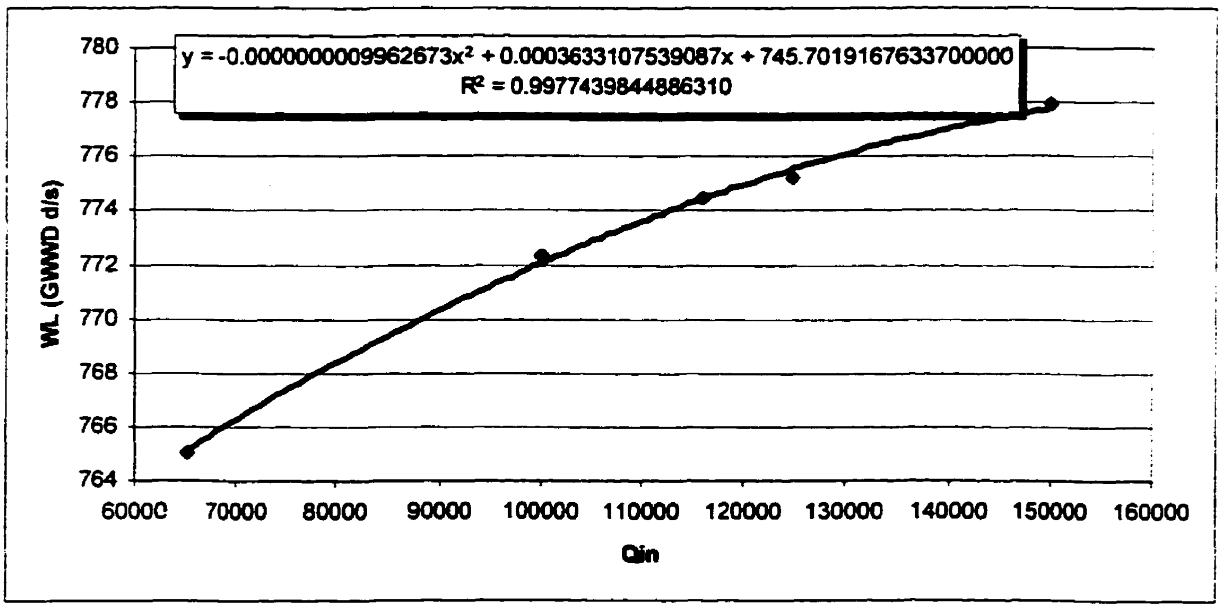


Figure B.8 Relationships for the floodway section from PTH 15 to GWWD (GWWD: Greater Winnipeg Water District bridge)

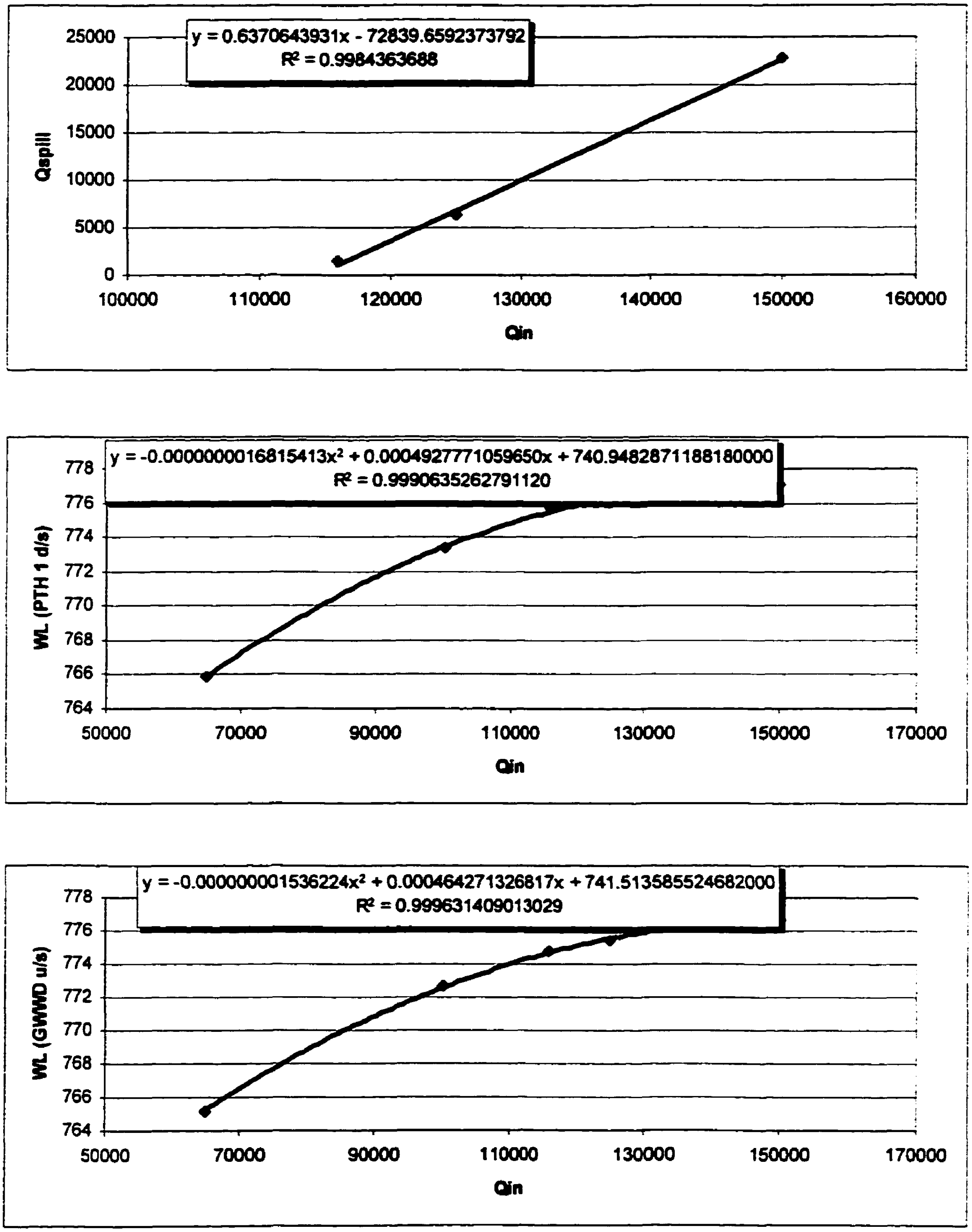
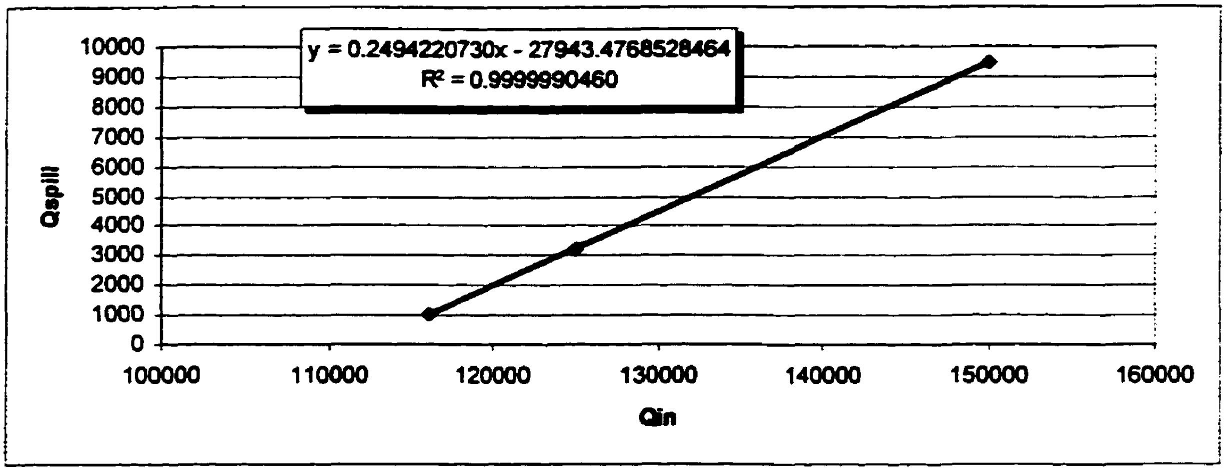
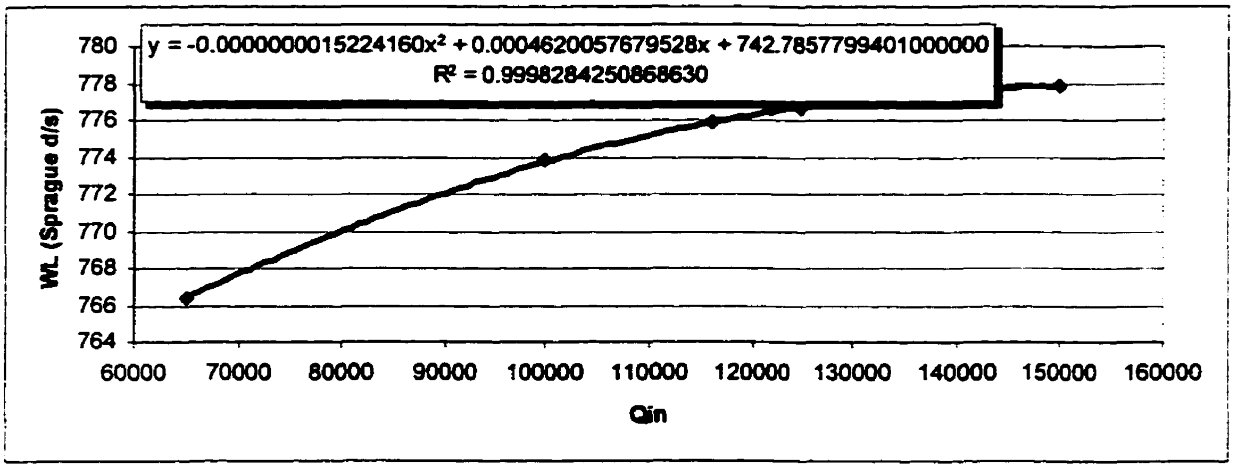


Figure B.9 Relationships for the floodway section GWWD to PTH 1





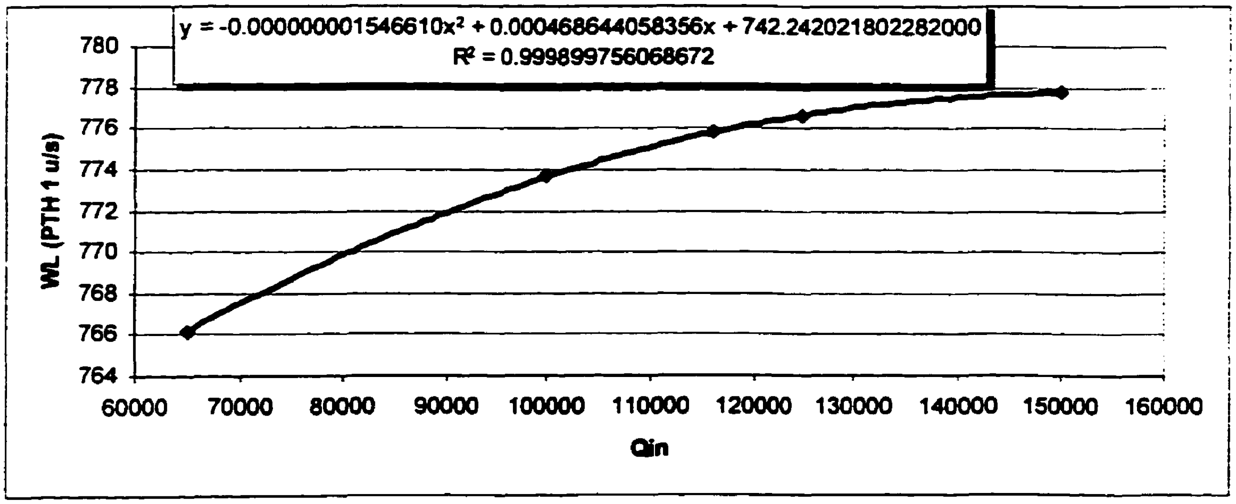
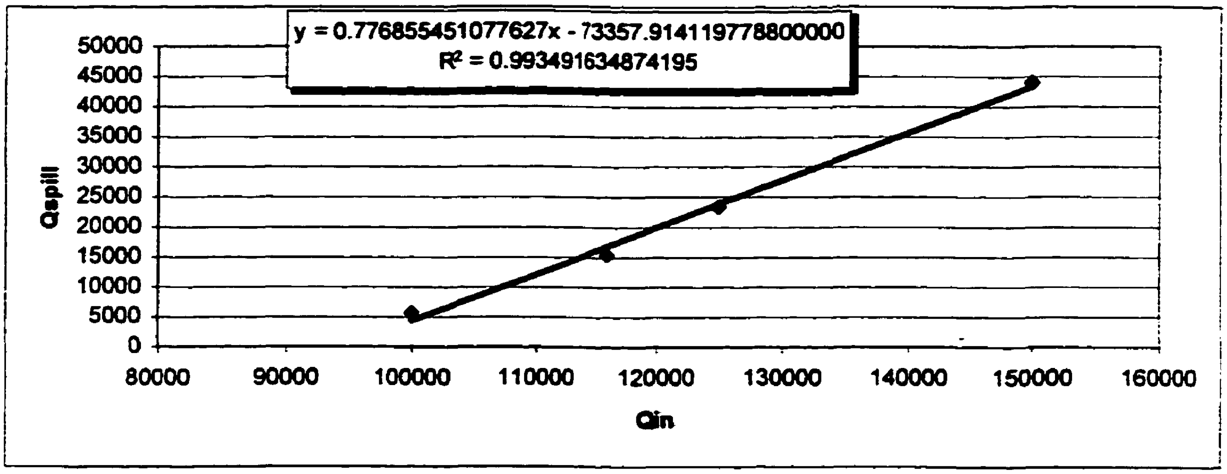
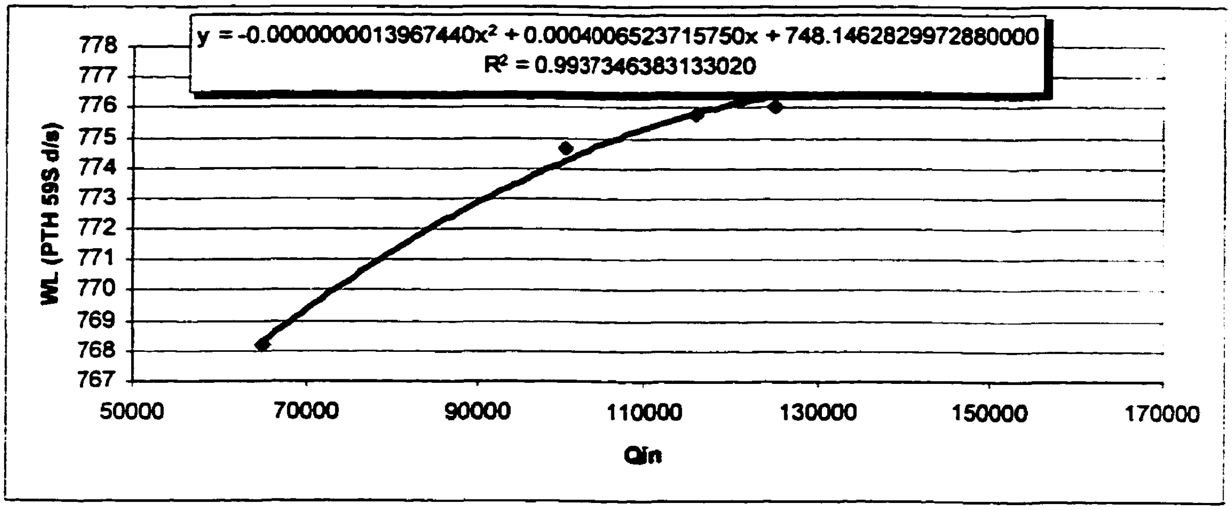


Figure B.10 Relationships for the floodway section PTH 1 to CNR Sprague





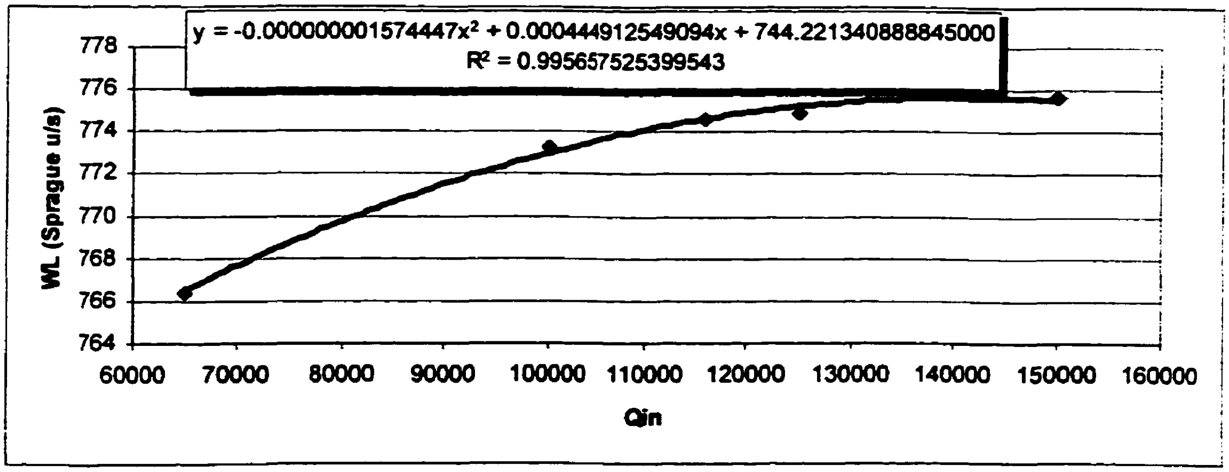
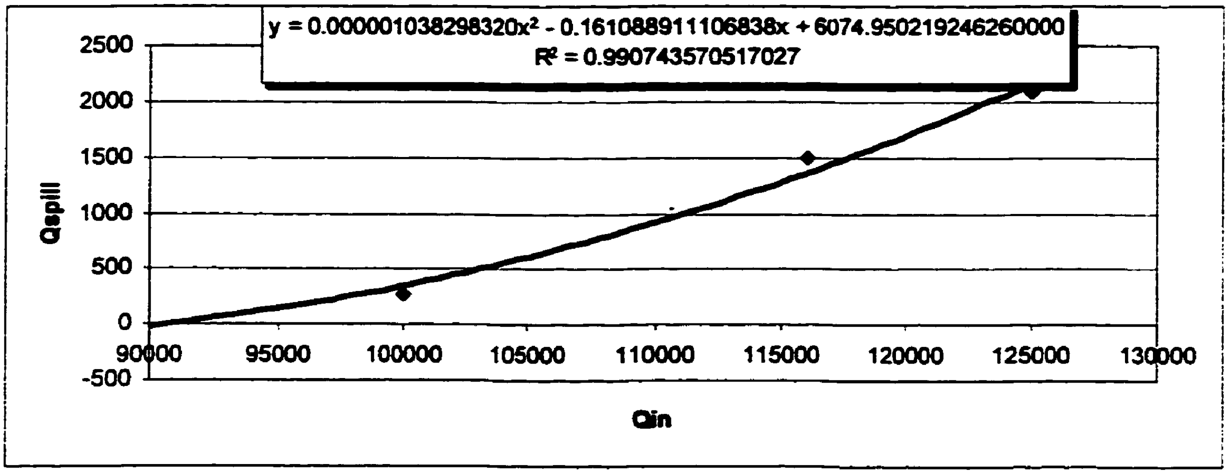
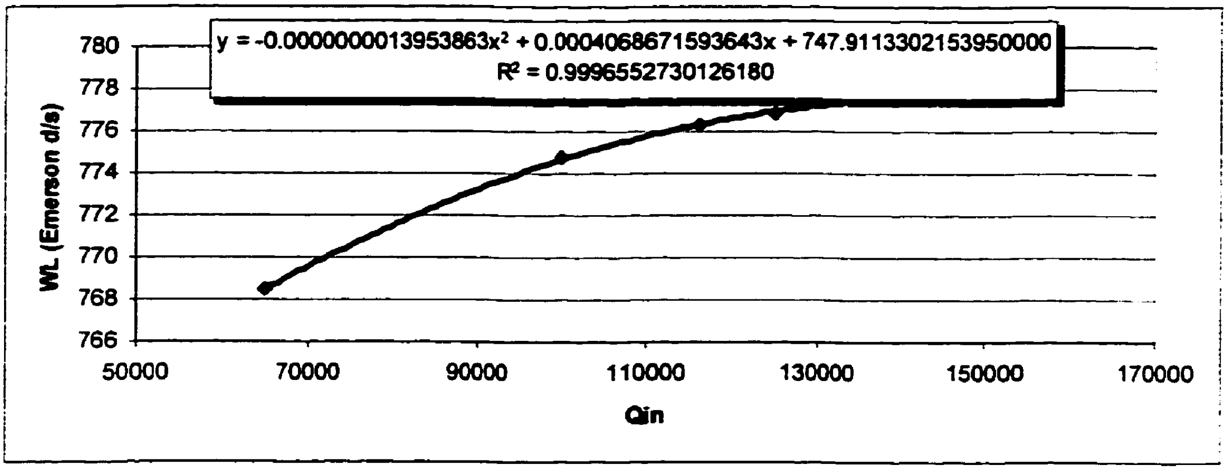


Figure B.11 Relationships for the floodway section CNR Sprague to PTH 59(s)





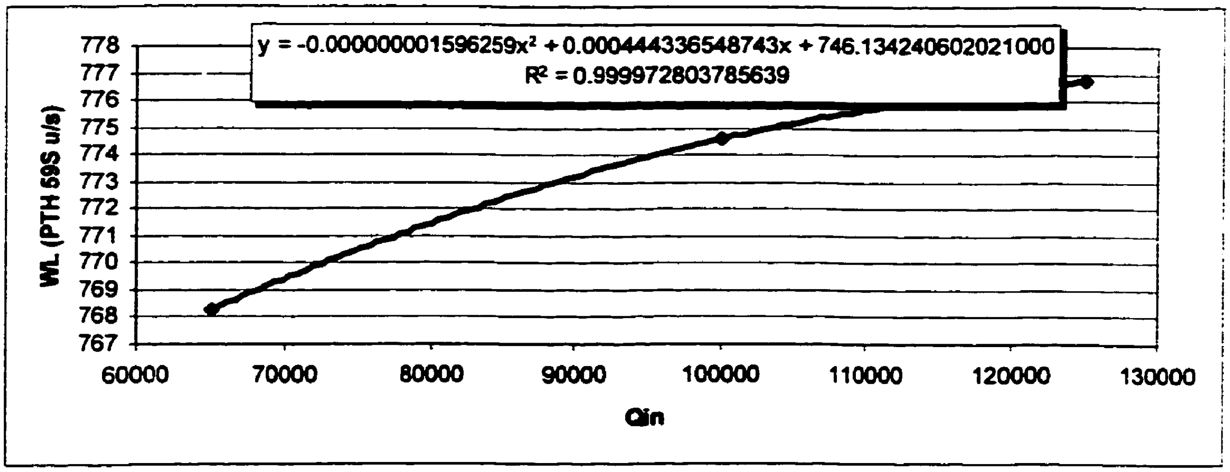
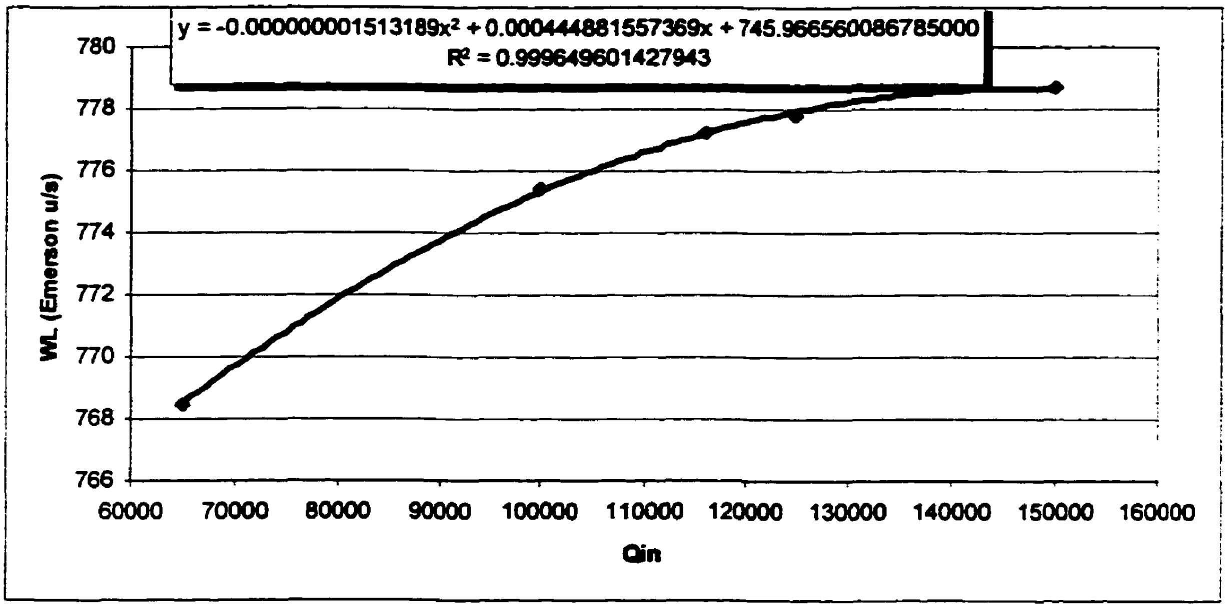


Figure B.12 Relationships for the floodway section PTH 59(s) to CPR Emerson



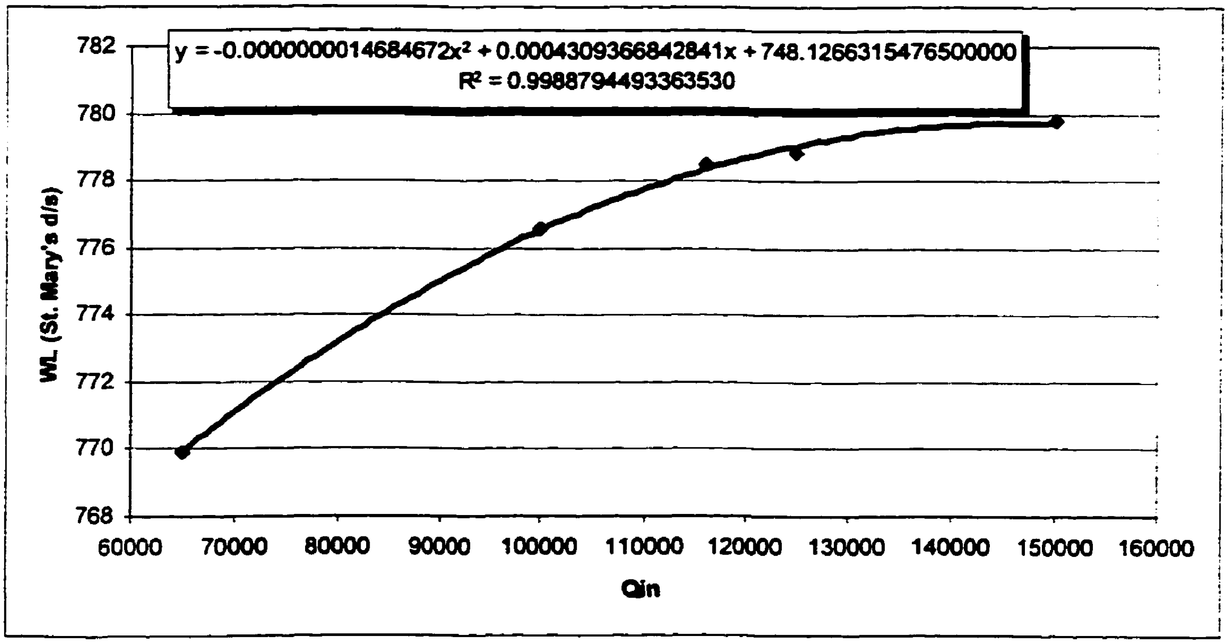


Figure B.13 Relationships for the floodway section CPR Emerson to St. Mary's Rd.

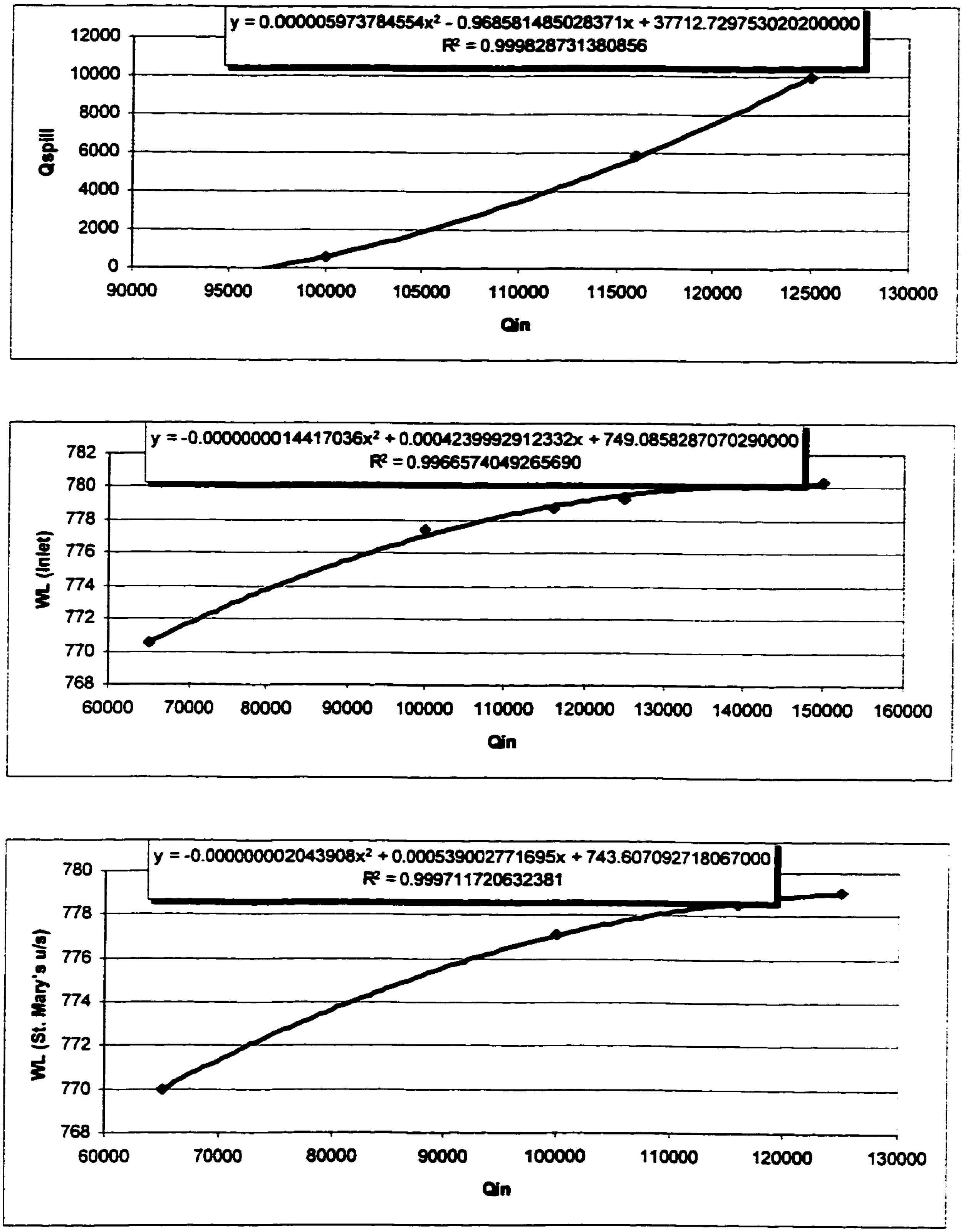


Figure B.14 Relationships for the floodway section St. Mary's Rd. to Inlet