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Numerical Modeling for Floodway Assessment—A Case Study

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ABSTRACT: Vitality of the Neebing-McIntyre Floodway based on a variety of climatic and geometrical boundary conditions has been evaluated. A numerical hydraulic model of the floodway was developed. Sensitivity analysis of the floodway indicates that the lower reaches are particularly receptive to fluctuations of water levels in Lake Superior. Variation in historic water levels (182.41 m and 183.60 m) of Lake Superior may produce a change in water elevation of 0.49 m. The water elevations in floodway show sensitivity to roughness coefficient due to presence of heavy cattails growth in the Diversion Channel.

Assessment confirms that flooding is likely to occur in the commercial district during a regional storm unless banks in this area are raised. Analysis of climate change scenarios indicates that the magnitude and frequency of rainfall events are likely to increase in the study area and water levels in Lake Superior are expected to decrease due to increased evaporation losses. Regular maintenance of the floodway is imperative to keep vegetation and sediment deposition to minimum to ensure operations of the floodway within its design parameters. Additional stresses in the form of increased magnitude and frequency of floods are expected due to regional climate change scenarios which will heighten the need for regular and effective maintenance of the floodway. The results of this paper are applicable to other similar situations where climate change scenarios are expected to occur.

INTRODUCTION

Flooding has been an ever recurring problem in the commercial district (also known as Intercity area) of Thunder Bay due to combined affects of low topography and shared floodplain by the Neebing and McIntyre Rivers. There are two types of flooding events each occurring as the spring peak caused by snowmelt and rainfall or the autumn peak caused by rainstorms. High water levels have been found to cause flooding of basements and streets as well to congest sanitary and storm sewers. The Lakehead Region Conservation Authority (LRCA) has examined several possible flood control schemes and one such proposal was developed in 1955 which recommended that a flood channel be built between the Kaministiquia River and the Neebing River. However, a proposal was put forward in 1970 by MacLaren and Associates that would later be adapted and used. This proposal involved constructing a floodway between the Neebing and McIntyre Rivers which would make it possible to hold an increased flow from both rivers. Construction began in 1979 on the Neebing/McIntyre Floodway which was completed in 1983 at a cost of $15 million (Gigliotti, 1989).

The Neebing/McIntyre Floodway has been designed for the regional storm also referred to as the Timmins storm which occurred in the year 1961. When this storm is transposed and centred over the Neebing and McIntyre watersheds, the storm is considered to produce 149.86 mm of rainfall in a 12 hour period (Proctor and Redfern, 1973). The design conditions used a Manning's roughness coefficient of 0.028 which is typical for straight excavated channels with grass growth. The floodway has been found to maintain a relatively straight channel with a trapezoidal shape and a design slope of 0.05% (Murphy, 1997). For the regional storm, the floodway channel has been designed to carry a flow of 156 m^3/s from the Neebing and McIntyre Rivers. A Diversion Structure located at the junction of the Neebing River and the Diversion Channel (Figure 1) limits the flow in lower sections of the Neebing River to 29 m^3/s. Excess flow runs through the Diversion Channel and into the floodway leading to Lake Superior. Determining factors in the design were the amount of urban disturbance and cost of construction. The channelization of the Neebing and McIntyre Rivers has caused the channel to become longer which lead to a reduction in velocity and thus adversely affecting the rate of sedimentation (Gigliotti, 1989).

Based on a variety of governing variables, this paper objectively reassesses and evaluates the present and future integrity of the Neebing/McIntyre Floodway system. Under different scenarios of rainfall events,

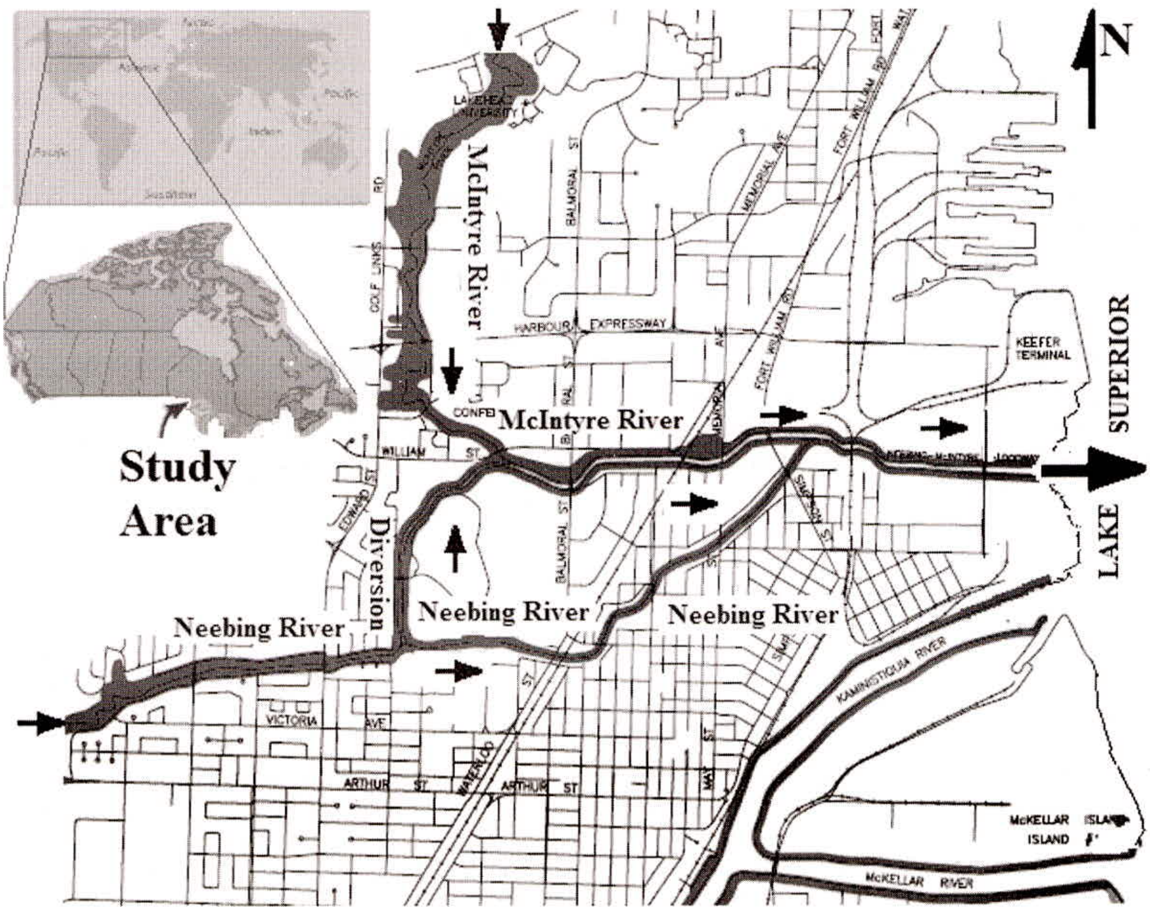

Fig. 1: Location Map of the Neebing/McIntyre Floodway System

climate change, boundary conditions such as change in roughness coefficient, and water level fluctuations in Lake Superior, this paper evaluates the performance of the floodway.

STUDY AREA

The Neebing/McIntyre Floodway system is located in Thunder Bay, Ontario, Canada and carries water from the Neebing and McIntyre rivers which flow through the City of Thunder Bay into Lake Superior (Figure 1). It essentially comprises of five elements namely the Neebing River, Diversion structure, Diversion channel, Floodway channel, and the McIntyre River. The lower portion of the McIntyre River below the confluence with the Diversion channel is also called the Floodway channel. The excess capacity of the Neebing River is diverted by a diversion structure into the Diversion channel which carries the excess water downstream into the Floodway channel (i.e., the lower portion of the McIntyre River). This portion was designed, realigned, and modified to increase its capacity to accommodate the excess flows of the Neebing River. The geometric description and other essential properties of each of the five components pertaining to the overall functionality of the Neebing/McIntyre Floodway system are as follows.

Neebing River

This River upstream of the Diversion Structure can be considered mature and a meandering stream with a general sandy stream bed (Murphy, 1997). The drainage basin area of the Neebing River is 216 km^2 and additional physiographic properties of the watershed are summarized in Table 1. The Neebing River has two major and several minor tributaries. The peak flow rate of the Neebing River for the regional storm has been computed at 156 m^3/s (Murphy, 1997). The alignment meanders considerably and the banks have a dense, healthy vegetation cover of grasses, shrubs and mature trees. There is some erosion occurring in localized areas which produces some sediment and large debris as sections of the bank are undermined and collapse carrying trees with them. Further, upstream of the Thunder Bay Expressway, the river channel is mature and meandering. The soil type for the balance of the watershed is generally fine sands and silts and, therefore, easily erodible which is evidently clear from the active erosion of the riverbanks. The activities of beaver throughout the upper reaches are evident. During major storm events such dams often wash out and the debris is carried downstream. The rush of water from the failed dams has the potential to pick up larger pieces of debris (large root balls and whole trees) and carry them downstream towards the Diversion Structure. Over the longer term, control of the beaver may result in less debris in the system. From the Diversion Structure and downstream back into the Floodway Channel, the Neebing River runs through a residential/commercial area. The banks are quite stable and well vegetated with large trees and brush growth. The Diversion Structure limits the flow through this reach and no significant conditions leading to flooding exists.

Diversion Structure

The capacity of the Neebing River downstream of the Diversion structure is less than 56 m^3/s, therefore the Diversion structure must meet this requirement. The exiting Diversion structure comprising of three orifices (3.1 m × 0.7 m) was designed to meet the necessary design requirements.

Diversion Channel

This is a man-made channel running along the north and west side of Chapple's Golf Course (Figure 2). The channel bottom is grassed and the side slopes are also generally grassed with some hardened sections on the curves. At the upstream end where the Diversion channel originates from the Neebing River, the channel invert is approximately 1 m above the river invert. It is noted that at the time of construction in 1983, this elevation difference was approximately 2 m. Therefore, it is easily discernable that a considerable sediment deposition occurs in this area during high flow periods due primarily to the reduction in velocity as the flow cross section significantly increases in the Diversion channel.

Table 1: Watershed Characteristics of the Neebing and McIntyre Rivers

River Name	*Basin Hydrologic Characteristics*					
	Basin Area (km^2)	*Channel Length (Km)*	*Maximum Drop (m)*	*Average Slope (m/m)*	*Peak Flow Rate (m^3/s)*	
					Historical	*Regional*
Neebing River	215.9	39.1	289	0.0074	74.8	156
McIntyre River	155.3	47.5	320	0.0067	69.2	127

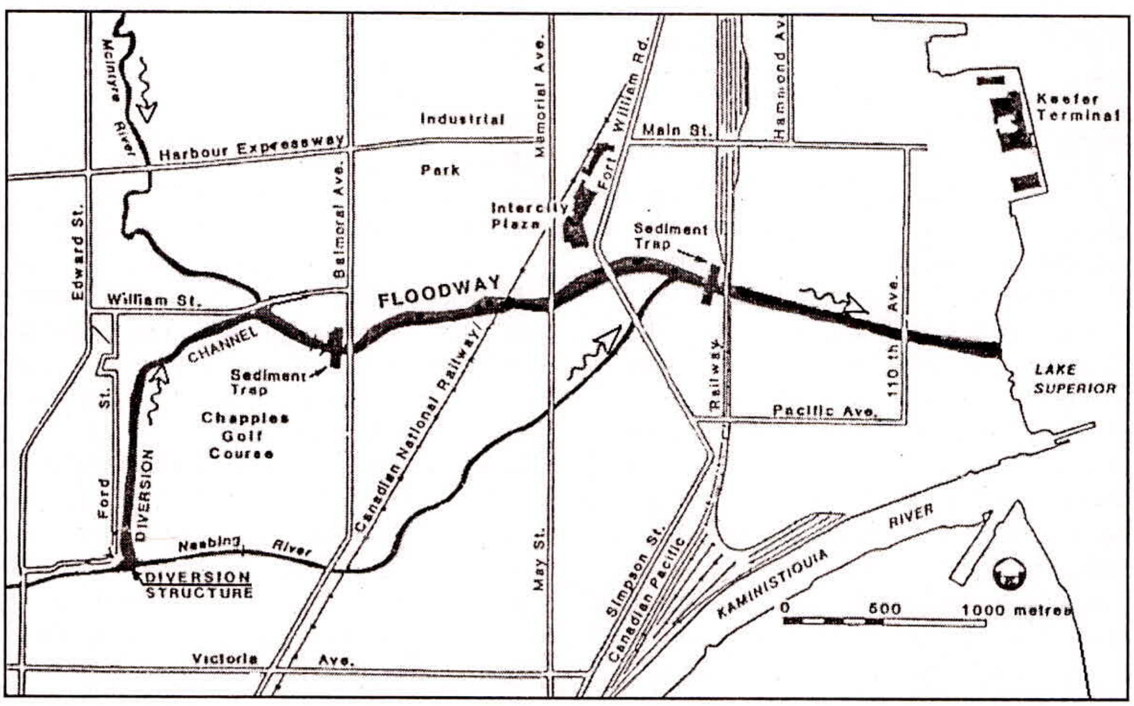

Fig. 2: Detailed Diagram of the Neebing/McIntyre Floodway

The channel banks on the "outside" of the bend at the junction of the Neebing River and Diversion channel are hardened with rip rap and thus no erosion occurs. The only problem noted in this area is that the north side bank of the Diversion channel is approximately 10 m from the old channel bank (i.e., Neebing River) and the headwall for the storm sewer from the north is no longer at the stream bank. A 1 m± deep channel connects the headwall to the bank of the Neebing River. This is not normally a problem but excessive sediment deposition can occur during a major flooding event requiring periodic removal of sediment from this river reach.

The bridge structures similar to the Ford Street Footbridge are not designed for any major lateral loading and because during major rainfall events water level tends to reach the underside of the web of the pre-cast girder for the main span of this bridge which has become a source of concern for potential failure. Since this bridge has existed prior to the construction and it also carries a 100 mm low pressure natural gas attached to the downstream side of the bridge at deck level, this bridge needs some improvements to remain functional.

Floodway Channel

The realigned and upgraded channel from the confluence of the Diversion channel with McIntyre River till it ends into Lake Superior is called the Floodway channel. The design channel capacity (156 m^3/s) is sufficient to carry the flows from the McIntyre River and the Diversion channel.

McIntyre River

This River is probably best characterized as a mountain stream. For most part, the stream bed is comprised of gravel, boulders and cobbles, and the flow velocity is relatively high. In the upper reaches, the river flows through glacial till and rock formations, and bedrock is exposed in many areas. The water is fairly clear indicating a light sediment load. The river runs through a small reservoir on the campus of Lakehead University where much of the sediment load is trapped. The drainage basin area of the McIntyre River is 155 km^2 and additional physiographic properties of the watershed are summarized in Table 1. The McIntyre River has no major tributaries but does have several tributaries shorter than 4 km (2.5 miles) in length. The peak flow rate for the regional storm has been computed at 127 m^3/s (Murphy, 1997). Downstream of this point the river runs through a relatively flat area with stable banks, except on the Confederation College grounds where there is a 500 m long river reach with several sharp bends and steep banks in a sandy soil.

FLOOD FLOW DATA COLLECTION AND ANALYSIS

Environment Canada hydrometric stations on both rivers record historic flood flow information. The flood flow data is available respectively since 1954 and 1972 on the Neebing River (02AB008) and McIntyre River (02AB016). Frequency analysis of instantaneous flows of both rivers indicated that the 100 year flow is approximately 110 m^3/s. Since 1977, several hydrologic models have been used on the Neebing and McIntyre Rivers to estimate the 100 year flow for the design of the Diversion and Floodway channels. The estimated 100 year flow was found to range from 121–156 m^3/s. The use of design flow of 156 m^3/s for the Floodway channel was considered conservative but appropriate.

Table 2: Comparative Summary of Peak flows on Neebing River and McIntyre River

Name of the Study		*July 2, 1997 Storm 50 Year (m^3/s)*		*100 Year Storm (m^3/s)*		*Regional (Timmins) 175 year Storm (m^3/s)*	
		Neebing River	*McIntyre River*	*Neebing River*	*McIntyre River*	*Neebing River*	*McIntyre River*
Original Floodway Design (1977)		–	–	105	86	156	128
Anderson Associates Flood line Study (1985)		–	–	113	124	128	129
Master Drainage Plan (1987)		–	–	86	98	121	141
Engineering Northwest Integrity Study (1998)	AMC-III	–	–	97	92	150	122
	Frequency Analysis	–	–	110	110	–	–
Other Considerations		80	65	–	–	–	–

The flow estimates specifically for the three storms, namely, the July 2, 1997 Storm (50 year), 100 year storm, and regional (Timmins) storm (175 year) are considered as summarized in Table 2. It is apparent from the table that the flow estimates on the Neebing River for the 100 year storm and regional storm respectively vary from 86 to 113 m^3/s and 121 to 156 m^3/s.

Historic water levels in Lake Superior are available since 1907 and were obtained from the Canadian Hydrographic Service. The maximum average daily water level observed in Lake Superior since 1907 is 183.65 m (geodetic), while the average water level is 183.14 m. Instantaneous maximum hourly levels as high as 183.92 m (geodetic) has been observed. In this paper, maximum daily average water levels were used.

HISTORICAL BACKGROUND ON THE USE OF HYDRAULIC MODELS

Past studies related to the design and operation of the Floodway channel include the original floodway design model, a 1987 Master Drainage Plan, and a Floodway Integrity Evaluation Study completed in 1998. All these studies have used the popular water surface computation model called as HEC-2 and its latest version as HEC-RAS (River Analysis System). The HEC-2 computer program developed by the Hydraulic Engineering Centre of the U.S. Army Corps of Engineers is widely accepted and used for computing water surface profiles for one-dimensional steady and gradually varied flows in channels comprising of any cross section such as bridges culverts and weirs. This program is based on the Standard Step Method of water surface computation and utilizes the Manning formula for frictional losses. The HEC-RAS is the latest version with some of the major capabilities including: user interface, hydraulic analysis components, and data storage and management.

ASSESSMENT OF EXISTING FLOODWAY MODELS

The Neebing/McIntyre Floodway system comprising of the Diversion structure, Diversion and Floodway channels was constructed between 1979 and 1983 to provide additional flow capacity in the two river systems and eliminate surface flooding in the Intercity area of the City of Thunder Bay in Ontario, Canada. In 1973, Proctor & Redfern completed a water surface profile model of the floodway system based on the HEC-2 program and design parameters of the floodway. Subsequently in 1985, the model was updated by Anderson Associates for the "Flood Fill Line Mapping Study". The existing floodway model (Proctor & Redfern, 1973) was again updated in 1998 by the Engineering Northwest by using the HEC-RAS for the "Neebing/McIntyre Floodway Integrity Evaluation Study". The input data was reviewed by Engineering Northwest to confirm that it reasonably represented existing flow conditions in both rivers. Changes were made by Engineering Northwest to the parameters in several sub-reaches to reflect the development which has occurred since the model was prepared. The input data was then imported into HEC-RAS and the additional cross sections and floodway bridges were added for the analysis. For example, the geometric verification of cross sections of the Floodway channel have been reported in the 1998 study by Engineering Northwest for assessing whether there were significant deviations from the designed and constructed cross sections of 1983. These sections were found to change somewhat due to natural processes and were more like natural sections than the man made sections (Figure 3). A slight deposition at the corners of the cross section and minor scarping of the bank cannot constitute to influence the flow carrying capacity. The short vertical sections of the banks appeared robust with the exception of a small area on the north bank beside the McIntyre Mall and another area on the north bank between Memorial Avenue and Fort William Road. In addition, the tops of banks were essentially at the designed and constructed elevations with the exception of a short area on the south bank immediately west of the Canadian Pacific Railway (CPR) Bridge.

There was no evidence of significant build-up of sediment around the bridge piers or under the bridges. A slight deposition (150+ mm) was found however in the shadow of the piers on the downstream side. A meandering flow channel has developed in the bottom of the Floodway thus indicating that the river is trying to establish a natural channel within the geometrical boundary of the man-made channel. At the mouth of the Floodway channel, significant erosion continue to occur behind the armour stone protecting the south bank of the floodway at the lakeshore due to considerable wave action.

DEVELOPMENT OF THE 2006–FLOODWAY MODEL

Digital copies of the original floodway design model (Proctor and Redfern, 1973) and its updated version (Engineering Northwest, 1998) were obtained from the LCRA. These versions were critically reviewed to ensure that they were properly representing all the

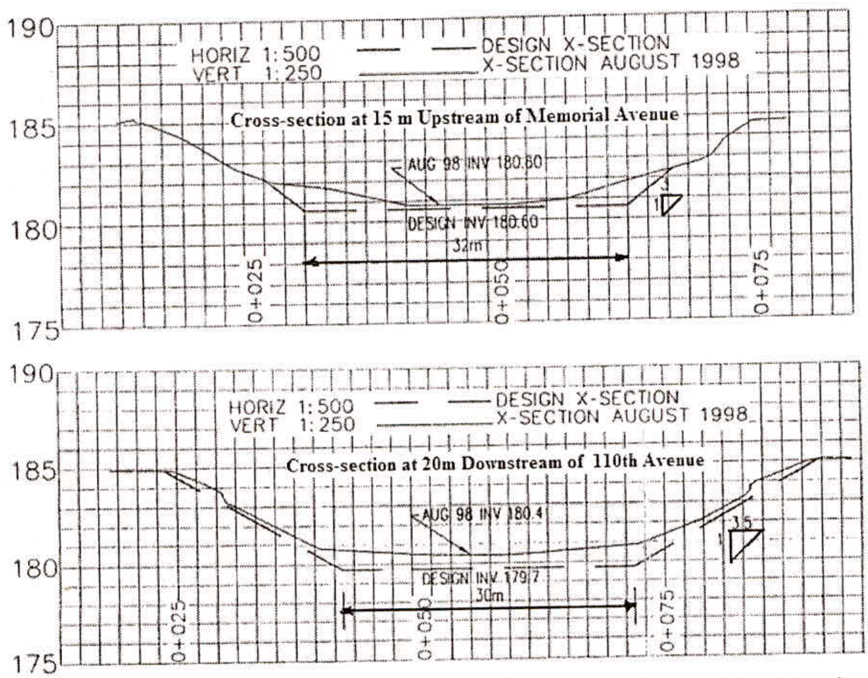

Fig. 3: Comparison of Geometric Properties (Design versus Year 1998) at two locations

components of the Floodway system (Figure 2). Our review indicated that digital copy of the HEC-2 model did not have representation for the turns or any of the bridge structures that are located along the floodway. Suitable modifications where required and as discussed later were accomplished. Once these modifications were completed, the model was imported into the HEC-RAS program.

Significant modifications that were carried out to the existing floodway model were: (1) all turns in the floodway were added to the model to ensure that the model adequately represented the actual floodway system, (2) all bridges and piers were added to the model, (3) surveyed sections used in the 1998 Floodway Integrity Study were also added to the model, and (4) Manning's "n" through the bridges were updated to represent friction offered by the presence of rip-rap along the side slopes under the bridges. In addition, case specific changes were carried out to the model in order to determine its sensitivity to certain parameters such as changes in Manning's 'n' and the effects of fluctuations in water levels of Lake Superior. All such modifications are briefly described below.

Determination of Turns along the Floodway

As indicated earlier that the existing digital model of the floodway required modifications to include the necessary turns at their appropriate locations. The locations of all turns were first identified using the map of the City of Thunder Bay. A site visit was also conducted to physically verify the locations of all turns in the floodway system such that their inclusion in the model accurately represents the field conditions.

Determination of Manning Roughness Coefficient through Bridge Structures

It was observed during site visits of the floodway that 6–12 inches of rip-rap of broken rocks lined the side slopes of the floodway under the bridges. Based on theses observations, it was decided that Manning roughness coefficient through the bridges be modified to improve the accuracy of the model. A review of pertinent literature indicated that a value of 0.035 of the roughness coefficient would be reasonable to represent the existing condition of the rip-rap on the side slopes under the bridges. A composite roughness coefficient value (n_{eq}) ranging from 0.030–0.031 was computed for the channel sections encompassing the bridges. This computation is based on the roughness coefficient values of 0.028 and 0.035 respectively for the bottom of the channel and the side slopes under the bridges. Additional information is provided by McKenna and Tomlinson (2006).

Inclusion of Cross Sections of Bridges and Additional Surveyed Cross Sections

For improved representation of the floodway in the digital model, the surveyed sections completed by Engineering Northwest in 1998 were utilized in developing the 2006–Floodway model. The sediment deposition over the years has transformed the channel cross sections to resemble as natural river cross sections (Figure 3).

Calibration and Validation of the 2006–Floodway Model

Models require calibration and validation before being considered to be representative of the real systems. The 2006–Floodway model developed in this paper was calibrated and validated as described below.

Model Calibration

Previous versions of this model have been calibrated and validated using a variety of flood flows (MacLaren Associates, 1970; and Anderson Associates, 1985). The preliminary test runs were conducted to check the validity of values used for various parameters specifically the values of roughness coefficient (Manning's 'n') used throughout the floodway and especially through the bridges. As described earlier, Manning's 'n' values through the bridges were modified to represent the exiting conditions. In additions, turns were added to represent as closely as possible the flow regime through the floodway. For example, the earliest version of this model used roughness coefficient values of 0.028 for the main channel and 0.03 for the over banks. For the 2006–Floodway model, the roughness coefficient of 0.04 was specified in the Diversion channel to account for the vegetation growth. Once the preliminary test runs provided reasonable flow conditions (i.e., water elevations along the floodway) for the storm events, the 2006–Floodway model was considered calibrated to represent the flow conditions in the floodway.

Model Verification

The 2006–Floodway model was verified by comparing observed data on water surface elevation for the July 2, 1997 storm event. This storm event of 100 mm of rainfall over the 18 hour storm was estimated to be 50 year storm event with peak flow of 80 m^3/s in the Neebing River and 65 m^3/s in the McIntyre River (Table 2). It is noted that this event has also been reported by Engineering Northwest (1998). Similarly, the results thus obtained from the 2006–Floodway model were also compared with those reported by Engineering Northwest (1998) to ensure that there were no obvious discrepancies.

Observed Water Surface Elevations during July 2, 1997 Storm Event: During this storm event, the LRCA reported water surface elevations reaching the underside of the Ford Street Footbridge (Figure 2) which corresponds to an elevation of 186.15 m. Also, water surface elevation of 186.15 m at the upstream face of the Diversion structure was approximated by Engineering Northwest (1998) based on the information contained in a home video and photos taken during the event. Observations made on water levels in the Intercity area during this storm event show that the water surface was at approximately 300 mm below the top of the bank.

It was also reported that during and after this storm event considerable debris was blocking the orifice of the Diversion structure and a silt plume of up to 1 m thick was present in front of it. This silt plume is a product of the high sediment load which the Neebing River normally carries during heavy storm events. Due to the increase in cross sectional area upstream of the Diversion Structure, the flow velocity tends to decrease causing sediment to be deposited. This deposited sediment forms a plume which causes the water level to increase upstream. In order to validate the 2006–Floodway model, it was run with a silt plume present and the Diversion structure being partially blocked. Also, the Lake Superior water level in the model was set to 183.40 m (geodetic) which was the recorded water level on July 2, 1997. The results comparative to the observed water level of the 2006–Floodway model and past floodway models are summarized for comparison in Table 3.

Table 3: Comparison of Past and Present Floodway Models

Location	*July 2, 1997 Storm Event Water Levels in the Floodway Based on the Water Level in Lake Superior at 183.40 m*		
	Observed Water Levels	*1998–Floodway Model (Engineering Northwest Study)*	*2006–Floodway Model (Present Study)*
Diversion Structure	186.15	186.05	186.10
Ford St. Footbridge	186.15	186.05	186.13
Intercity Area (Fort William to Memorial Avenue)	184.20	184.30	184.30

Even though there are significant developmental differences between the 2006–Floodway model with bridges and turns etc. being added and the floodway model (Engineering Northwest, 1998), the variations in water levels at selected stations are minimal (Table 3). It is apparent from this table that although the 2006–Floodway model does compute slightly higher water surface elevations but these levels are within the statistical margin of error. Similar comparative water surface elevations for these two models are also apparent from Table 4.

Effect of Extension of Floodway into Lake Superior

The 2006–Floodway model was modified to include cross sections beyond the shore-line into Lake Superior. This modification was carried out to include the observations that under natural flow conditions the floodway water plume jetted out into Lake Superior. In an attempt to model this condition, additional cross sections were inserted into the model which extended out into the lake. These cross sections were constructed based on a Thunder Bay Harbour depth soundings map and information provided by the LRCA staff. The cross sections thus constructed adequately accounted for the lake bottom topography and dredging that was completed during the construction of the floodway. Table 5 presents a summary of the results of such a comparison (i.e., when additional cross sections into the lake are applied, and when all such cross sections are removed from the model) of water surface elevations at various stations along the floodway. It is noted that providing additional cross sections into Lake Superior did not significantly vary the water surface elevations in the floodway.

APPLICATION OF 2006–FLOODWAY MODEL

Once the 2006–Floodway model was found through the validation process to represent reasonably the response of the Neebing-McIntyre Floodway system, the validated model was used with the additional cross sections into Lake Superior to better represent the actual floodway system. In this model, additional cross sections upstream of the Diversion structure were also included to account for the formation of a silt plume since its development represents an expected occurrence during large flow events. In the following, the 2006–Floodway model is applied using a range of scenarios where different input variables are used to assess the response of the Neebing-McIntyre Floodway system.

Table 4: Comparison of Water Surface Elevations of the 2006 & 1998–Floodway Models

Floodway Model	*Water Surface Elevation (m) for the Regional Storm ($Q = 284\ m^3/s$) with Lake Level at the mean water surface 183.11 m*							
	20 m U/S of 110th Avenue	*CPR Weir*	*33 m D/S Fort William Road*	*15 m U/S of Memorial*	*15 m D/S of Balmoral*	*Entering Diversion Channel*	*Diversion Structure*	*Foot Bridge*
	Station 10.5	*Station 19*	*Station 23.6*	*Station 27*	*Station 36.7*	*Station 41.6*	*Station 57.3*	*Station 58.4*
1998	183.52	184.28	184.53	184.74	185.24	185.53	186.68	186.73
2006	183.53	184.36	184.67	184.82	185.21	185.50	186.86	186.89

Table 5: Comparison of 2006 Model with and Lake Superior Cross Sections

Model with Cross Sections	*Water Surface Elevation (m) for the Regional Storm ($Q = 284\ m^3/s$) With Lake Level at the mean water surface 183.11 m*							
	20 m U/S of 110th Avenue	*CPR Weir*	*33 m D/S Fort William Road*	*15 m U/S of Memorial*	*15 m D/S of Balmoral*	*Entering Diversion Channel*	*Diversion Structure*	*Foot Bridge*
	Station 10.5	*Station 19*	*Station 23.6*	*Station 27*	*Station 36.7*	*Station 41.6*	*Station 57.3*	*Station 58.4*
Extending into Lake Superior	183.53	184.36	184.67	184.82	185.21	185.50	186.86	186.89
Ending at Lake Superior	183.52	184.36	184.66	184.81	185.21	185.50	186.86	186.89

Effect of Lake Superior Water Levels on the Floodway

Water levels of Lake Superior between the years 1918 and 2004 were obtained from U.S Army Corps of Engineers. The highest, lowest, mean, and median lake levels were used for assessing the response of the floodway system. Water level records were according to the International Great Lakes Datum (IGLD) which is 0.31 m higher then geodetic elevations and therefore 0.31 m was subtracted from the water surface elevations to convert them into geodetic levels. The water levels thus obtained for the median, mean, high, and low were respectively 183.12, 183.11, 183.60, and 182.41 m (geodetic).

Based on variations of lake level in the 2006–Floodway model, it was observed that Lake Superior water levels have the greatest effect on the water surface elevations in the lower reaches of the floodway system. On the other hand, lake levels have little effect on water surface elevations in the Diversion channel and no effect on water levels at the Diversion structure. Varying the lake level between high (183.60 m) and low (182.41 m) exhibits that Lake Superior could still change the water surface elevation in the floodway by a minimum of 1cm up to Station #54 (near Chapples Bridge within the Diversion channel). It is apparent from Table 6 that the regional storm caused a maximum water surface change of 0.49 m for the variation of lake level between the mean (183.11 m) and high (183.60 m).

Effect of Various Values of Roughness Coefficient on the Floodway

Previous versions of the floodway models used Manning's roughness coefficient of 0.03 for the right and left over banks, and 0.028 for the centre of the channel. The 2006–Floodway model was run using a range of Manning's 'n' values, for example, the roughness coefficient ranging from 0.022 to 0.034 was applied to the main channel to determine how significantly the water surface elevations are affected. As expected and also apparent from the results exhibited in Figure 4 that change in the roughness coefficient had the greatest affect on the water surface elevations during the larger storm events. Water surface elevations for the two different stations along the floodway are shown for different values of 'n'. The Lake Superior water level in the 2006–Floodway model was set at its historic high level (183.60 m geodetic). Similar results for additional stations were also observed along the floodway.

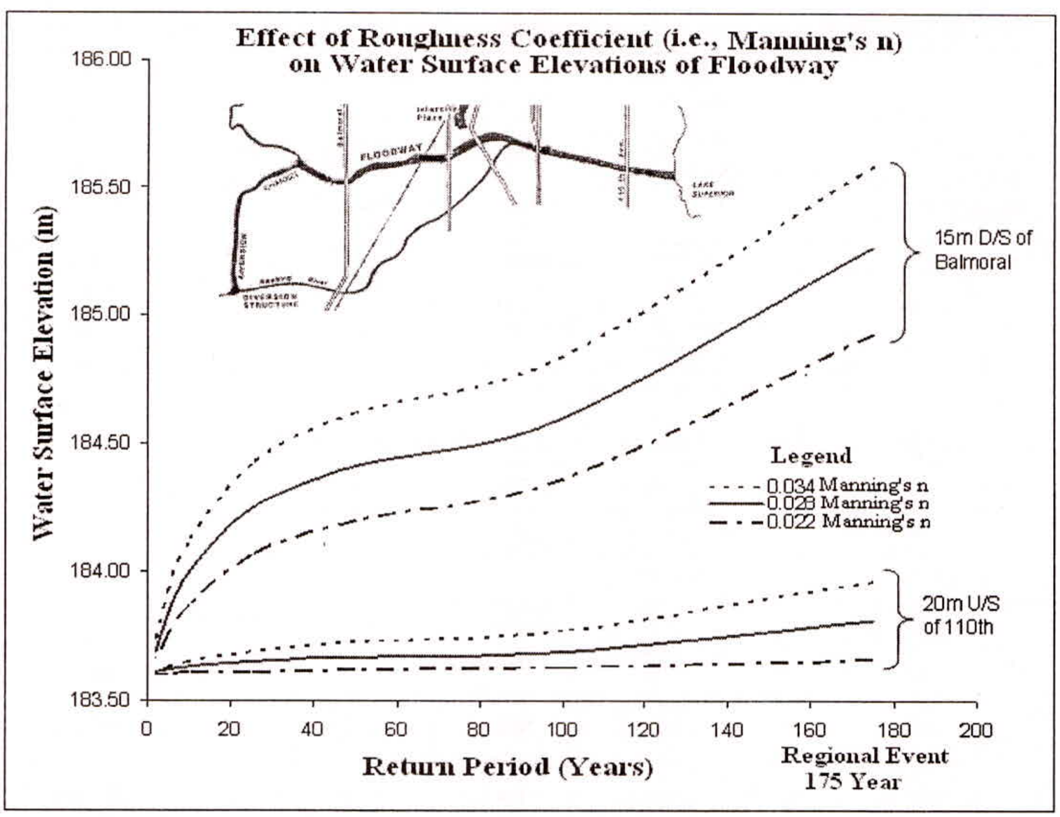

Fig. 4: Floodway Sensitivity to Manning's 'n' Coefficient

Table 6: Variation in Water Surface Elevation for Historic Lake Levels

Historic Lake Superior Water Surface Levels	*Water Surface Elevation (m) for the Regional Storm (Q = 284 m^3/s)*							
	20 m U/S of 110th Avenue	*CPR Weir*	*33 m D/S Fort William Road*	*15 m U/S of Memorial*	*15 m D/S of Balmoral*	*Entering Diversion Channel*	*Diversion Structure*	*Foot Bridge*
	Station 10.5	*Station 19*	*Station 23.6*	*Station 27*	*Station 36.7*	*Station 41.6*	*Station 57.3*	*Station 58.4*
High Level (183.60 m)	183.81	184.48	184.75	184.97	185.27	185.55	186.86	186.90
Low Level (182.41 m)	183.43	184.33	184.64	184.80	185.19	185.49	186.86	186.89
Median Level (183.12 m)	183.53	184.36	184.67	184.82	185.21	185.50	186.86	186.89
Mean Level (183.11 m)	183.52	184.36	184.66	184.82	185.21	185.50	186.86	186.89

Table 7: Water Surface Elevations for various Combinations of the Roughness Coefficient of the Diversion Channel

Storm Event Return Period	*Water Surface Elevation (m) for the Regional Storm (Q = 284 m^3/s) with Lake Level at the high water surface 183.60 m*				
	Entering Diversion Channel	*Diversion Channel*	*Diversion Channel*	*Diversion Structure*	*Foot Bridge*
	Station 41.6	*Station 44*	*Station 54*	*Station 57.3*	*Station 58.4*
	Roughness Coefficient of 0.028 for the centre channel and 0.03 applied to the right and left banks of the Diversion Channel				
Regional	185.55	185.76	186.6	186.86	186.90
100 year	184.83	185.12	186.03	186.29	186.31
50 year	184.61	184.95	185.85	186.11	186.13
25 year	184.42	184.79	185.68	185.93	185.95
10 year	184.12	184.54	185.38	185.62	185.63
5 year	183.93	184.38	185.15	185.40	185.40
2 year	183.73	184.11	184.8	185.06	185.06
	Roughness Coefficient of 0.040 for the centre channel and 0.040 applied to the right and left banks of the Diversion Channel				
Regional	185.55	185.94	187.02	187.27	187.28
100 year	184.83	185.32	186.37	186.62	186.63
50 year	184.61	185.14	186.17	186.41	186.42
25 year	184.42	184.98	185.97	186.21	186.21
10 year	184.12	184.71	185.62	185.85	185.85
5 year	183.93	184.52	185.36	185.58	185.58
2 year	183.73	184.22	184.95	185.17	185.18

Effect of Various Values of Roughness Coefficient on the Diversion Channel

Based on an observation made during the site visit of the Neebing/McIntyre Floodway system that cattails have extensive growth in the Diversion Channel, it was decided to assess the effect of cattails growth on water surface elevations by increasing the value of roughness coefficient in the Diversion channel. An appropriate value of the roughness coefficient based on extensive literature search identified the range of 0.030 to 0.045 to be most appropriate for representing cattail growth in the Diversion channel. Thus, a value of 0.040 was considered to represents the tall growth of cattails in the Diversion channel. Additional detail is provided by McKenna and Tomlinson (2006) with regard to the selection of values of roughness coefficient and photos of the state of vegetation growth in the Diversion channel. A summary of results of two different roughness coefficients applied to the Diversion channel is given in Table 7. It is apparent from the table that water surface elevations within the Diversion channel were raised on the average by 0.23 m while the water levels in the Neebing above the Diversion structure rose on the average by 0.35 m.

RESULTS AND DISCUSSION

Once the modified and updated 2006–Floodway model was calibrated and validated using past events, it was used to assess the integrity of the Floodway and Diversion channels. For this purpose numerous flooding scenarios as described earlier were conducted for the regional (Timmins) storm event with an estimated return period of 175 years. In the following specific areas of concern with respect to flooding situation are presented.

Potential Flooding Areas along the Floodway

The Intercity area and the Diversion structure carry a higher risk of flooding in the Neebing/McIntyre Floodway. The Intercity area is low lying causing a higher risk of flooding and the Diversion structure carries potential factors (silt plume formation, potential orifice blockage from debris) which could cause overtopping. The 2006–Floodway model was run with several different event values to determine flooding potential. It was found that the regional storm combined with a high lake level produced flooding in the Intercity area and downstream of Balmoral, and the 100 year event produced bank high water levels at stations 34 and 34.1 (approximately 150 m downstream of Balmoral).

Potential Flooding in the Intercity Area

Flooding could be a problem in the Intercity area between Memorial Avenue and Fort William Road. Also, in the area downstream of Balmoral certain cross sections, for example, stations # 30.2, 34.1, 36.8 show some flooding potentials. The 2006–Floodway model verified flooding for both the mean and high lake levels in this area for the regional storm. This portion of the floodway is both sensitive to changes in Lake Superior water level and the Manning's 'n' coefficient in the Diversion channel. For studying flooding potentials in an area, the option for extending the elevations on the left and right banks vertically up was invoked.

Potential for Overtopping of the Diversion Structure

The top of the Diversion structure is at an elevation of 186.95 m. Factors which affect the water surface elevation at the Diversion structure include: Blockage of Diversion structure from debris, silt plume formation at the entrance of the Diversion channel, and roughness coefficient in Diversion channel (cattail growth). Cattail growth in the Diversion channel combined with a silt plume formed in front of the Diversion channel and partial blockage of the Diversion structure was found to create a water surface elevation of 187.42 m at the Diversion structure for the regional storm with an estimated return period of 175 years. A water surface elevation of 187.42 m is 0.47 m higher than the top of the Diversion structure (Figure 5). Water surface elevations for the 2006–Floodway model were computed using high lake level (183.60 m), with an increased Manning 'n' value of 0.04 used for cattail growth in the Diversion channel, the silt plume in place, and the Diversion structure 50% blocked with debris. Water surface elevations for the Diversion structure were computed based on the original design model with no limiting factors being applied.

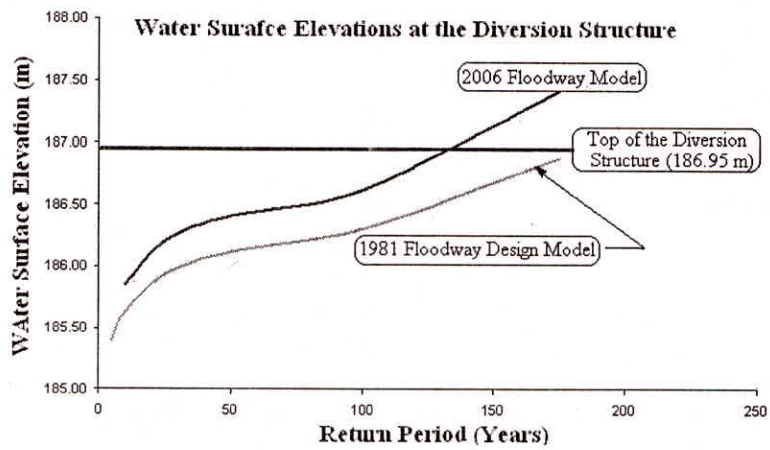

Fig. 5: Water Surface Elevation at Diversion Structure as computed by the Original Design Floodway Model, and the 2006 Floodway Model

Overall Findings of 2006–Floodway Model

In general, the 2006–Floodway model predicted slightly higher water surface elevations compared to the earlier versions of the floodway models. Water surface elevations in the floodway system for a variety of storm events with Lake Superior's water level being set at historic high (183.60 m) are exhibited in Figure 6.

ASSESSMENT OF EFFECTS OF CLIMATE CHANGE ON THE FLOODWAY

Boundary conditions, (lake levels, event flows) are determined based on hypothetical situations which could occur due to global warming. Background information regarding climate change for the Thunder Bay region, and information on how the boundary conditions were determined are summarized by McKenna and Tomlinson (2006).

Changes in Precipitation Events

Many changes in precipitation events are expected over the next century due to global warming. An increase in the magnitude and frequency of large precipitation events is expected for the Thunder Bay region. Also, it is expected that the area will experience an increase in total seasonal precipitation for all but summer months. The potential percent increases in the magnitude of rainfall events are shown in Table 8. Storm magnitudes over the Neebing and McIntyre watersheds could increase by up to 20% by Year 2090. This corresponds to possible flow increases of up to 15%.

The Figure 7 shows the potential changes in water surface elevation at the Station 15 m upstream of Memorial Avenue based on flows for the Years 2050 and 2090. The change in flow caused a fairly uniform change in water surface elevation throughout the floodway except at stations located closer to Lake Superior. An average increase in water surface elevation of 0.13 m is apparent when current flows and Year 2050 flows were compared. An average increase of 0.28 m is expected when current flows and Year 2090 flows are compared. These future water surface elevations were computed solely based on a change in flow and do not consider changes that will inevitably occur in the watershed over the course of time and sediment accumulation that will occur throughout the floodway. Changes to the watershed and the floodway itself will further exacerbate the flooding situation should remedial actions are not undertaken.

Table 8: Potential Percentage Increases in the Magnitude of Precipitation Events

Return Period (years)	*Percentage Increase in Precipitation Event Magnitude*	
	Year 2050	*Year 2090*
100	7.4	16.0
50	9.1	17.8
25	6.1	18.2
10	8.6	15.5
5	3.5	12.5
2	5.5	11.1

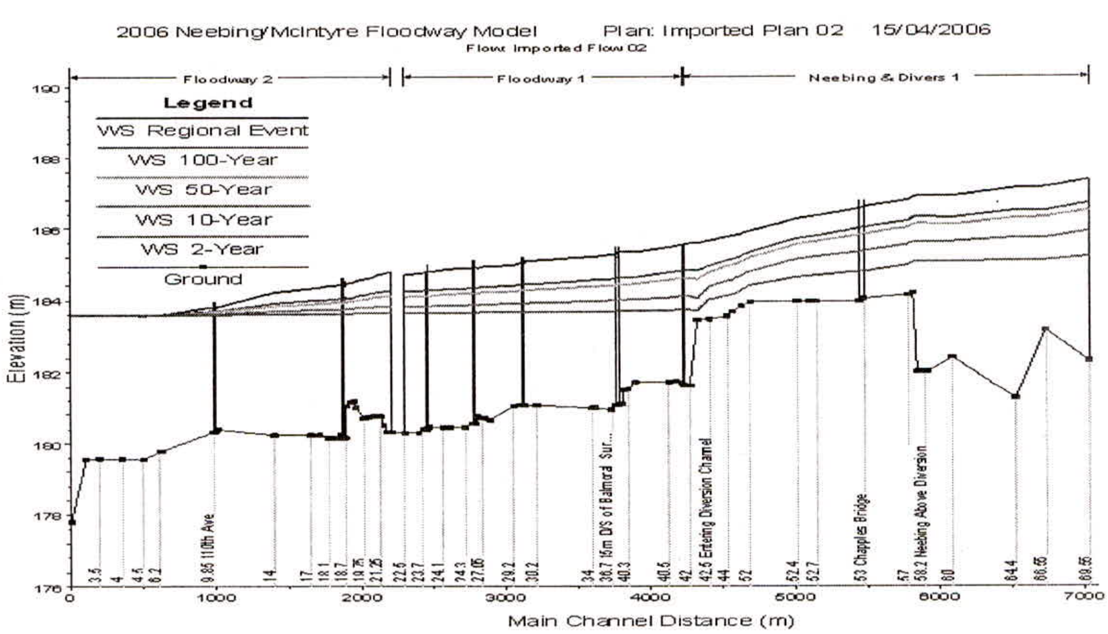

Fig. 6: Water Surface Elevations in Floodway for Different Storm Events

Increase in the Frequency of Large Storm Events

An increase in the frequency of large storm events is expected for the Great Lakes Region and the rest of Canada. Based on the frequency analysis of extreme events over the past 100 years in the Great Lakes Region, it has been shown that the frequency of such events has been higher relative to the long-term average during the past 3–5 decades (Klaassen, 2006). This trend is expected to continue over the next 100 years, possibly resulting in a 40 year event becoming just a 10 year event. This trend will cause an increased burden on the floodway in many ways. Past studies conducted on the floodway have determined that sediment traps should be dredged every 4 years. As the intensity of events increases, the dredging period will decrease as sediment traps will fill up at an increased rate.

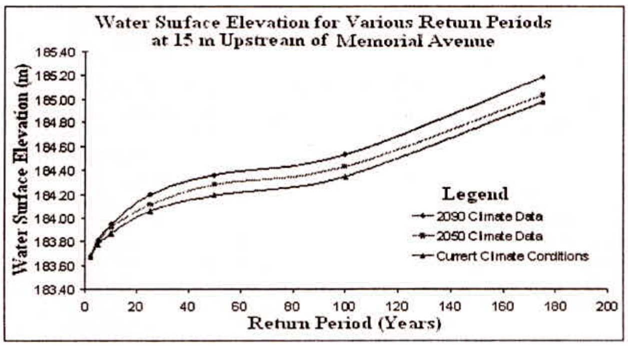

Fig. 7: Change in Water Surface Elevation due to a Possible Increase in Flow

Increase in Total Seasonal Precipitation

The precipitation in general is expected to increase by an average of 10% by Year 2025–2034 for all but summer months. This will have an affect on the antecedent conditions of the watersheds and the base flows of the Neebing and McIntyre Rivers. The 1998 Neebing/McIntyre Integrity Evaluation Study also indicated that for the 100 year storm, the peak flow increased by 30% as a result of wet antecedent conditions compared to average antecedent conditions.

Decreases in Lake Superior Water Level

Climate models have continually predicted water levels in Lake Superior and the other lakes to steadily decline over the next century. A drop in water level of 0.22 m could be expected by Year 2030. A drop of 0.29 m was interpolated for the Year 2050, and a drop of 0.42 m is expected by the Year 2090. The 2006-Floodway model was run with the lower predicted lake levels and the higher predicted flows. It was found that floodway stations near Lake Superior sustained a drop in water surface elevation even though the flows were increased to account for climate change. This is due to the fact that the water level in Lake Superior greatly affects the water surface elevation in the lower portion of the floodway. This trend was found to quickly diminish at stations further up the floodway. It was noted near Station # 18 (next to the CPR Bridge) that water surface elevations began to return to their higher than normal levels due to increased flows. The water surface elevations are exhibited in Figure 8 for the Station 20 m upstream of 110th Avenue and in Figure 9 for the Station 15 m upstream of Memorial Avenue. Potential decreases in the mean Lake Superior water level for the Years 2050 and 2090 were applied to the floodway in conjunction with the possible increases in flow volumes for the same years. Decreases in Lake Superior water level are subtracted from the mean water level. Current mean Lake Level is at 183.12 m (geodetic). It is noted that at Station 20 m upstream of 110th Avenue which is nearest to Lake Superior, the current climate conditions create a higher water surface elevation than the future climate conditions. However, as one moves upstream the floodway to Station 15 m upstream of Memorial Avenue (Figure 9), it can be seen that future climate conditions produce a higher water surface elevation then current climate conditions.

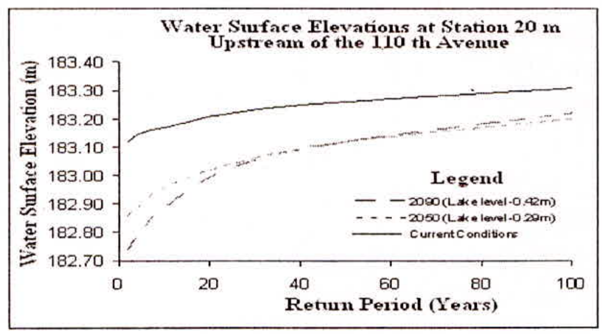

Fig. 8: Water Surface Elevations at Station 20 m Upstream of 110thAvenue

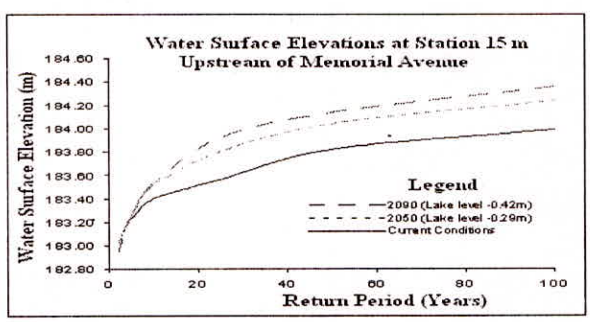

Fig. 9: Water Surface Elevations at Station 15 m Upstream of Memorial Avenue

CONCLUSIONS AND RECOMMENDATIONS

It was found through modeling of the Neebing/ McIntyre Floodway that additional cross sections going out into Lake Superior have no effect on the water surface elevation in the floodway. These results were obtained based on the 2006–Floodway model. The version of this model with sections leading out into Lake Superior was chosen as it more closely represented the local topography.

The sensitivity of the floodway water surface elevations due to changing Lake Superior water levels was demonstrated to apply only to the lower reaches of the floodway. As Lake Superior water levels were run through the 2006–Floodway model using historic Lake high and lows, impacts were greater in lower reaches. These changes have an effect up until the Diversion Structure where there is no change in water surface elevation at all.

In general, the floodway capacity was found to be adequately suited for an event up to the 100 year return period. When the regional storm event was simulated using the 2006–Floodway model, it was found that overtopping of the banks occurred in the Intercity area. Flooding was also a concern at the Diversion structure as overtopping occurred.

Climate changes which may occur in the Thunder Bay region may place increased stress on the floodway. Increased flows are expected, which will raise water levels in the floodway and in the Neebing and McIntyre Rivers. Decreased water levels in Lake Superior will counteract some of these effects arising from increased flows in the lower reach of the floodway. Some of the salient recommendations that evolved from the analysis are as follows:

- Diversion structure should be regularly maintained by removing silt plum and debris blocking the orifices especially after major storm events.
- The Diversion channel should be periodically maintained as the growth of cattails and other vegetation can have an impact on water levels.
- Bank levels through the Intercity area should be raised to deal with the threat of flooding from the regional storm event.
- The future designs of floodway characteristics should take into account the impacts of future climate changes such as a decreasing lake level and an increase in storm intensities.

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