NATIONAL RESEARCH COUNCIL CANADA
OCEAN, COASTAL AND RIVER ENGINEERING

Simulation of Flood Scenarios on the Lower Pembina River Flood Plains with the Telemac2D Hydrodynamic Model Phase 3

Type of Report - Unclassified

OCRE-CTR-2012-22

T. Faure

R. W. Jenkinson

National Research Council
Ottawa, Ontario
K1A 0R6

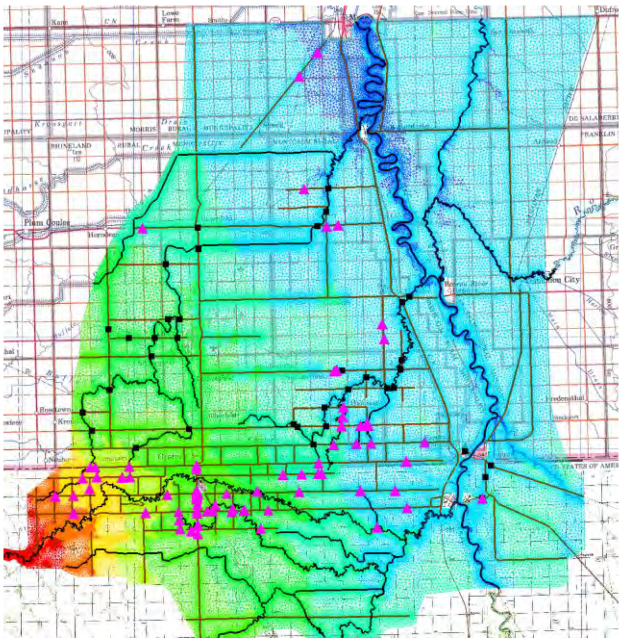
October 2012

Prepared for:
International Joint Commission
234 Laurier Avenue West, 22nd Floor
Ottawa, ON
K1P 6K6

National Research Council Canada | Conseil national de recherches Canada

Ocean, Coastal and River Engineering | Génie océanique, côtier et fluvial

Simulation of Flood Scenarios on the Lower Pembina River Flood Plains with the Telemac2D Hydrodynamic Model – Phase 3



Type of Report - Unclassified
OCRE-CTR-2012-22

T. Faure
R. W. Jenkinson
National Research Council
Ottawa, Ontario
K1A 0R6

October 2012

Prepared for:
International Joint Commission
234 Laurier Avenue West, 22nd Floor
Ottawa, ON
K1P 6K6

Table of Content

List of Tables

List of Figures

List of Plates

Simulation of Flood Scenarios on the Lower Pembina River Flood Plains with the Telemac2D Hydrodynamic Model PHASE 3

1. Introduction

In phase one of the Pembina modelling with the Telemac 2D software, a first 2 dimensional model was prepared to simulate the flooding along the lower Pembina River downstream of Walhalla, ND. This model did not extend far enough North and South of the Pembina River to be able to properly simulate scenarios where the elevated roads and the infrastructure could be adjusted. [Ref 1]

In phase two of the project, the model was extended to cover the Aux Marais River and a large portion of Buffalo Creek, and Louden and Rosebud Coulees. Several scenarios were simulated where the road network was altered, and where proposed diversions and a floodway had been simulated to redirect some flood waters from the Pembina River to the adjacent coulees and rivers. [Ref 2].

The size of the second phase model was much larger than that of the first phase and represented accurately the lower Pembina River and its banks. It was successful in simulating flood propagation over the road network, but it described the other flow passages (coulees) with only a coarser cross section. This was designed to minimize the size of the model and the time it took to simulate a full month flood hydrograph. During some scenarios, extra water arriving from artificial diversions or from the removal of roads was added to the existing rivers and coulees, and the model was not able to reproduce with accuracy the flooding along their banks because of their coarse channel description.

Also, all scenario simulations were performed with the same 2006 spring flood on Red River and Pembina River, which was a relatively large event that flooded the Pembina flood plain in many of the cases, providing only small differences between scenarios.

In this phase 3 of the Pembina flood project, many of the small rivers and coulees have been described with a better description of the cross sections geometry, therefore providing a better representation of the conveyance of each channel. As an example, much of the Aux Marais River was described with a 3 to 6 m grid instead of the original 30 m. This fine description of the rivers has increased significantly the size of the model, and a new and faster cluster of processors were required to run with a reasonable speed.

At the same time, other improvements were carried out such as extending the model from Letellier to Morris so that changes in the flow hydrodynamics on Aux Marais River or the Buffalo Creek would develop without being constrained by the downstream boundary. (The main features of importance are indicated in Figure 1.

More bridges were added to the model, in particular on Buffalo Creek, the Aux Marais River, and under railway tracks that were affecting the propagation of floods.
Another important modification to the modelling of the scenarios was that they were simulated with four different flood hydrographs (instead of the same 2006 hydrograph): 1:10, 1:50 and 1:100 year return period annual floods and the 1:20 year summer flood. This allowed a wide range of flood severity and consequently a progression in the protection required in each case. Spring flood hydrographs at Walhalla were developed by the USACE, summer flood hydrographs at Walhalla were developed by Manitoba Water Stewardship (MWS) and the remaining hydrographs were developed by NRC.

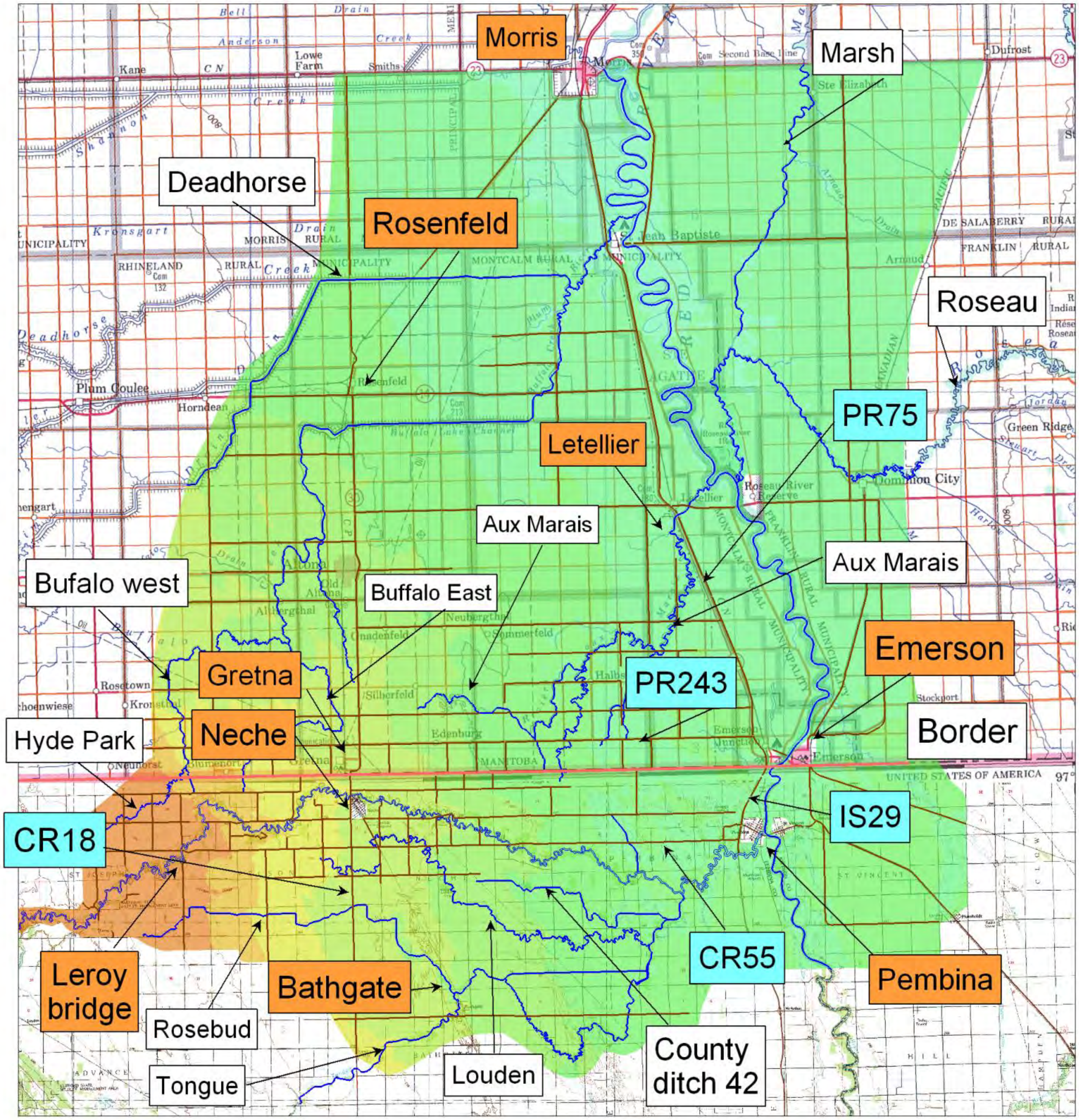


Figure 1 - General layout of phase 3 model

2. The Telemac-2D model – Blue Kenue

The Telemac software is developed by the *Laboratoire national d'hydraulique et environnement d'Électricité de France* (EDF) in Chatou. It solves the two-dimensional shallow water equation using finite element techniques, and is used by many organisations around the world. Telemac 2D is now available as an open source software where users have access to all the source codes.

The pre- and post-processor for Telemac is Blue Kenue, software developed by the Canadian Hydraulics Centre, which is freely available.

3. Preparation of the Phase 3 model

The reader is encouraged to read first Reference 2 that describes in more detail the previous models. Phase 3 model includes the same details as the previous phase 2, with additional improvements which are described below.

Phase 3 model has in excess of 359 000 nodes and close to 717 000 elements.

The geographic system in which the model was prepared is UTM (Universal Transverse Mercator) zone 14. All levels are referenced to CGVD28.

3.1 Model extension

Since a significant amount of water was going to be redirected North over the border, instead of moving East to the Red River, it was necessary to relocate the northern model boundary further to the north. The new model was extended from Letellier to Morris, so that the known data from the Water Survey of Canada (WSC) gauge on the Red River could be used directly to control this boundary.

The model was also extended slightly to the west, around Leroy, so that any outflow from the Pembina River would be able to flow towards Rosebud Coulee or Hyde Park Coulee. See
Figure 2 - The extended areas are shown in green.

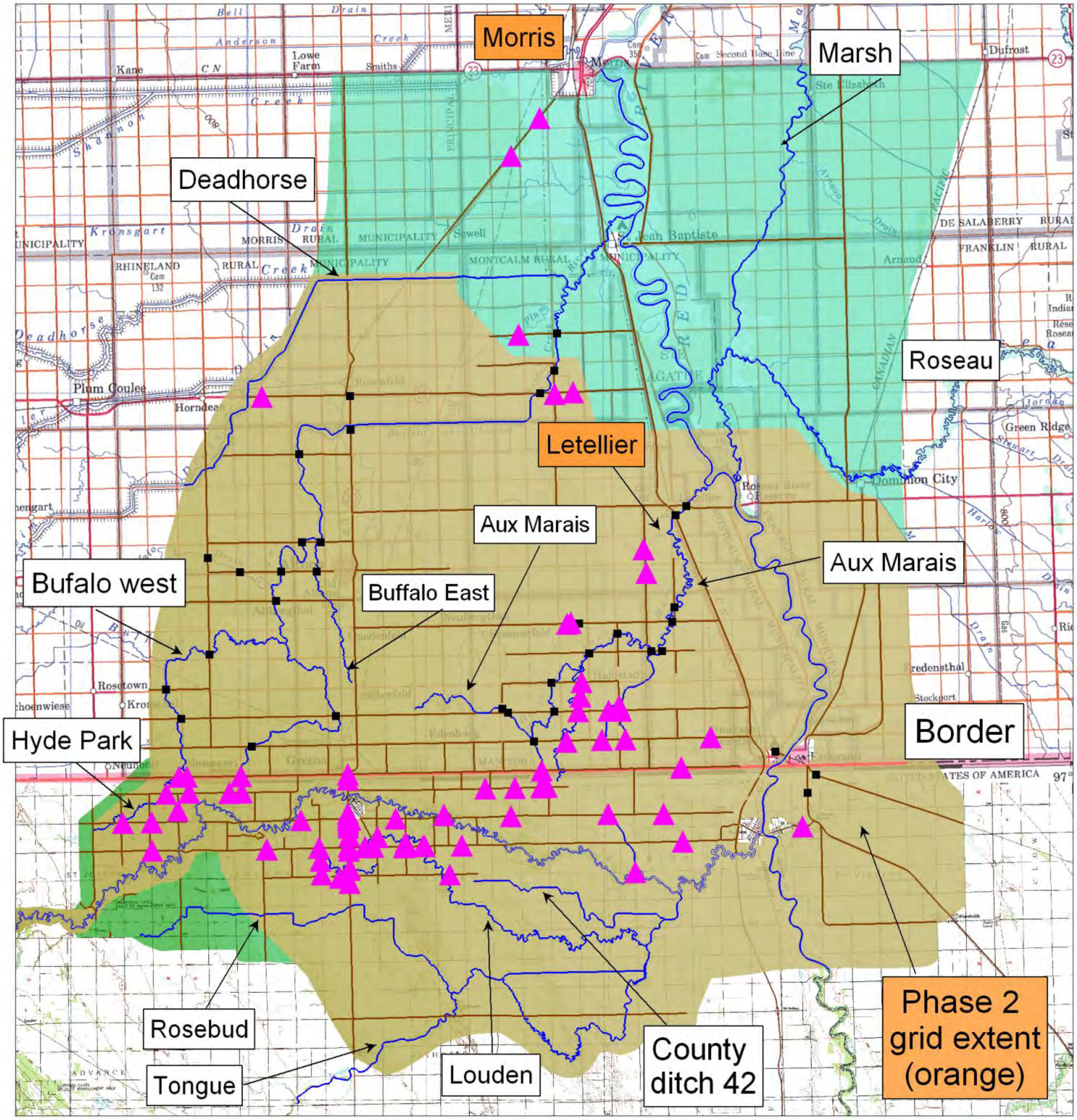


Figure 2 - General layout of phase 3 model showing culverts (red triangles) and bridges (black squares) represented in the model

3.2 Old Pembina River Meanders

The previous model showed that breakouts along the Pembina River occurred at the old meanders with such flows being controlled by the elevation of their former banks and not by the banks along the main channel. This had only been previously corrected for a few cases. The present model was further improved by setting the proper elevation of the banks of many old river cannels, providing a better representation for the breakouts and their discharge. This has resulted in a slightly larger capacity of the Pembina River main channel.

3.3 Modelling Small Rivers

One of the main differences from the previous model is the description of the smaller rivers and coulees so that a better representation of the channels' conveyance could be achieved. Large portions of Aux Marais, Buffalo, Louden, Rosebud and Tongue were processed as showed on Figure 3. The channels, which were previously described with a 30 to 40 m grid, are now described with a mesh size better suited for these smaller rivers: 3 to 10 m. The finer cross sections provided a more accurate estimate of conveyance for the channels.

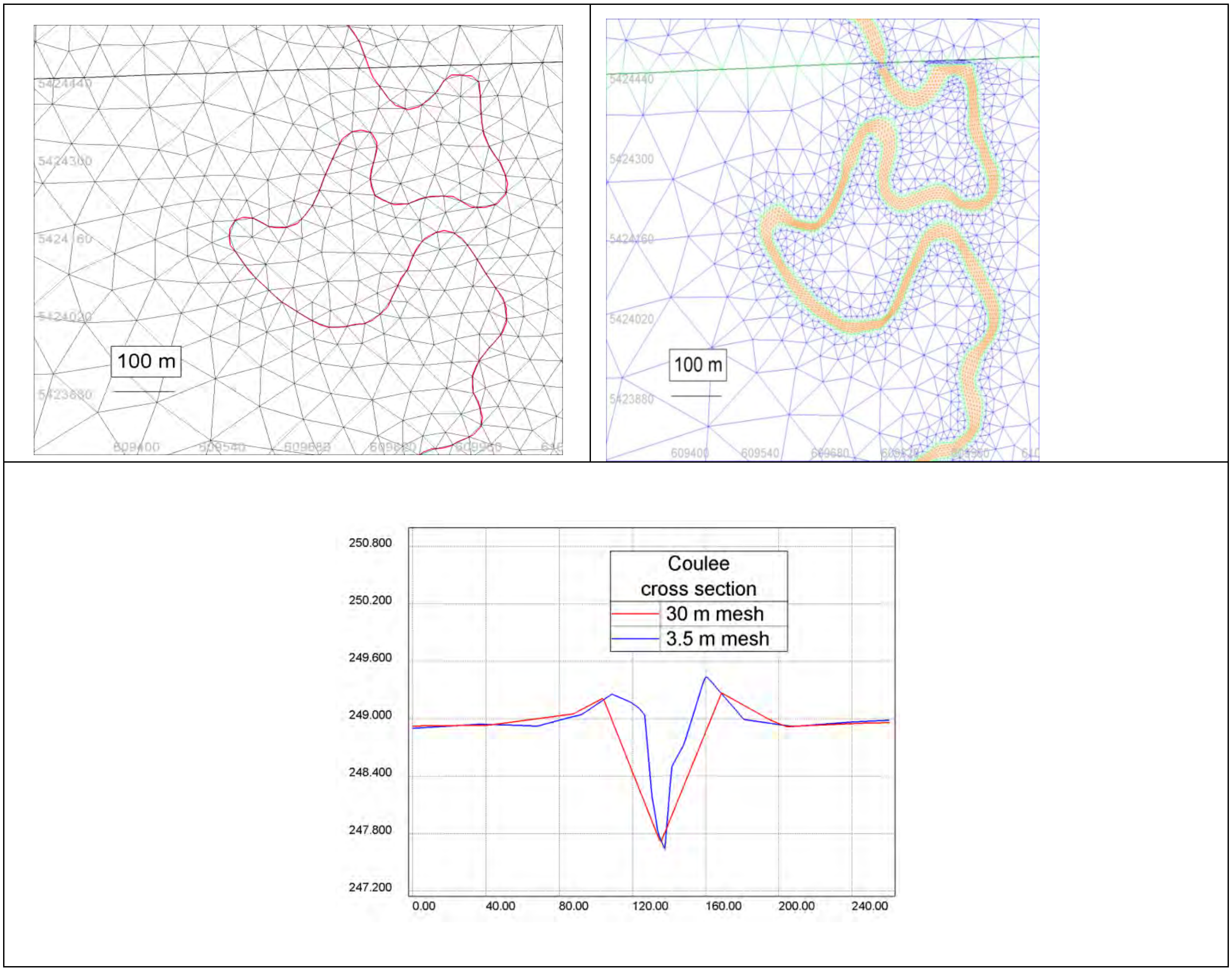


Figure 3 – Refinement of smaller river description, and corresponding channel cross section

The Roseau River and Marsh River were included in the model as a coarse mesh since their flooding was not critical to the present study.

The numerical description of the Pembina River main channel remained unchanged with a mesh element size of about 7 m x 20 m. With this grid definition, the main channel is described by a series of cross sections having 7 points, every 20 m along the stream. The description of the Red River channel also remained unchanged with 50 m elements along the river, and between 20 and 35 m crosswise.

3.4 Roads

Roads were added wherever a new bridge was included in the model (many of which having been surveyed recently), so that water which would be stopped by the road embankment, would be directed towards the bridge opening. They are shown on Figure 2.

The Morris north extension of the model included PR 75, 30, 217, 14, 23, 200 and 246 with the CP rail embankment south west of Morris.

3.4.1 Two short roads close to the River

Most of the important roads were included in the model. Two short sections, which were not specifically in the model, were identified and examined to assess their influence on flooding. This assessment is presented in Appendix 1.

3.5 River runoff flows

The model was prepared so that local runoff could be simulated along some of the main channels: Louden Coulee; Rosebud Coulee; Aux Marais River; Buffalo River; the drainage regions between Walhalla and Neche; Neche and Pembina; and the drainage region between Akra and Bathgate. Inflows were injected at several locations (shown on Figure 4) in an attempt to distribute the flow volume over the drainage basin rather than injecting all the hydrographs at one location.

For Aux Marais and Buffalo, the estimated runoff was distributed over 4 locations. For each location the volume was based on the local upstream drainage area and the timing of their hydrographs were adjusted so that the 2006 total flow measurements at Christie and Rosenfeld respectively, were correctly reproduced. The same volume distribution and timing were kept for the 2009 flood and the 4 return period events.

For Louden Coulee and Rosebud Coulee a simpler injection at 2 locations was chosen; the volume of flow was prorated based on upstream drainage area, but their timing was identical.

The runoff from the two drainage regions between Walhalla and Neche and between Akra and Bathgate, were injected at one location, close to the edge of the model, while runoff between Neche and Pembina was injected at two locations shown on Figure 4.

The derivation of the hydrographs for these rivers is presented in presented in Section 9.

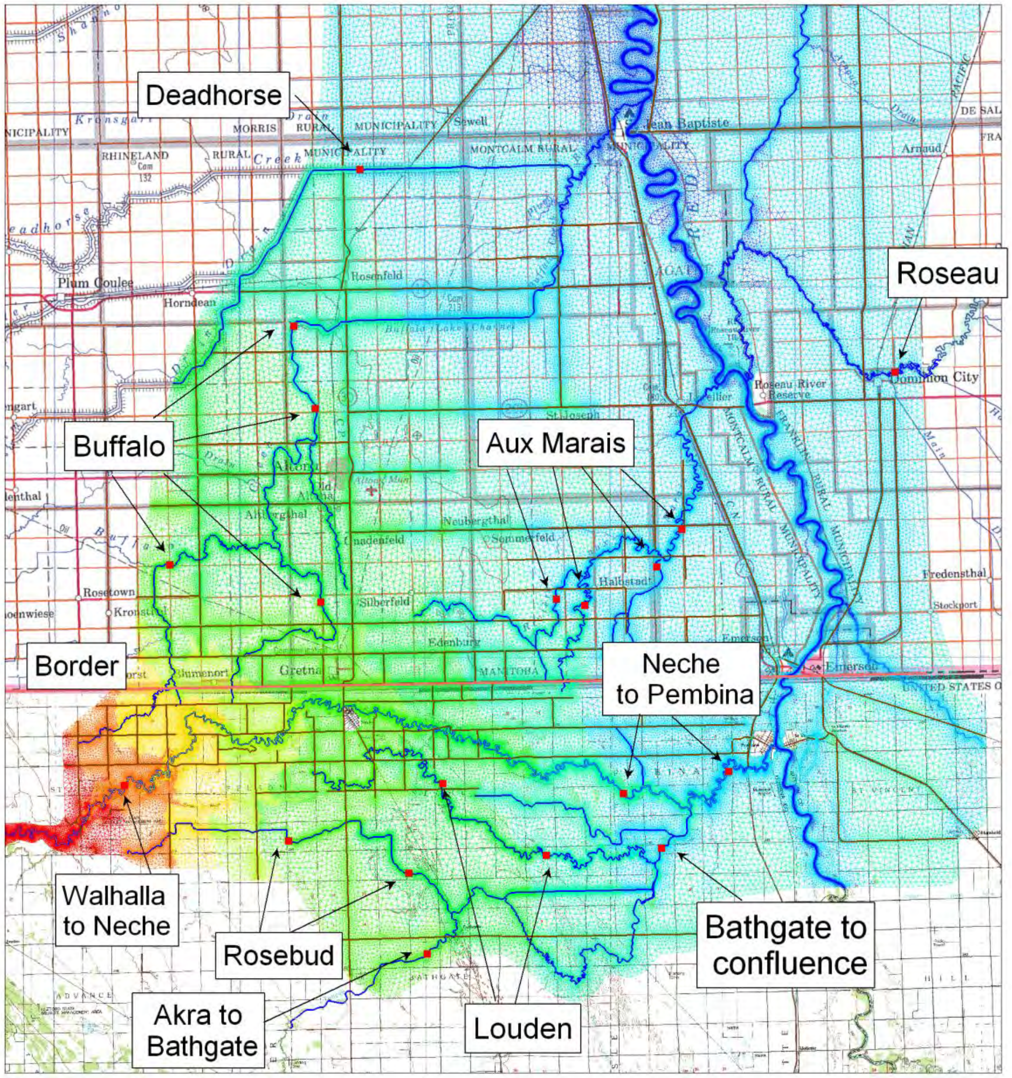


Figure 4 - Location of local runoff injection

3.6 Topography

Several LiDAR surveys were used for defining the topography. Their coverage is represented in Figure 5 which indicates also the year the surveys were obtained. Reference 1 explains in detail the characteristics of each of the surveys.

It should be noted that the match between the edges of the various surveys was very good (in most cases differences were less than ± 10 to 20 cm (3.9 to 7.9 inches)). Also the 1999 survey was provided with a 5 m resolution; therefore it was not as accurate as the others (with a 1 m resolution). Because of this, the thalweg of Buffalo River was lowered by 30 cm to compensate for its coarse description.

In places where there was no LiDAR coverage for the model, land elevation was estimated from local topography maps.

Because of the large number of data points in the LiDAR data product (estimated at 700 millions) they had to be reduced to a more manageable set without losing the topographic information. A coarser resolution was prepared in the flat flood plains, still maintaining a high resolution to describe the roads and the river channels. A final set of 16 millions points was developed and applied to the 359 000 nodes in the model, a sample of which is shown on Figure 6. In this case all the data points describing the roads and the river main channels were kept but a coarser 25 m (82 ft) grid resolution was retained for describing the flood plains.

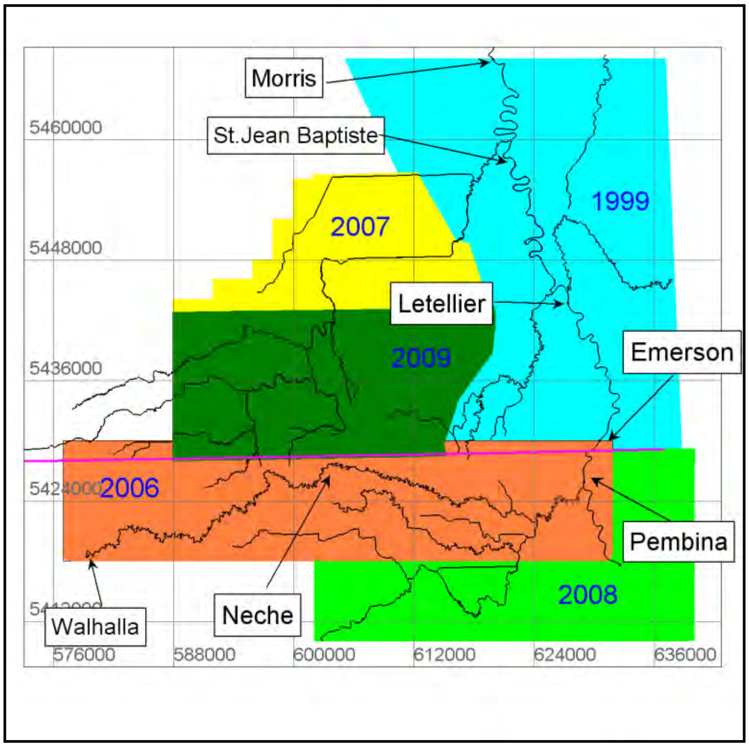


Figure 5 - LiDAR overall coverage

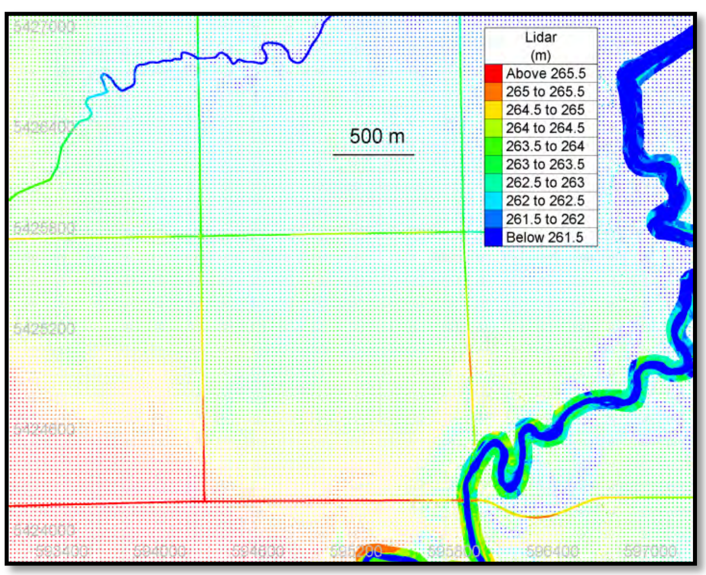


Figure 6 – Sample of reduced LiDAR data information

3.7 River bathymetry

The same river bathymetry was used in this version of the model as with the previous phase two version of the model.

The Red River was surveyed in 2009 by Agriculture and Agri-Food Canada (AAFC) from 10.5 km south of the Pembina River junction to the bridge at Letellier. AAFC had also previously surveyed a short portion of the river, a few kilometres downstream from St-Jean Baptiste. This short bathymetry measurement was used to prepare the bathymetry, using linear interpolation, for the portion of the Red River from Letellier to Morris.

3.8 Culverts and Hydraulic Structures

In the spring of 2010, North Dakota State Water Commission (NDSWC) surveyed many roads in the region surrounding Neche, with particular attention to Hwy 18, in order to identify culverts, boxes and bridges which had not been previously inventoried. (Figure 7)

In the fall of 2010, Agriculture and Agri-Food Canada (AAFC) inventoried bridges and culverts in the Aux Marais region (Figure 8), and in the spring of 2011 they surveyed the Buffalo Creek area. These culverts and bridges were added to the existing infrastructure in the model, increasing the total number of culverts represented in the model from 36 to 73. The list of culverts is shown in Table 1.

It is to be noted that when several large boxes (larger than 1.5m (5 ft)) were present under a road they could not be represented as culverts, as most of the time they would not operate at full flow. They were considered as bridges with a water passages corresponding to the width of the box assembly.

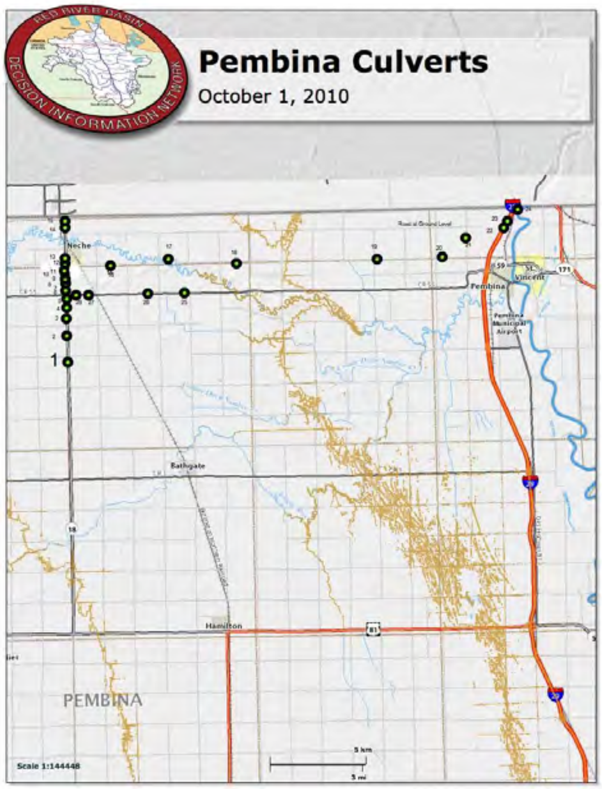


Figure 7 - Surveyed culverts in the Neche area (courtesy NDSWC)

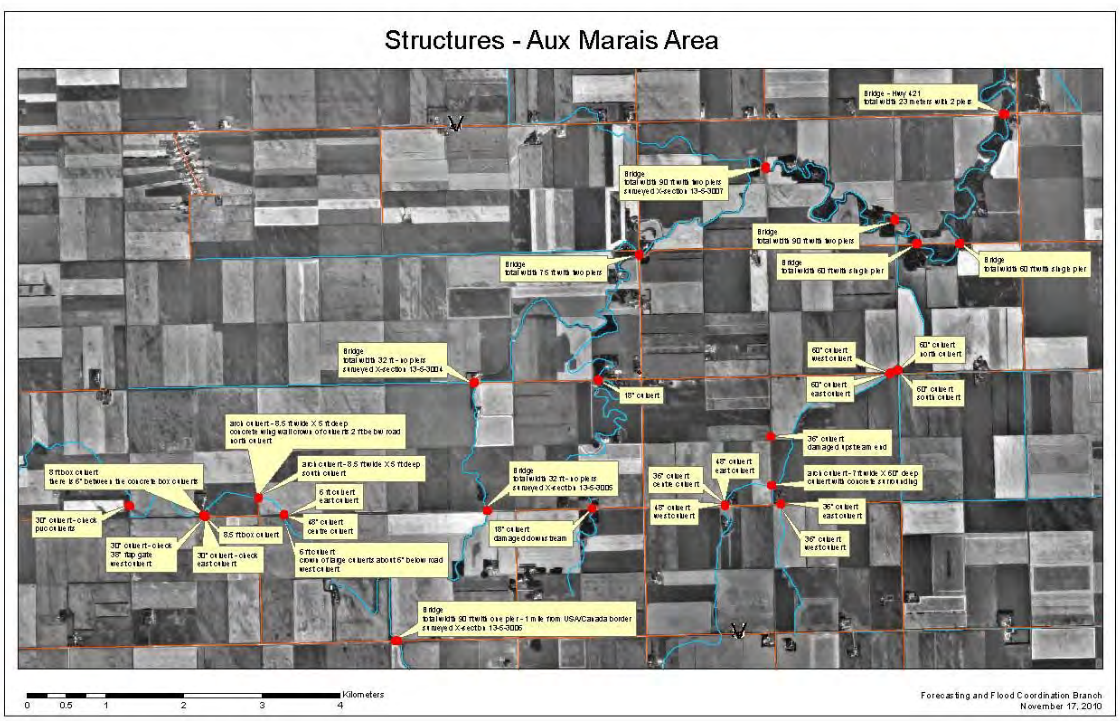


Figure 8 - Surveyed culverts in Aux Marais area (courtesy Agriculture and Agri-Food Canada)

UTM_X	UTM_Y	Culvert Number	Description	Diameter (m)	Number of pipes	Length (m)
596499.3	5428380.5	1	BD_ROAD_DIKE_NO_4E	0.762	1	10.363
596060.6	5428375.0	2	BD_ROAD_DIKE_NO_4W	0.6096	1	12.192
599535.0	5428421.7	3	BD_ROAD_DIKE_NO_5	0.762	1	7.62
616456.0	5428751.0	4	BD_ROAD_DIKE_NO_6	1.02	4	19.126
596628.1	5427504.4	5	TWP_ROAD_NO_1	0.9144	2	18.288
598886.6	5427470.2	6	TWP_ROAD_NO_4	0.4572	1	15.85
609787.8	5424521.4	7	Louden_coule_across_road	1.52	2	12
613282.8	5427719.7	8	TWP_ROAD_NO_8	0.772	3	10.668
614943.3	5427760.6	9	TWP_ROAD_NO_9	1.5	4	7.315
616421.4	5427800.0	10	TWP_ROAD_NO_11	0.6096	1	12.497
616740.9	5427810.1	11	TWP_ROAD_NO_15	0.6096	1	11.278
621722.3	5423039.3	12	drain_under_HY55_back_to_Pembi_R	1.22	4	10
595349.6	5427427.0	13	Hyde_Park_Coulee_Large_box	1	5	5
607216.6	5424989.8	14	Louden_coule_across_road	1	3	10
608792.0	5424680.7	15	Louden_coule_across_road	1	3	10
609865.9	5424481.7	16	Louden_coule_across_road	1.52	2	12
625963.5	5430616.4	17	across_RD_drain_close_to_RED	0.61	2	10
603939.3	5424397.7	18	from_Pembina_County_2	1.07	1	11
608245.3	5426028.2	19	from_Pembina_County_3	1.22	2	11
602913.4	5425941.9	20	from_Pembina_County_5	1.22	1	11
608709.8	5424423.1	21	from_Pembina_County_18	0.91	1	11
600992.3	5424300.8	22	from_Pembina_County_94	0.91	1	11
606513.7	5424428.6	23	from_Pembina_County_98	0.76	4	11
606994.8	5424443.0	24	from_Pembina_County102	1.35	2	11
611961.0	5424529.3	25	from_Pembina_County104	1.07	1	11
620187.7	5426295.3	26	from_Pembina_County_47	1.07	3	10
624384.8	5424735.5	27	from_Pembina_County105	1.22	2	10
592896.7	5425789.8	28	from_Pembina_County_68	1.22	1	10
594539.2	5424245.4	29	from_Pembina_County_50	0.91	2	10
595997.6	5426479.2	30	from_Pembina_County_66	0.61	2	10
604077.3	5422896.2	31	from_Pembina_County__1	0.91	1	10
594528.1	5425820.3	32	from_Pembina_County__7	0.7	4	10
611287.8	5422901.0	33	from_Pembina_County_85	0.91	2	10
605107.7	5422700.8	34	my_own_to_go_under_road	1	1	10
631119.1	5425592.7	35	culvert_under_171_east_Pembina	5.6	1	25
624300.0	5428913.3	36	across_RD_drain_close_to_RED	0.61	2	10
605639.5	5423161.8	37	2-4'Hx6'W_Concrete_Box	1	3	12
605632.6	5423165.3	38	2-4'Hx6'W_Concrete_Box	1	2	12
605659.3	5423248.7	39	Highway_#18_survey_oct2010	0.762	1	10
605614.3	5424229.8	40	Highway_#18_survey_oct2010	0.6096	1	10
605597.9	5424471.5	41	Highway_#18_survey_oct2010	0.914	1	10
605635.9	5425053.3	42	2-30"Hx50"W_CMP_arch	1.1	2	12
605575.7	5425523.4	43	Highway_#18_survey_oct2010	0.6096	1	10
605574.8	5425714.9	44	Highway_#18_survey_oct2010	0.762	1	10
605577.3	5425848.8	45	1-41"Hx70"W_CMP_arch	1.1	2	12

UTM_X	UTM_Y	Culvert Number	Description	Diameter (m)	Number of pipes	Length (m)
605564.3	5425991.6	46	Highway_#18_survey_oct2010	0.762	2	10
605558.3	5426129.3	47	Highway_#18_survey_oct2010	0.9144	3	10
605557.5	5426335.6	48	Highway_#18_survey_oct2010	0.9144	3	10
605548.6	5428235.2	49	Highway_#18_survey_oct2010	0.762	2	10
605550.9	5428520.4	50	Highway_#18_survey_oct2010	1.524	2	10
610953.6	5426248.7	51	1.5_miles_S_Border_surv_oct2010	0.6096	1	10
614724.6	5426161.8	52	1.5_miles_S_Border_surv_oct2010	0.6096	1	10
623313.2	5426298.4	53	1.5_miles_S_Border_surv_oct2010	0.9144	1	10
618697.4	5433713.0	54	18"culvert_AuxMarais_sma_branch	0.46	1	10
618502.7	5432065.6	55	18"culvert_AuxMarais_sma_branch	0.46	1	10
620212.1	5432095.2	56	44"culvert_AuxMarais_sma_branch	1.12	3	12
604015.7	5423675.6	57	same_as_38	1	5	12
599596.1	5427502.1	58	TWP_ROAD_NO_5	0.4572	1	12.802
605651.7	5422424.4	59	Highway_#18_survey_oct2010	0.6096	1	10
616334.0	5465415.7	60	under_rail_west_of_Morris	1	3	12
614730.7	5463292.5	61	under_rail_west_of_Morris	1	3	12
620808.2	5432380.3	62	trib_marais_smallbridge2_1m	1	2	12
615157.0	5453236.5	63	on_tributary_of_Buffalo_1_36in	0.9	1	12
622179.8	5441172.7	64	add4_2x39in_Marais_underroad	1	2	10
617877.4	5436979.1	65	add1_culv_2x0.9diam_underroad	0.9	2	10
618087.7	5437045.0	66	add2_culv_2x0.9diam_underroad	0.9	2	10
622342.4	5439839.2	67	add3_culv_2x1diam_underroad	1	2	10
619827.6	5430460.6	68	culAuxMarais_1mile_northborder	0.91	2	10
621165.3	5430498.9	69	culAuxMarais_1mile_northborder	0.91	1	10
605689.7	5425986.7	70	add_to_drainNeche_small_basin	1	3	10
617839.9	5430373.8	71	add_on_small_trib_marais	0.76	2	10
618627.2	5432895.7	72	add_on_small_trib_marais	0.76	2	10
620908.1	5432101.7	73	add_on_small_trib_marais	0.91	2	10

Table 1 - List of groups of culverts represented in the model

3.9 Bridges

As with the previous version of the model, all bridges along the Pembina River and along the Red River were simulated with an increased bottom friction. The smaller bridges over Aux Marais River and Buffalo River, as well as some of the recently surveyed water passages under roads were represented in the model with their specific width and bottom elevation. A total of 36 bridges were defined as listed on Table 2. They are represented by the black squares on Figure 2. Included in the list are three bridges under railway tracks, close to Emerson and St. Vincent, which control the East and West bypass overflows.

UTM coordinate X	UTM coordinate Y	Average Width (m)	Elevation (m)	
617315.9	5453294.0	24.0	233.2	Buffalo creek
617155.9	5451214.3	20.5	235.7	Buffalo creek
616361.6	5449939.0	23.8	235.2	Buffalo creek
624596.6	5443600.0	21.0	233.3	Aux Marais
623969.3	5443102.2	24.1	234.6	Aux Marais
623921.4	5437913.8	11.0	236.7	Aux Marais
623775.8	5437107.7	13.5	237.2	Aux Marais
623205.1	5435449.0	11.1	237.4	Aux Marais
622622.7	5435446.6	11.4	237.4	Aux Marais
620725.9	5436419.0	14.8	238.1	Aux Marais
619112.1	5435310.5	11.4	239.0	Aux Marais
618570.1	5436995.4	10.0	239.7	Aux Marais
617002.2	5433678.6	10.0	239.7	Aux Marais
617165.0	5432038.6	9.0	240.0	Aux Marais
616006.8	5430381.6	14.4	240.6	Aux Marais
614566.0	5431987.0	5.5	241.0	Aux Marais
614242.0	5432204.0	4.8	241.2	Aux Marais
629583.6	5429807.5	15.3	237.5	Emerson
631876.0	5428509.0	25.6	237.5	St Vincent
631408.5	5427493.8	24.2	238.5	St Vincent
631099.5	5425591.3	140.0	237.8	St Vincent
603806.7	5439932.0	6.6	245.3	Buffalo creek
601513.7	5438267.5	20.5	245.0	Buffalo creek
597764.0	5435261.2	9.1	247.7	Buffalo creek
595339.0	5433277.0	13.3	250.2	Buffalo creek
596173.5	5431648.8	16.8	251.8	Buffalo creek
604888.8	5431798.6	11.2	249.6	Buffalo creek
600139.2	5430075.0	7.0	252.6	Buffalo creek
602815.9	5446504.4	3.8	242.5	Buffalo creek
604023.6	5441577.5	2.1	244.0	Buffalo creek
597667.1	5440693.1	10.0	249.1	Buffalo creek
599478.2	5439891.5	12.0	247.0	Buffalo creek
601801.8	5439904.3	38.0	244.9	Buffalo creek
602986.0	5441579.6	4.2	244.0	Buffalo creek
605668.4	5449780.4	1.8	241.8	Buffalo creek
605707.7	5447873.0	5.0	242.5	Buffalo creek

Table 2 – List of small bridges represented in the model

3.10 Model boundaries

The model was run with:

- prescribed flows at the five upstream boundaries:
  - Pembina River at Walhalla
  - Red River, 10.7 km upstream from the confluence with the Pembina River
  - Tongue River with flows corresponding to those at Akra
  - Roseau River with flows corresponding to those at Dominion City
  - DeadHorse Creek
- prescribed elevations on the Red River at Morris.

The model was also allowed to have free outflow at its northern boundary west of Morris, overtopping PR 23, and on the main channel of Marsh River, east of Morris.

4. Model Calibration

The model was recalibrated using the 2006 flood as done in previous versions of the model. Water levels at Pembina, Neche and Walhalla were obtained from USGS hydrometric gauges. These were provided in the NGVD29 datum and converted to the same datum as the LiDAR survey (CGVD28) by subtracting 4 cm. Levels at Emerson, Letellier and Morris were obtained from Water Survey of Canada (WSC) hydrometric gauges.

Discharge data at Walhalla (hourly flows) and Neche were available from USGS.

The discharge on the Red River upstream boundary (daily flows) was obtained from the estimate at the Emerson gauge, from which 90% of the Pembina River flow at Walhalla, (shifted by 4 days) was subtracted, accounting for the Pembina flow which does not flow into the Red but rather flows north.

During the 2006 flood simulation, no runoff was considered in Rosebud and Louden coulees, as it was reported to be very small. The runoff on Aux Marais and Buffalo were considered, as measured by WSC at the Christie and Rosenfeld gauges respectively.

During the model calibration, the various friction coefficients were adjusted so that simulated levels closely matched measurements at the peak of the flood including:

- in the main channel;
- at the top of the banks (usually full of trees); and
- the flood plains.

During low flow periods, the bottom friction coefficients were kept the same as during peak flood, which explains the difference between simulated and observed levels (Figure 9). During the calibration, it has been assumed that the flows estimated by USGS at Walhalla and by WSC at Emerson were correct and did not need adjustment.

Table 3 shows the friction coefficients used. These coefficients are close to those expected from similar rivers. It should be noted that these coefficients are 2D, (they are applied with a velocity vector with two components) and they also reflect the fact that the continuous fluid has been discretized to a very fine mesh. Therefore they may not be directly comparable to coefficients found in 1D models.

Region	Friction coefficient (Strickler formulation)
Red River, Morris to Letellier	20
Red River, Letellier to Emerson	26
Red River, Emerson to upstream	35
Red River, top of banks	18
Pembina River, Walhalla to CR 55	39
Pembina River, CR 55 to Neche	45
Pembina River, Neche to Pembina	38
Pembina River, top of banks	15
Flood plain	20
Tongue River	40
Border drain	30
Coulees and drains	35 to 37

Table 3 - Bottom friction coefficients (Strickler formulation)

Figure 9 shows the model calibration performance comparing simulated to observed water levels at Walhalla, Neche, Pembina and Emerson. The model reproduces the gauge measurements very well when the flood is close to its peak. At Neche between April 13 and 24, levels were within 4 cm (1.6 inches) of the measurements. In these figures, day 0 corresponds to 1st April, 2006. The inflows at Walhalla were hourly average flows, whereas they were daily average flows for the Red River.

- **Walhalla levels.** The Walhalla gauge was not functioning during 5-6 April 2006 (day 4-5 in the simulation). The 0.5 m difference during day 6 to 10 may come from a local effect, such as ice. Separate runs were performed with a level boundary, forcing the model to reproduce the measured levels at Walhalla and to let it calculate the flow required. In this case the same kind of discrepancy in the flows was noted.

Figure 10 shows the model calibration performance considering water levels at Letellier and at Crossing 6 (see location on Figure 16) which had been measured by Manitoba Water Stewardship for a few days during the 2006 flood. Simulated levels on the US side of the crossing are about 6 cm (2.4 inches) higher than measured.

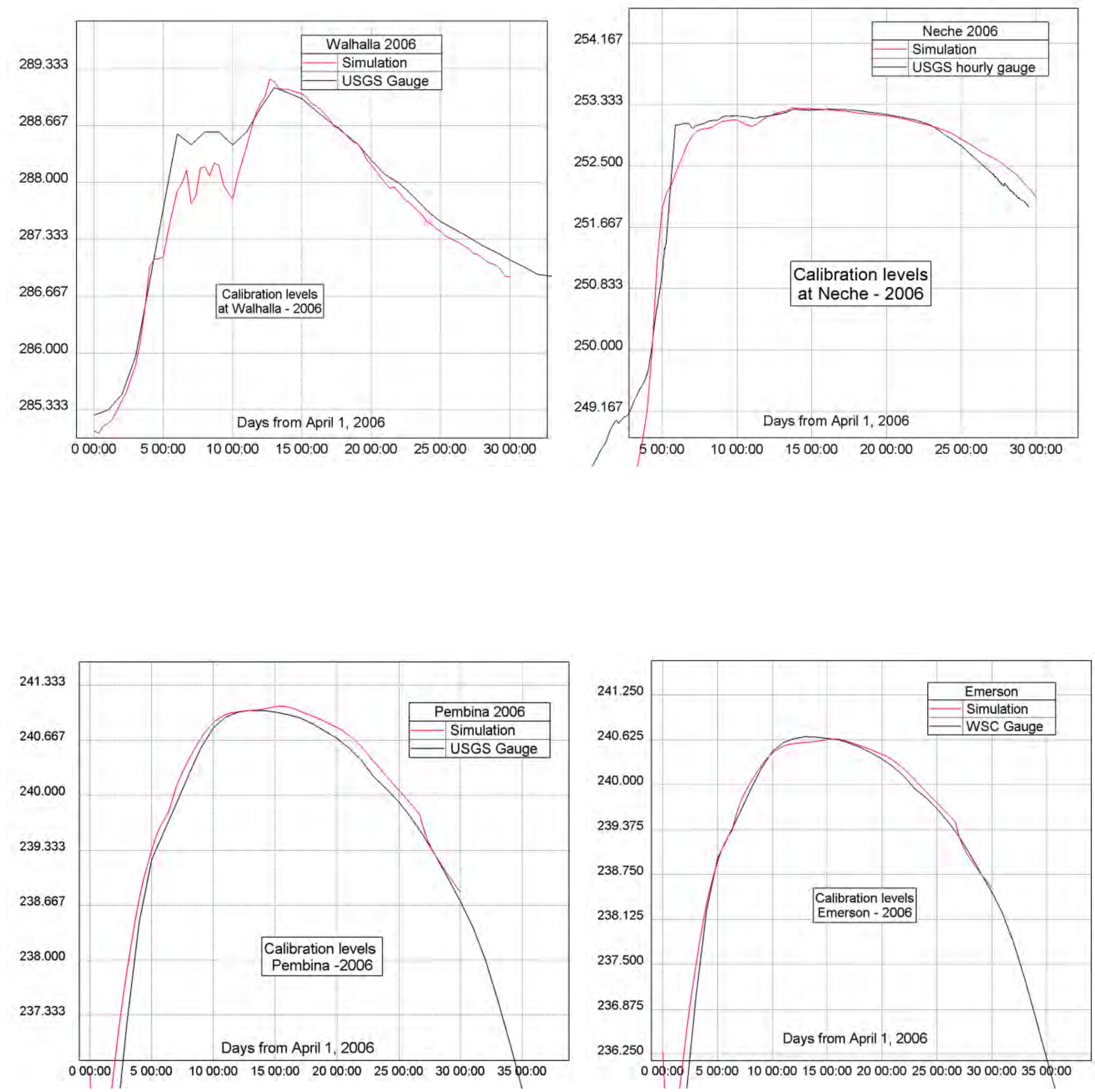


Figure 9 - Calibration water levels (in metres) - 2006 flood

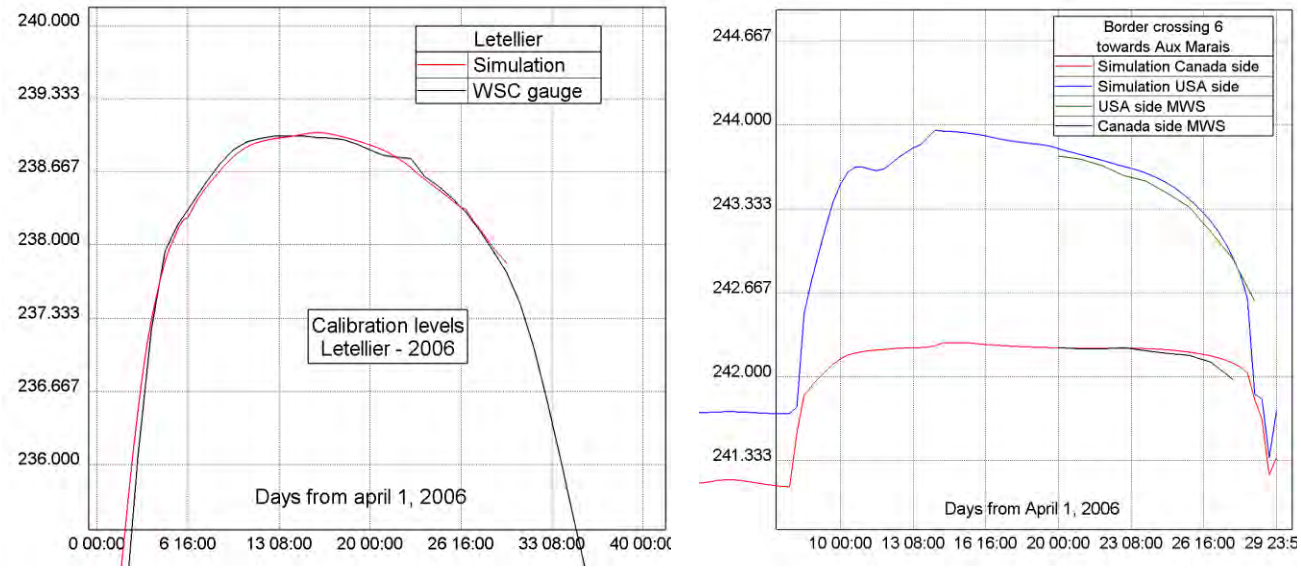


Figure 10 - Calibration water levels (in metres) for Letellier and Crossing 6 - 2006 flood

Figure 11 shows the discharge through a north–south section across the entire model at Neche, slightly west of Highway 18. It shows that at their peaks:

- 30 m^3/s (1059 cfs) flows in the fields north of Neche, in a west-to-east direction;
- 122 m^3/s (4308 cfs) flows in the fields south of Neche;
- 200 m^3/s (7063 cfs) flows through the main channel (note the increase of 15 m^3/s from the previous model due to the better representation of the banks of old meanders); and
- 350 m^3/s (12 360 cfs) is the total maximum flow, which is higher than the estimate from USGS of 306 m^3/s (10 806 cfs), but the USGS estimated the peak 2 days later than the model. Also, the model flow hygrograph shows a narrower peak, with a shape similar to the peak of the Walhalla hygrograph. At the beginning of the flood event, USGS flow estimate at Neche is higher than the flow at Walhalla; that indicates the existence of other significant inflows, but these were not modelled. It should be noted also that in the model, there is no loss of water between Walhalla and the Leroy Bridge. This may explain the peaks at Walhalla and Neche being close to each other.

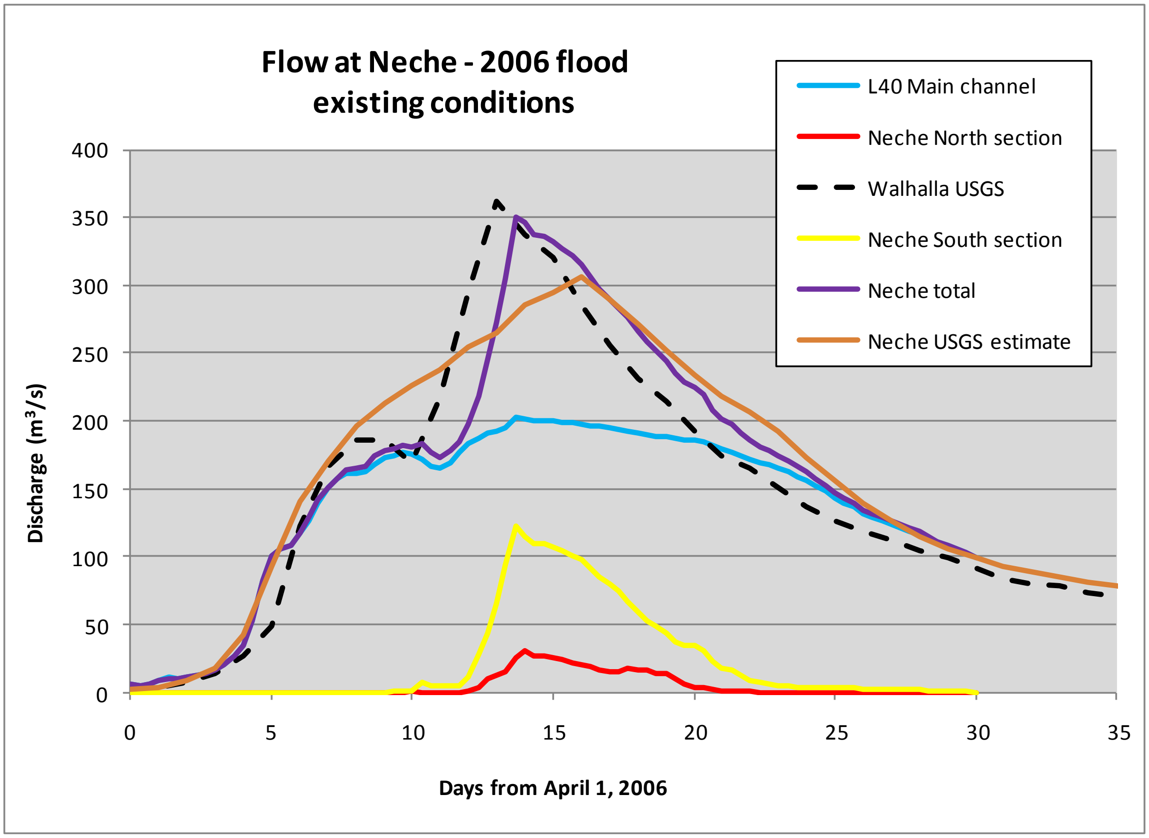


Figure 11 - Discharge at Neche through a north–south section across the whole model—comparison with gauge estimate

Similarly, Figure 12 shows the flows north and south of the River in sections across the flood plains located about 8 km downstream from Neche. The main channel shows a maximum flow of 145 m^3/s (5121 cfs), representing an increase of 13 m^3/s from the previous model.

The comparison of Figure 11 and Figure 12 shows that north breakouts occur downstream from Neche, resulting in an increase in flow going north, and that the flow within the main channel has decreased significantly from what it was at Neche (from 200 to 145 m^3/s). The total flow remains unchanged since no local inflow (runoff) was considered.

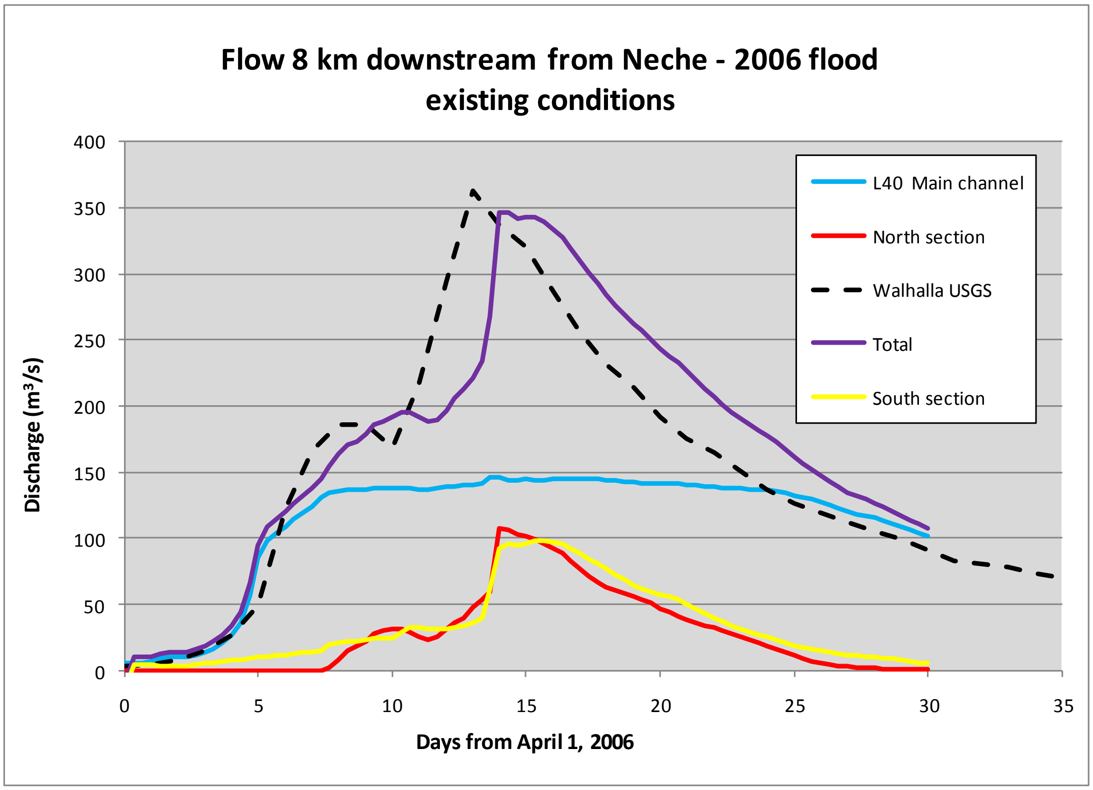
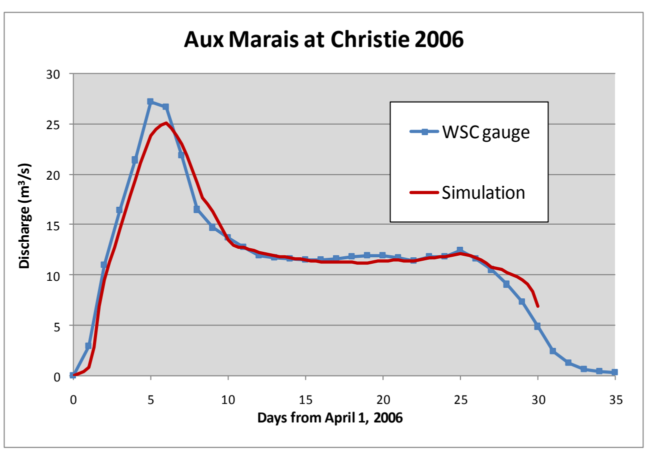


Figure 12 - Discharge 8 km from Neche through a north–south section across the whole model

Figure 13 shows a very good comparison in flows between the gauge estimates and the simulation for the Aux Marais River and Buffalo Creek. It is noted that the Buffalo Creek flow was due to local runoff occurring before the Pembina flood, whereas the Aux Marais flow began as runoff, followed by approximately 12 m^3/s (424 cfs) of overflow from the Pembina River which was reproduced by the model. This last constant flow estimate was used to calibrate the discharge through the four large culverts at Border Crossing 6.



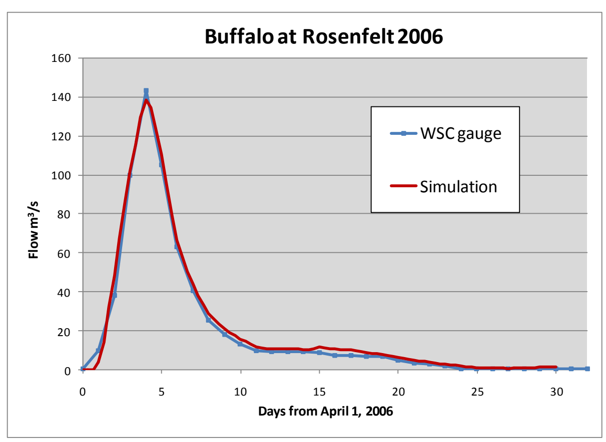


Figure 13 – Flow in Aux Marais and Buffalo Creek during the 2006 event

4.1 Comparison with Aerial Photos

The flood simulations with the new model were compared with aerial photos taken during the 2006 flood by the North Dakota State Water Commission. They are shown here in four sections (Figs. 13 to 16). The photos were taken at dates close to the peak of the flood.

In general, the flooding simulated by the new model is quite similar to the previous model. One of the improvement is visible on Figure 16, identified inside the red circle, which was previously shown as flooded due to a break out at an old meander. The meander has been closed in the new model, resulting in much less break out.

The new model included many more culverts under CR 18, and Louden Coulee was extended to one mile west of CR 18 (Figure 15). Even with these improvements, flooding on both sides of CR 18 looks more extensive than what is shown on the aerial photographs. Along the border there is still some flooding west of CR 18 (red circle), although flooding east of CR18 (yellow circle) produced by the model is more extensive than shown on aerial photographs. This seems to indicate several things:

- more refinements should be included, such as the ditches, which run along many roads but were not included in the model;
- the inflow at Walhalla may be overestimated, resulting in too much flow at Neche; this may be confirmed by the USGS estimate of the flow at Neche that was 45 m^3/s (1589 cfs) less than what the model had predicted (Figure 11); and
- there was a significant overtopping of the banks between Walhalla and Leroy, taking water away from Pembina River. This effect was not reproduced since the model, at this location, was only 800 m wide and did not allow water to flow North or West. Although the storage within these 800 m inside the numerous old meanders was modelled.

On Figure 16 the breakout shown in the yellow circle is simulated, but two breakouts (blue circle) still appear in the simulation, at the location of old meanders, even though they had been properly represented in the model. This seems to indicate that either there was too much model produced flow in the Pembina River main channel or its simulated conveyance was too small.

In spite of these deficiencies, many areas are still well simulated (for instance flooding around Horgan Ridge towards Rosebud Coulee in Figure 15), and it is felt that the present model can still be used very effectively to simulate the relative effect of changes in the infrastructure arrangements in the Pembina River flood plain.

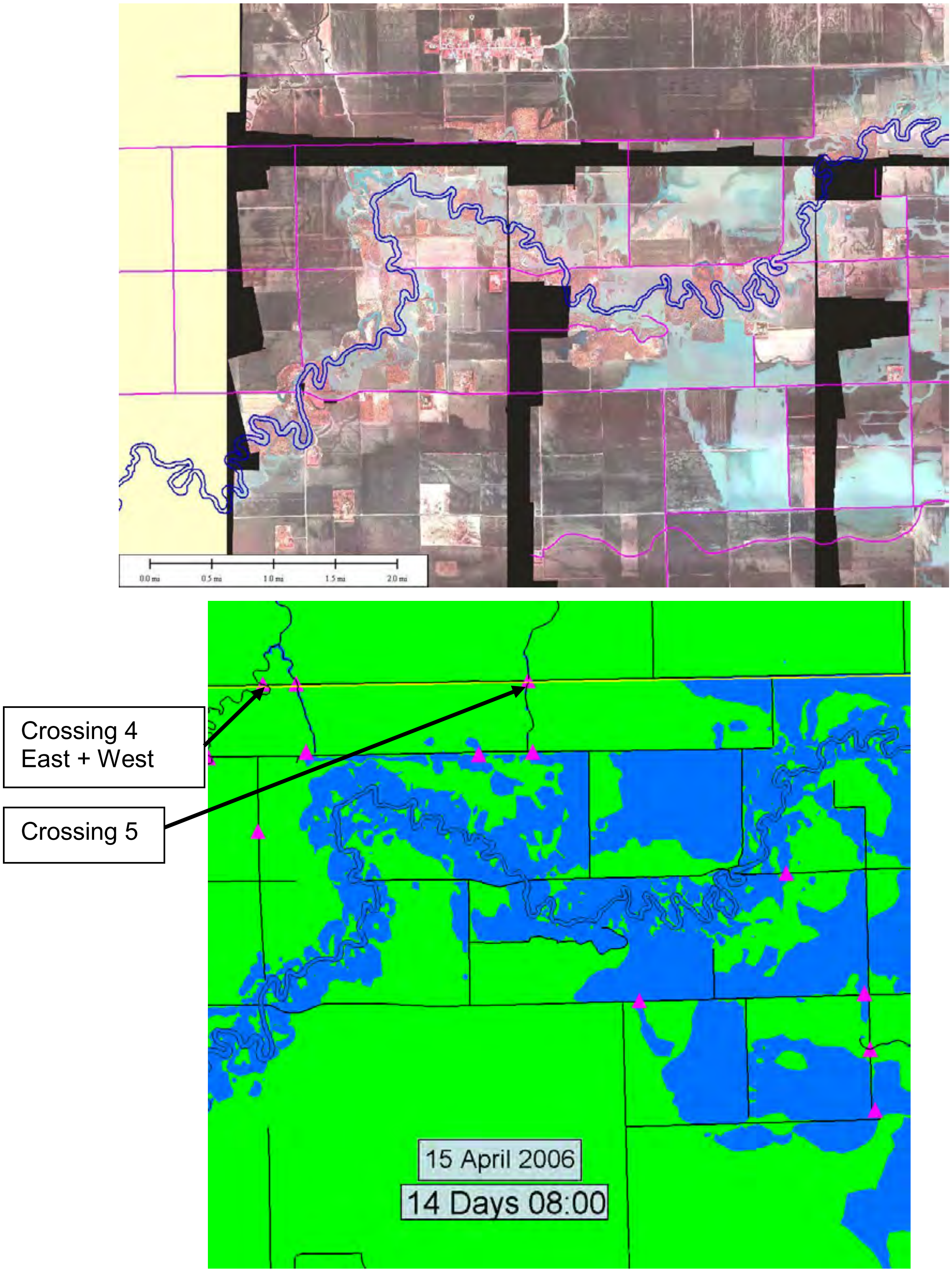


Figure 14 - Comparison of model simulated water extent versus aerial photos 1

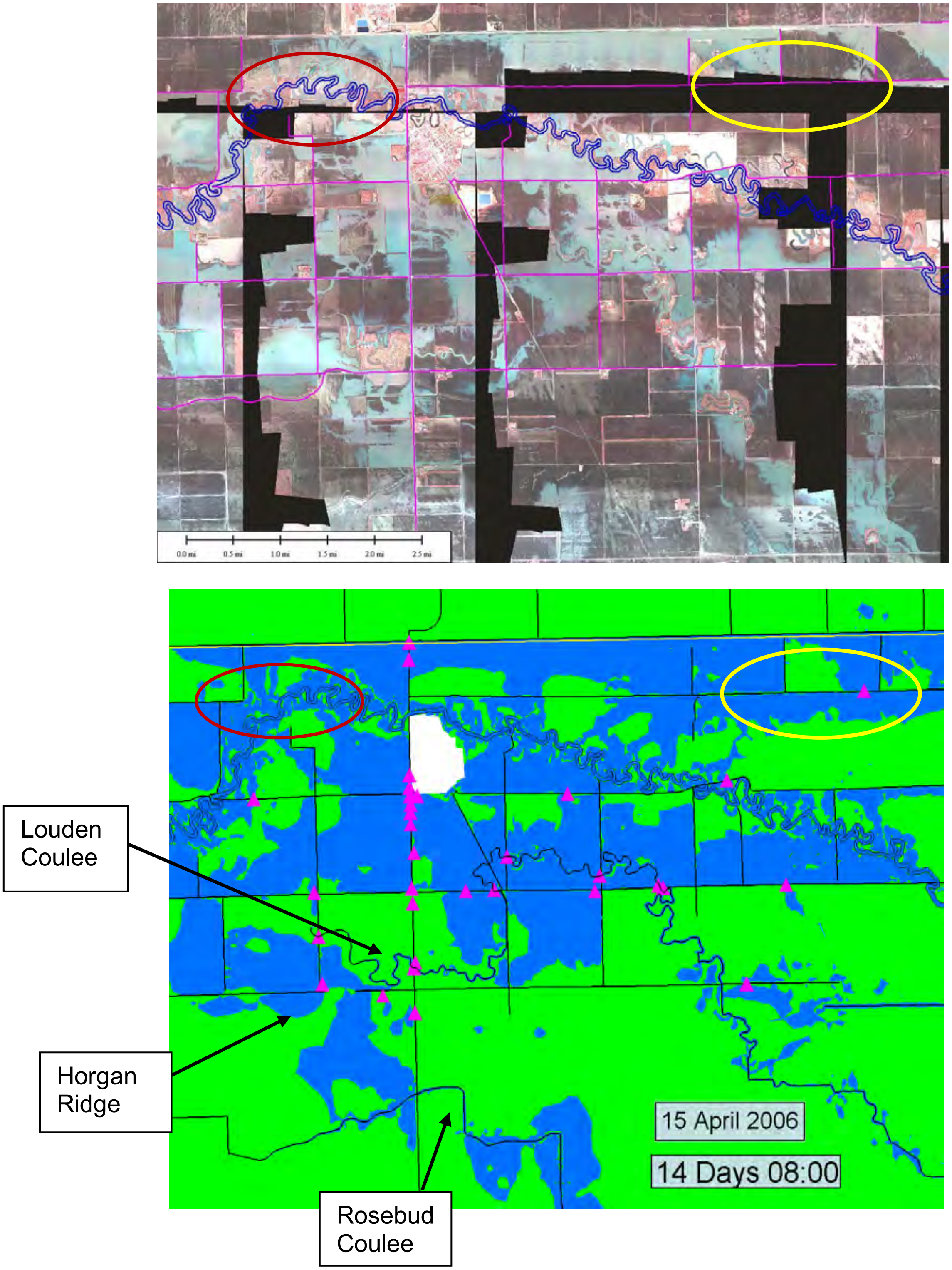


Figure 15 - Comparison of model simulated water extent versus aerial photos 2

(Note white areas are protected by dykes)

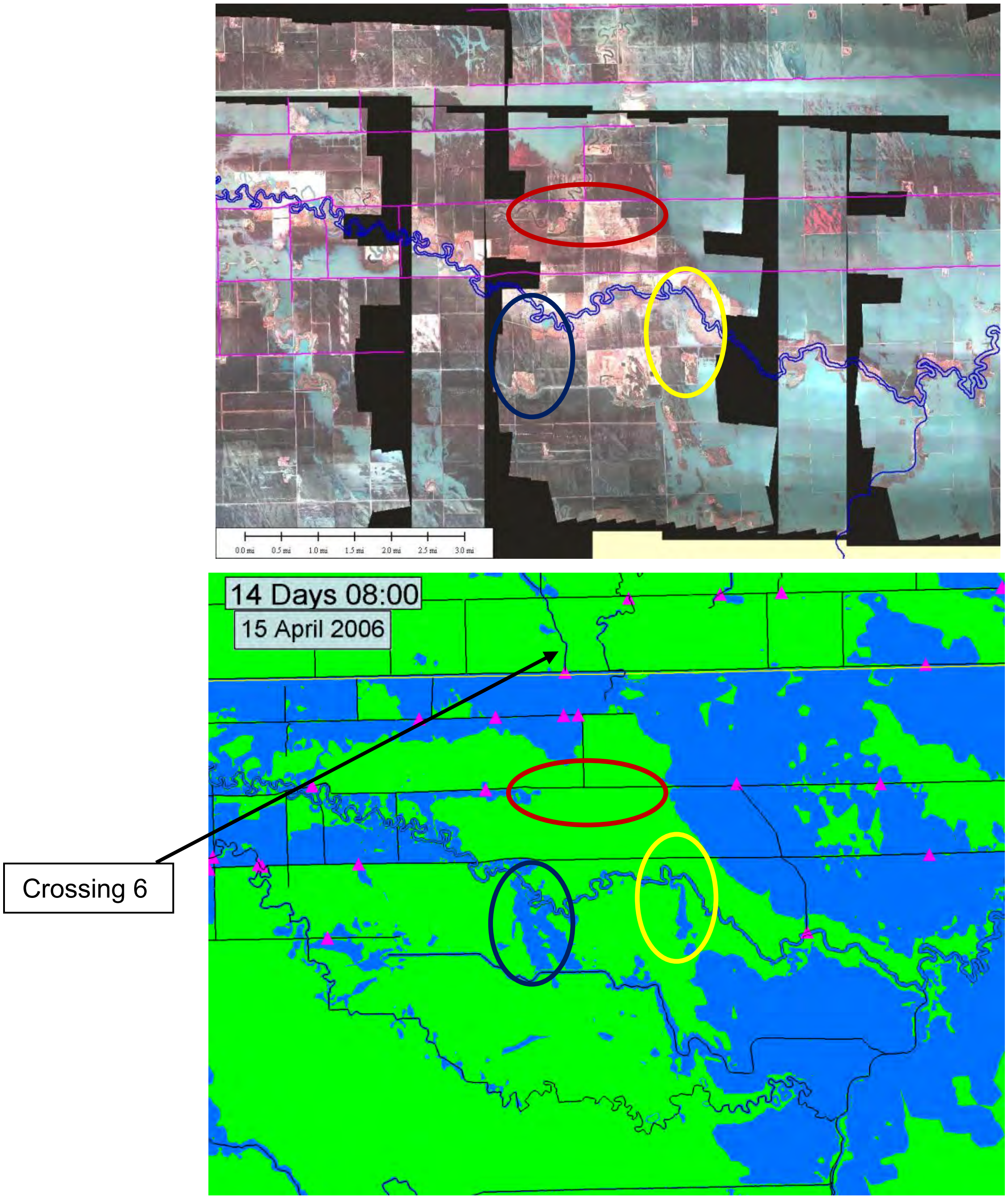


Figure 16 - Comparison of model simulated water extent versus aerial photos 3

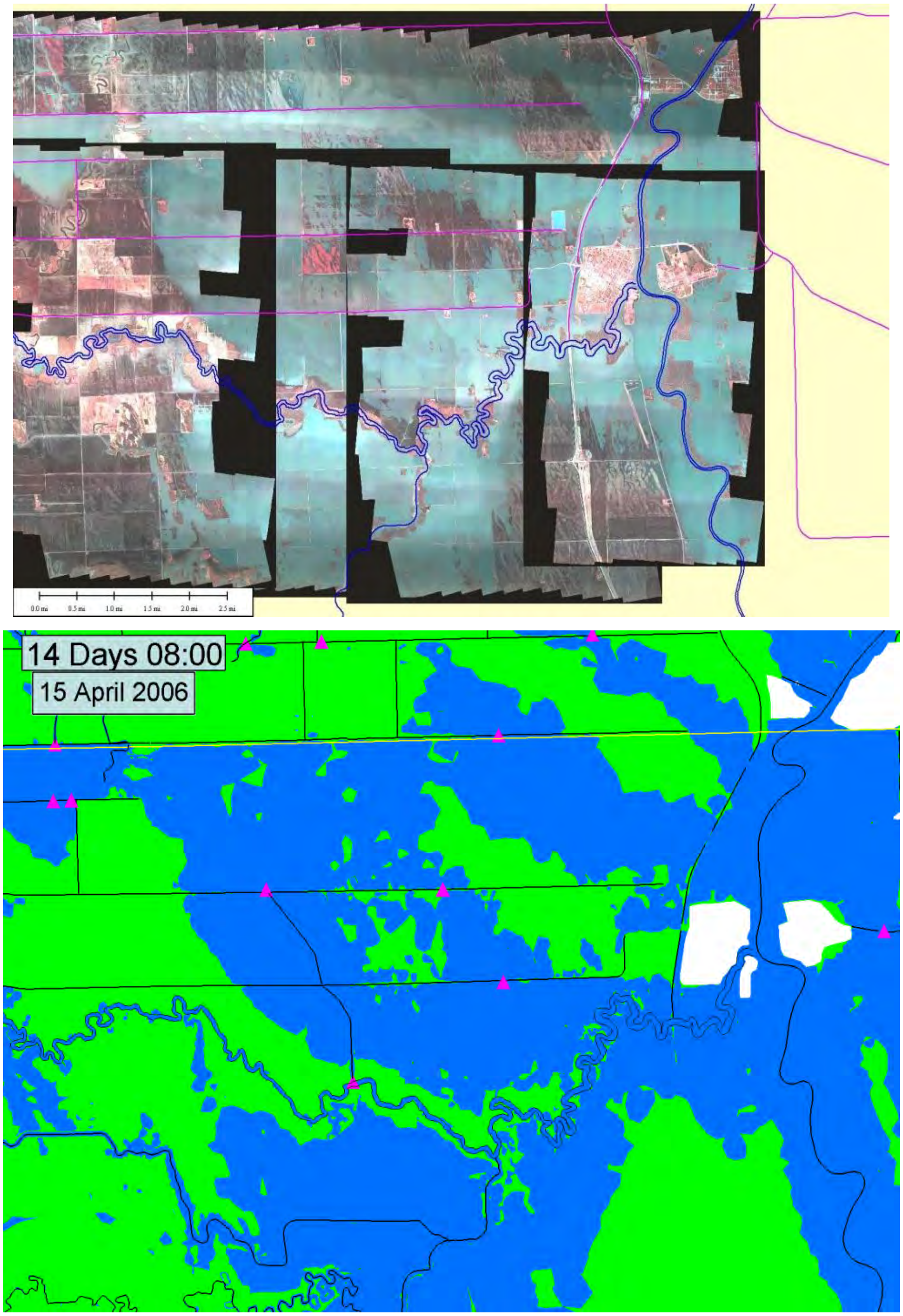


Figure 17 - Comparison of model simulated water extent versus aerial photos 4

5. Model Verification – 2009 flood

The 2009 flood was chosen for model verification, using the same type of boundary condition as the calibration period (2006 flood):

- Input flow hygrograph at Walhalla (hourly data) as estimated by USGS gauge in March-April 2009;
- Input flow hygrograph in the upstream boundary on the Red River (daily average), derived from the WSC gauge at Emerson, minus 90% of the Pembina discharge, which had been shifted by four days;
- Water level at the downstream boundary on the Red River, as measured by WSC gauge at Morris; and
- In the new model, runoff for Aux Marais River and Buffalo Creek were also included as estimated by WSC.

During the 2009 spring flood, erosion took place at the Switzer Ridge (see Figure 18) creating a narrow channel through the ridge. The site was surveyed by USACE in late January 2010 and these topographic data were included in the topography for the 2009 flood simulation. The erosion was assumed to have taken place very quickly (less than one hour) 24 hours before the photo was taken i.e. 20 April, 2009 at 13h00 or day 50 in the model simulations. Before that date, the data from the 2006 LiDAR survey were used for the whole ridge.

Figure 18 - Aerial photo of Switzer Ridge erosion during the 2009 flood - 21 April, 2009

5.1 Verification of levels and flows – 2009 flood

Figure 19 and Figure 20 show the verification levels at Walhalla, Neche, Pembina, Emerson and Letellier.

The model reproduced the gauged measurements well when the flood is close to its peak. The largest discrepancy was at Emerson, on the order of 20 cm (7.9 inches) one week before the peak. There is a time shift on the Red River where the modelled flow arrives 1.5 days later than observed. For the 2006 event a 4 days time shift between the Red and the Pembina at Walhalla was set. For the 2009 event this time shift would have needed to be adjusted. On these figures, day 0 corresponds to 1 March 2009.

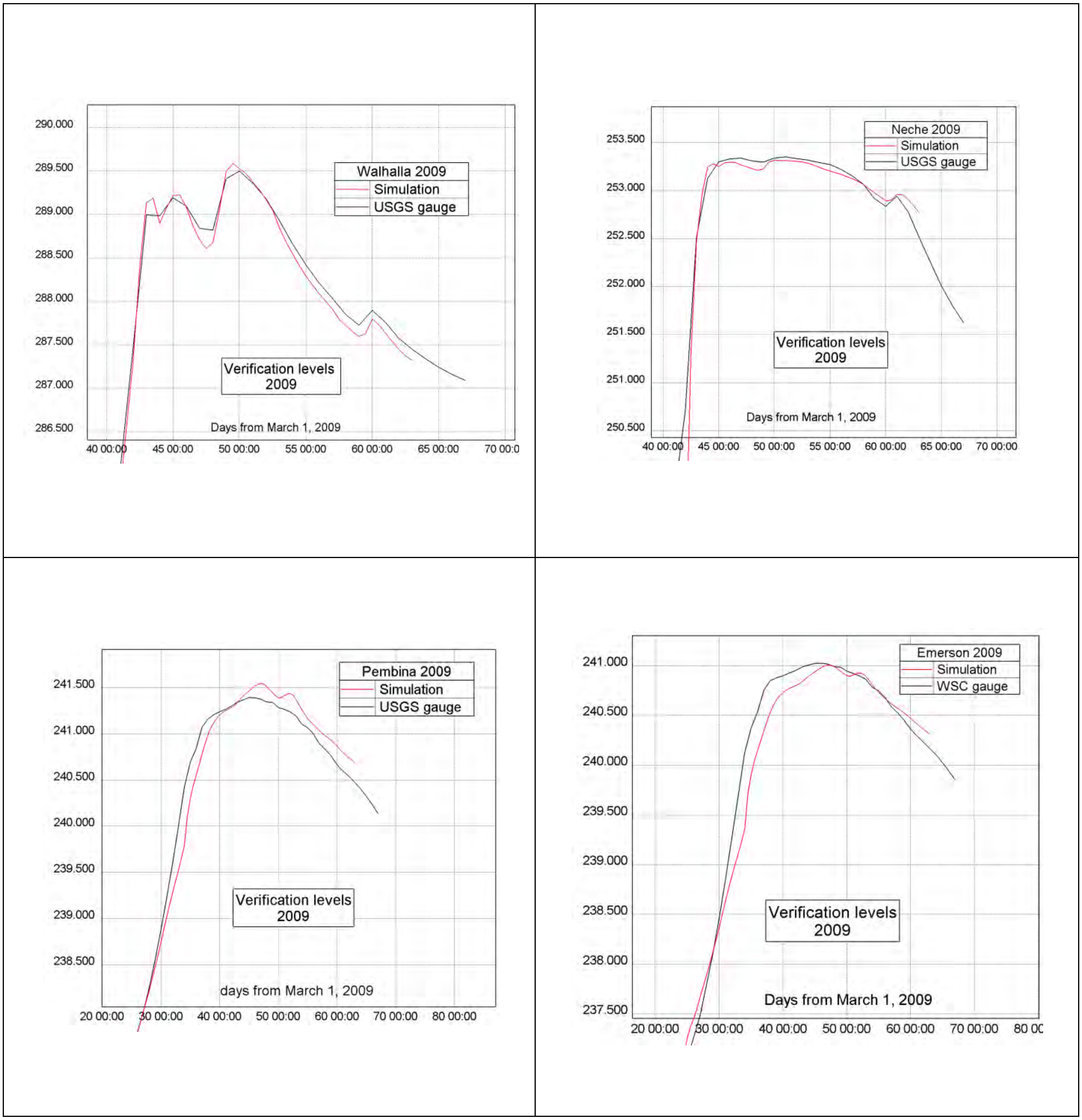


Figure 19 - Verification water levels (in meters) – 2009 flood

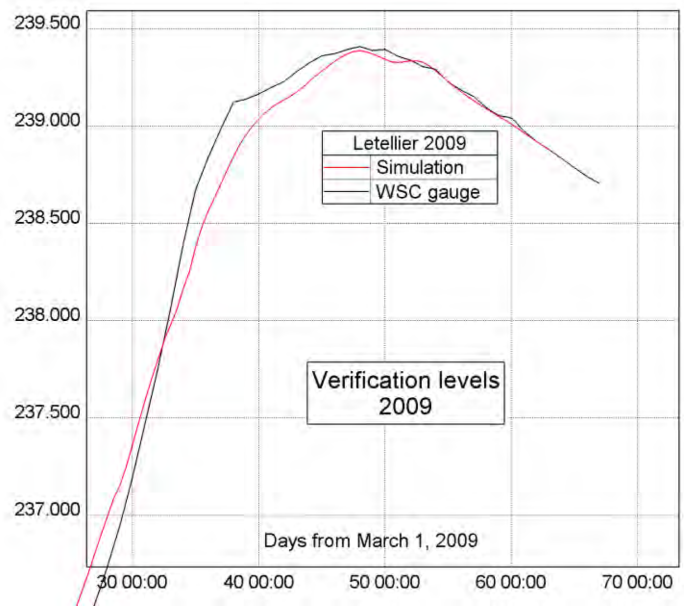


Figure 20 - Verification water levels (metres) at Letellier gauge

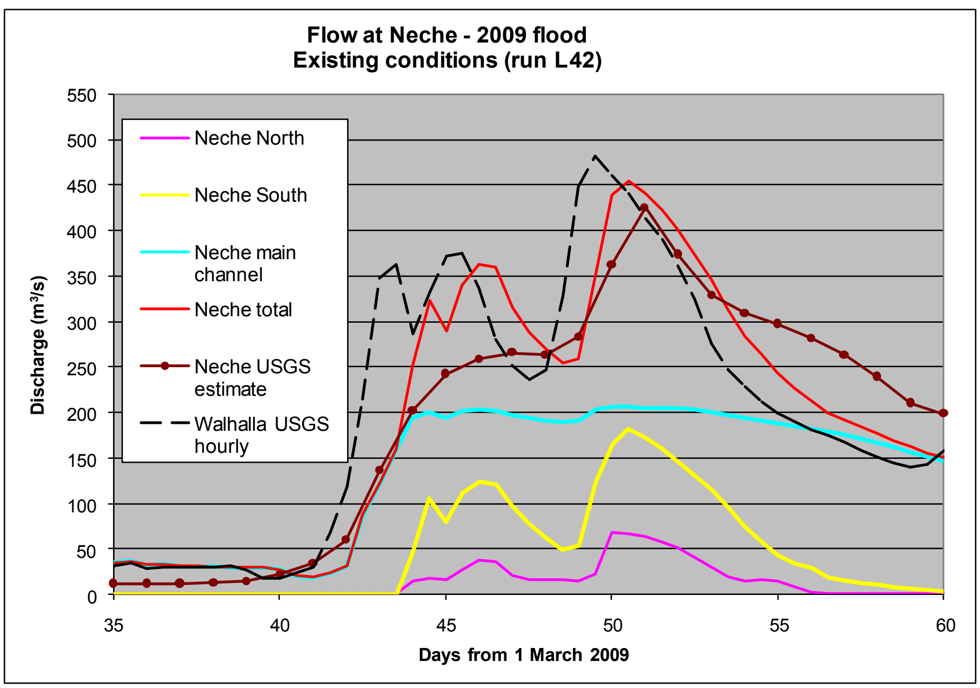


Figure 21 - Discharge at Neche through a north–south section across the whole model—comparison with gauge estimate

Figure 21 shows the comparison between the simulated discharge at Neche and that estimated by USGS during the 2009 spring flood. The two hydrogram peaks at Walhalla are still very strong at Neche due to their long period (five days).

The flows through the Aux Marais River and Buffalo Creek are plotted on Figure 22. The Buffalo Creek discharge was observed to be mostly local runoff as was observed during the 2006 event simulations.

For the Aux Marais River, local runoff was observed until day 36, at which time Border Crossing 6 started to discharge water through the 4 large culverts; and water started to go over Switzer Ridge and headed eastward along the border road. In the model, the border road was overtopped about 5 km east of the ridge, to eventually progress north to the Aux Marais drainage, giving the 26 m^3/s (918 cfs) at the Christie gauge.

Measurements at the Christie gauge show a larger flow (peaking at 34 m^3/s (1,200 cfs)) indicating that the model underestimates flow by 8 m^3/s (283 cfs). It was assumed that this water was crossing the border road from south to north. On Figure 22, the model also shows two peaks originating from the two peaks observed at Walhalla. The WSC gauge has one peak, with the peaks likely being dampened by large storage behind roads.

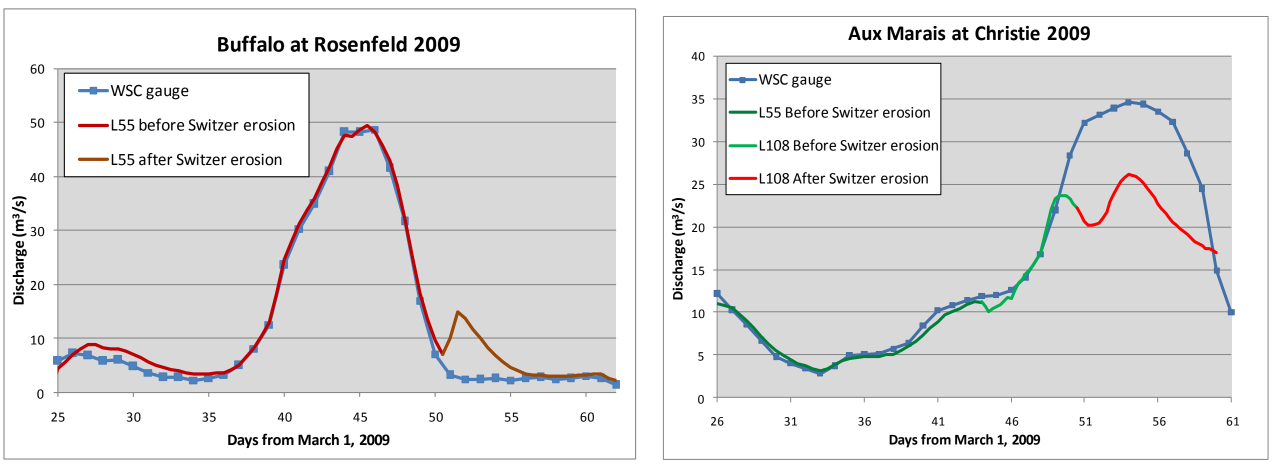


Figure 22 - Flow in Aux Marais River and Buffalo Creek during the 2009 event

Accurately modelling the 2009 event requires an understanding of where the water is coming from and an appropriate landscape representation. A preliminary investigation shows that the water going north into the Aux Marais comes from the flow over Switzer Ridge. This is seen on Figure 23 with the red line representing a streamline from the ridge to Aux Marais, overtopping four roads. The three yellow lines show streamlines going east toward Red River, which was receding on April 22, 2009. A more recent elevation survey of all the roads in this region would help the model in more accurately representing this very complex system.

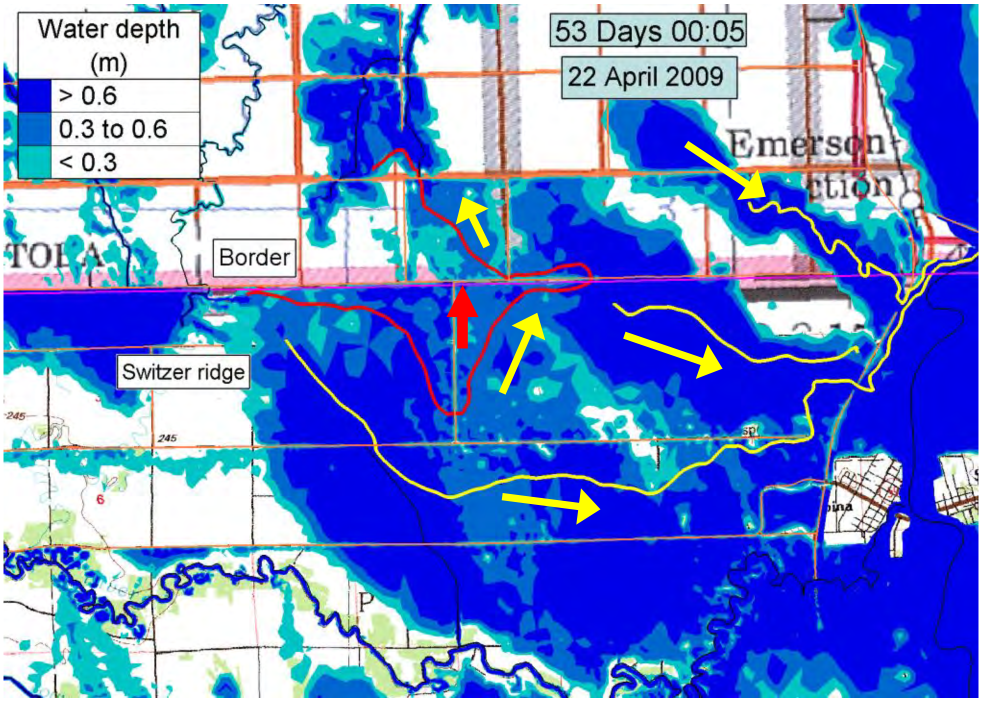


Figure 23 – Detail of streamlines as simulated during the end of the 2009 event, with overtopping towards Aux Marais River

Red arrow indicates overtopping in 2009 (same as Figure 24)
Yellow arrows indicate flow direction

The aerial photos taken from the helicopter on 21 April along the border do not show overtopping of the border road west of Crossing 6, but they show that the border road was overtopped 5.2 km (3.2 mi) east of the Crossing (red arrow on Figure 24). At this location the LiDAR 2006 survey shows a low elevation point that was at the same elevation as the peak water level during the 2009 simulation, and therefore the border road was not overtopped in the model. In an attempt to simulate the overtopping shown on Figure 24, a 20 cm (7.9 inches) erosion at the red arrow was created in the model. The water then did overtop the border road but with only a very small discharge, not large enough to explain the 8 m^3/s discrepancy between measured and model estimated discharge in the Aux Marais River at Christie.

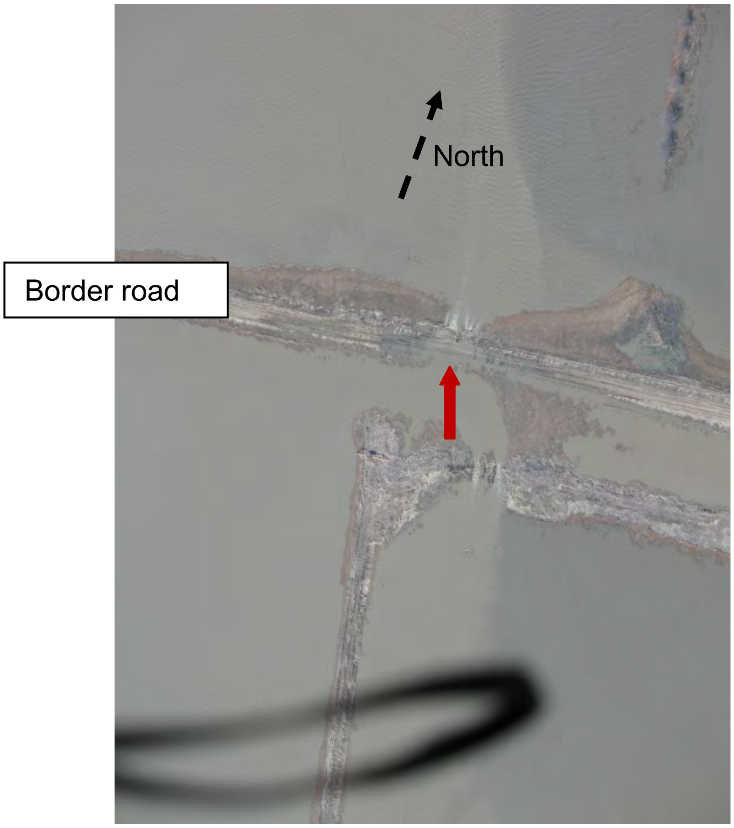


Figure 24 - Road overtopping at W 97 20 03, 21 April 2009 (day 51)

5.2 Comparison with 2009 aerial photos

The calibrated model results were compared with aerial and satellite images to evaluate flood extent modelling performance. Figure 25 and Figure 26 show the progression of the flood over the Red River floodplain compared with RADARSAT images. Note the cloud on these two figures which originate from the Google Earth photographs.

In general, these photos show that the model simulates the actual flood extent very well, and that the water levels are consistent with what was observed during the 2009 flood. In particular the effect of the road 1 mile north of the border is visible inside the red circle on Figure 25. On Figure 26 the correct water elevation provides a very good local flood extent along Aux Marais (red circle), and flooding along Rosebud (green circle) and Tongue River are also quite similar.

Figure 27 and Figure 28 show two photographs taken from a helicopter with the corresponding flooding as simulated by the model.

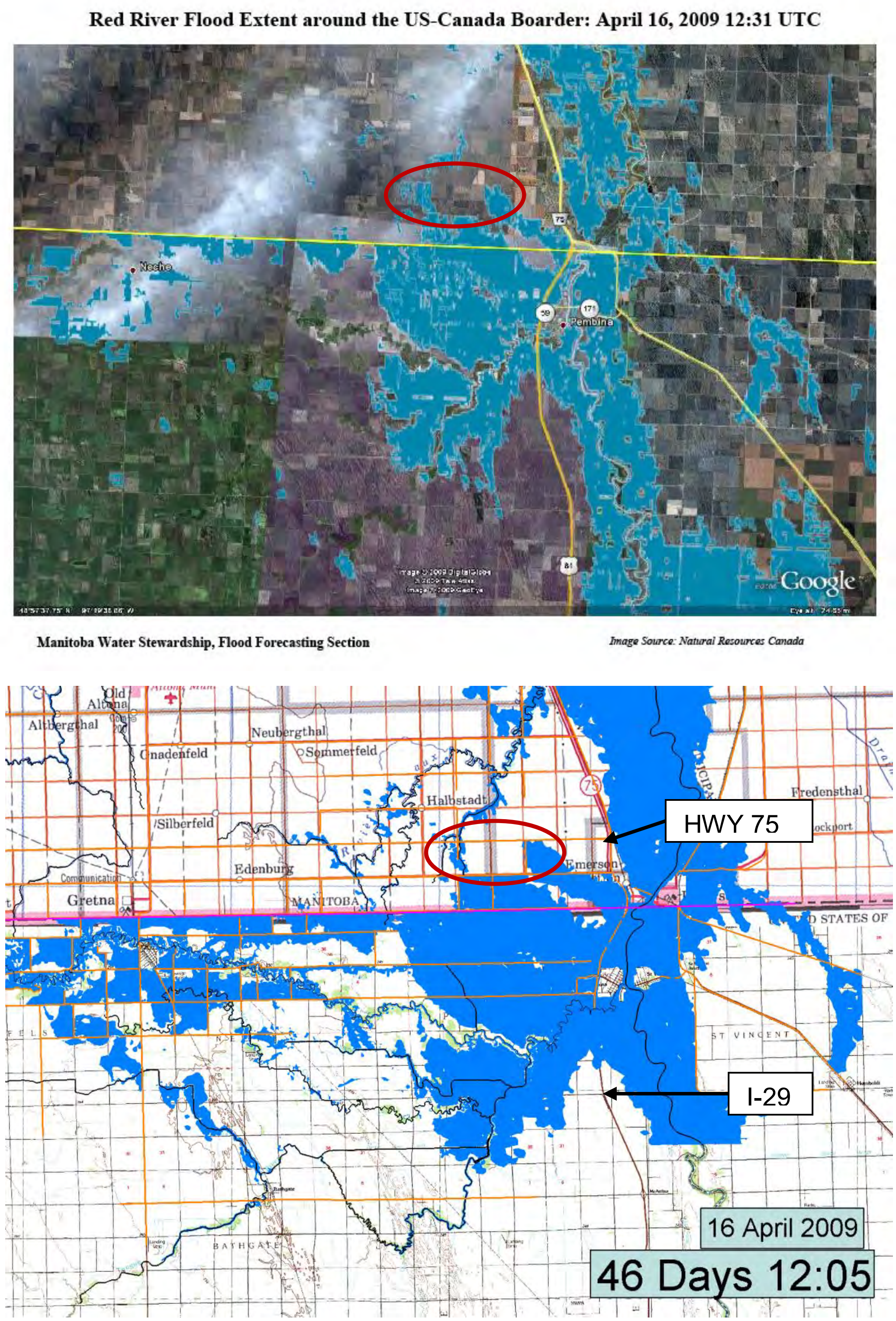


Figure 25 - Flood extent comparison with RADARSAT - 16 April 2009

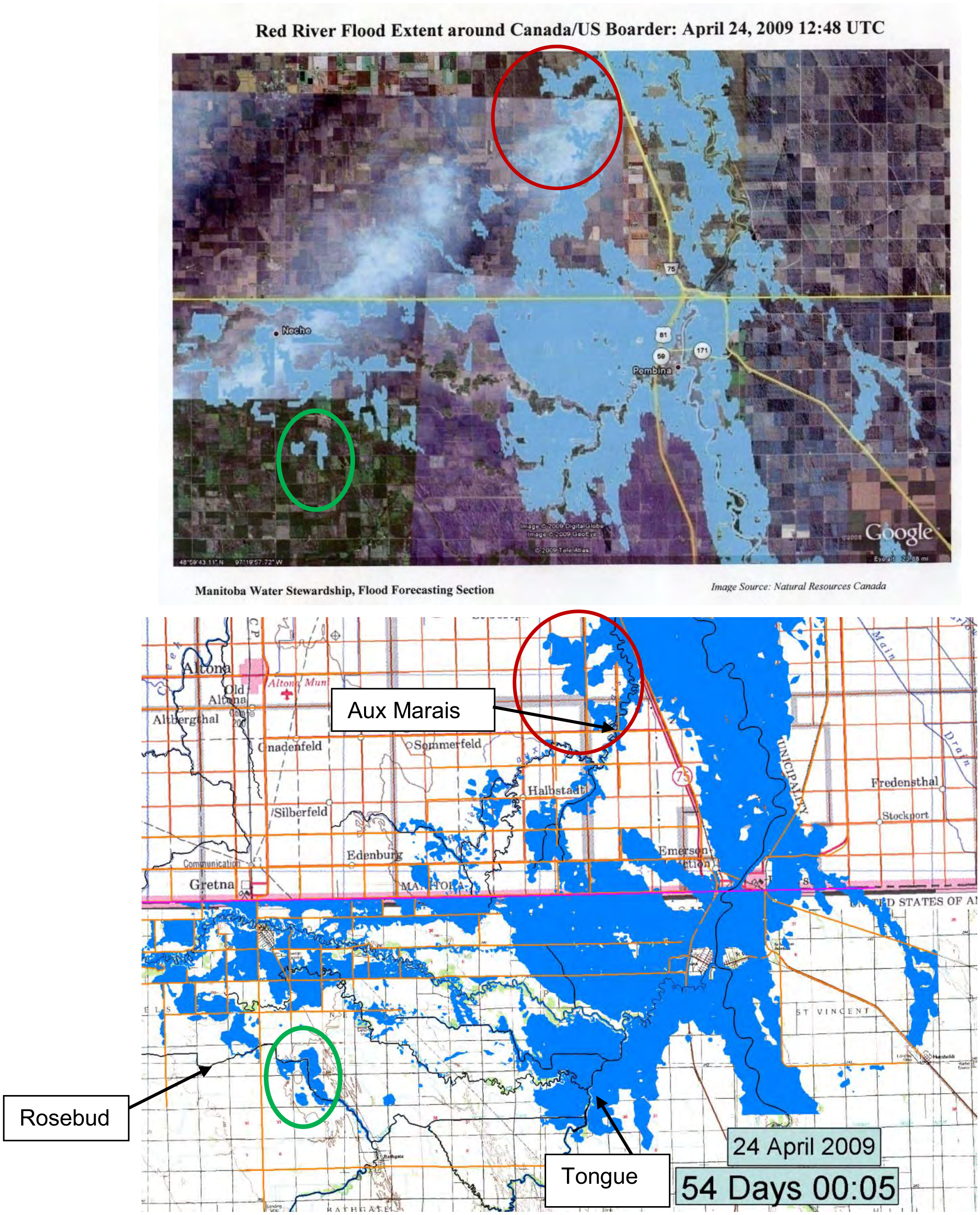


Figure 26 - Flood extent comparison with RADARSAT - 24 April 2009

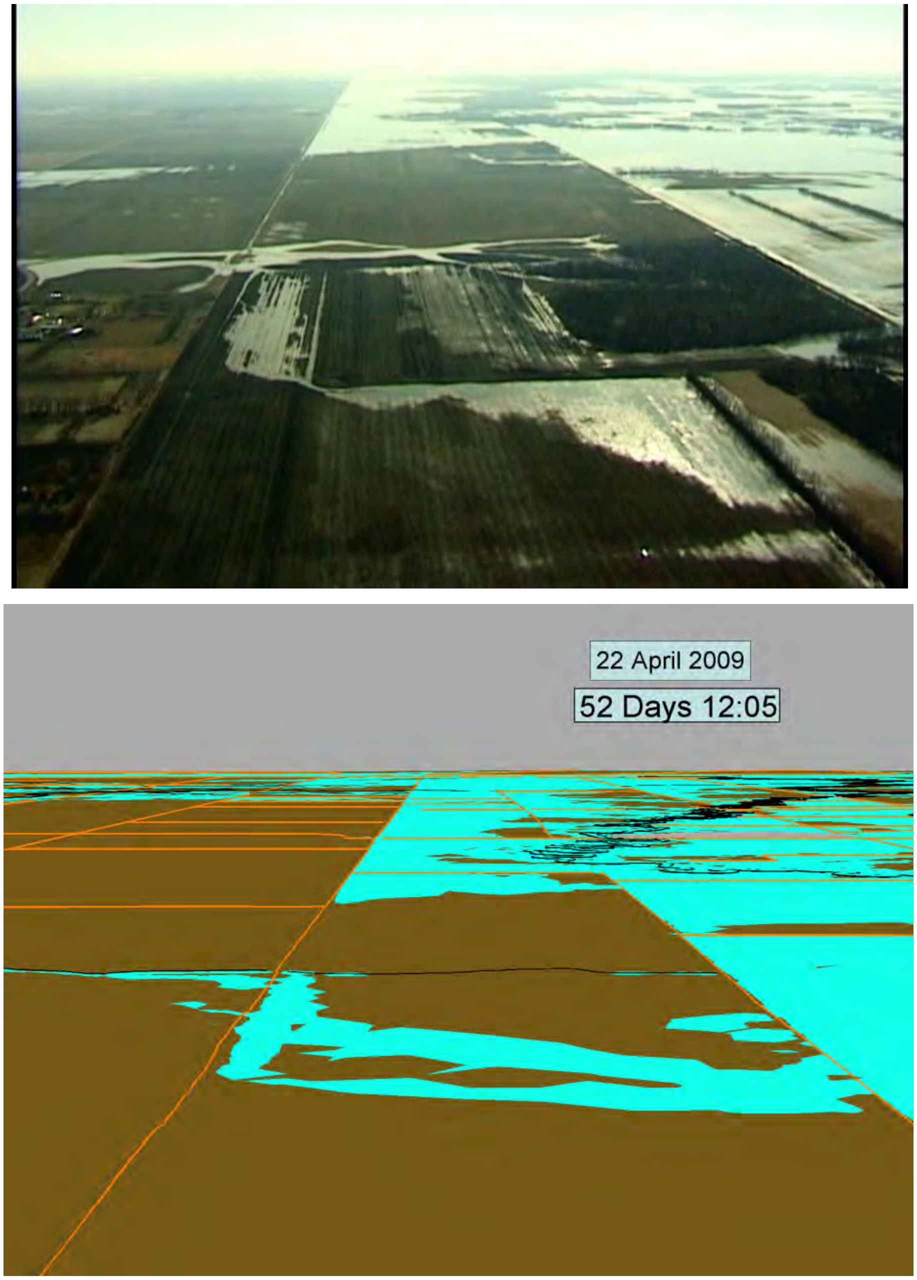


Figure 27 - Flooding around crossing number 5 - 2009 spring flood

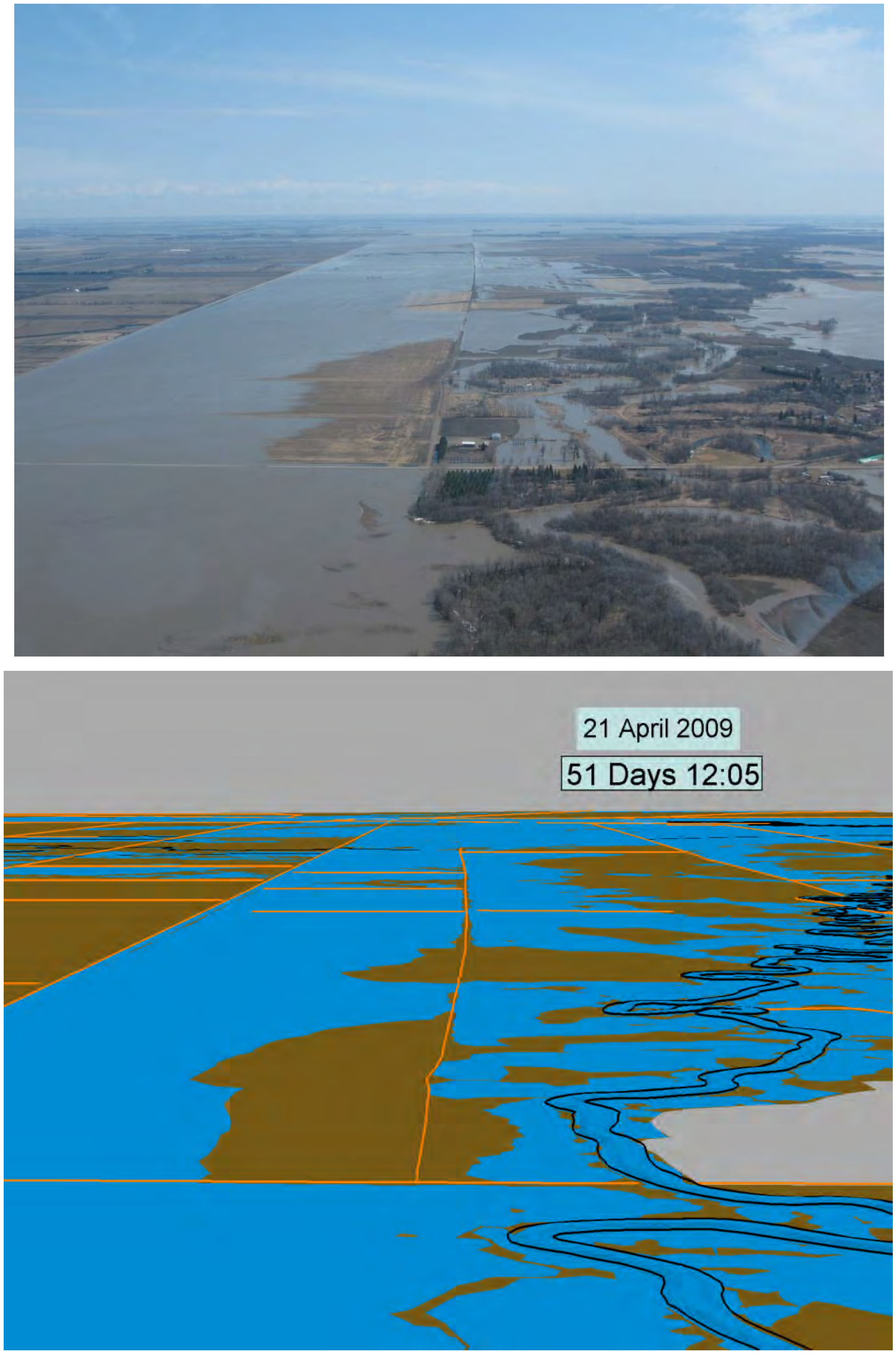


Figure 28 - Flooding east of Hwy 18, north of Neche

5.3 Simulation of 2009 flood without the Switzer ridge erosion

Figure 18 showed the eroded channel that was created during the 2009 event. The effect of this channel was investigated by simulating the same event without the erosion. The changes are shown on Figure 29 and Figure 30 .

After the initial rush through the eroded channel, which lasted a few hours (Figure 29), the total flow across the ridge increased by 8 m^3/s (283 cfs). (This discharge was measured over a north-south cross section between the border road and the first road located ½ mile south of the border).

The level just upstream from the ridge dropped by 2 cm following the erosion (Figure 30), while it dropped only 1 cm on the US side of Crossing 6. These are only minor changes that would not have affected the general behaviour of the 2009 flood.

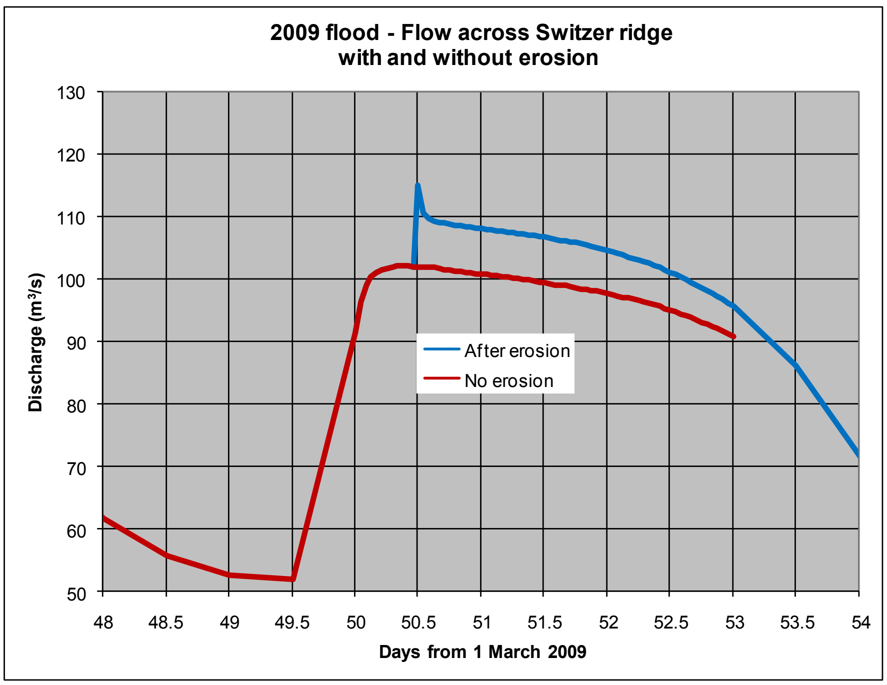


Figure 29 - Flow across Switzer ridge

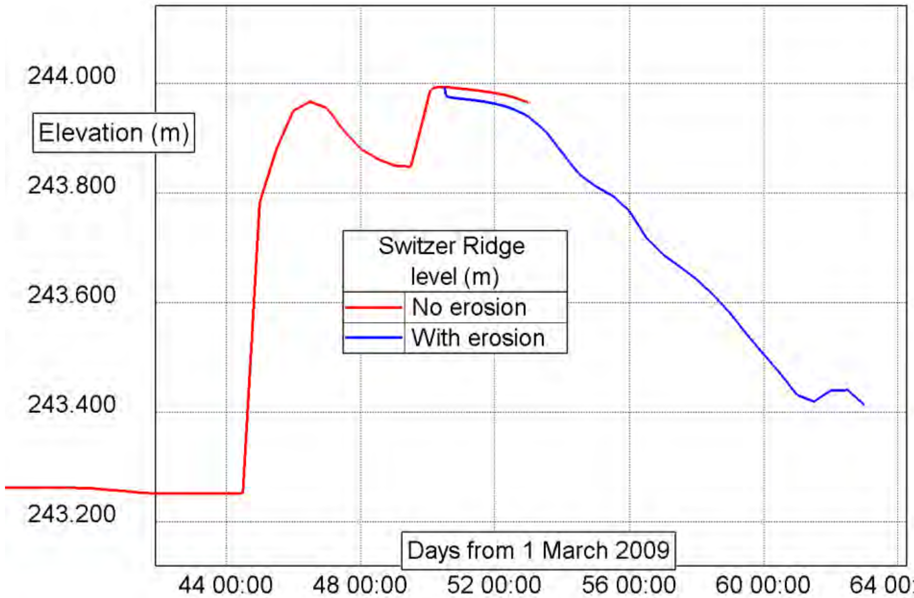


Figure 30 - Levels at Switzer ridge

6. Model Verification – 2011 flood

The model was also verified with the more recent 2011 flood, using the same type of boundary condition as the calibration period (2006 flood):

- Input flow hygrograph at Walhalla (daily data) as estimated by USGS gauge April 2011;
- Input flow hygrograph in the upstream boundary on the Red River (daily average), derived from the WSC gauge at Emerson, minus 90% of the Pembina discharge, which had been shifted by four days. (same derivation as for the 2006 event);
- Water level at the downstream boundary on the Red River, as measured by WSC gauge at Morris; and
- In the new model, runoff for the Aux Marais and the Buffalo Rivers were also included as measured by WSC.

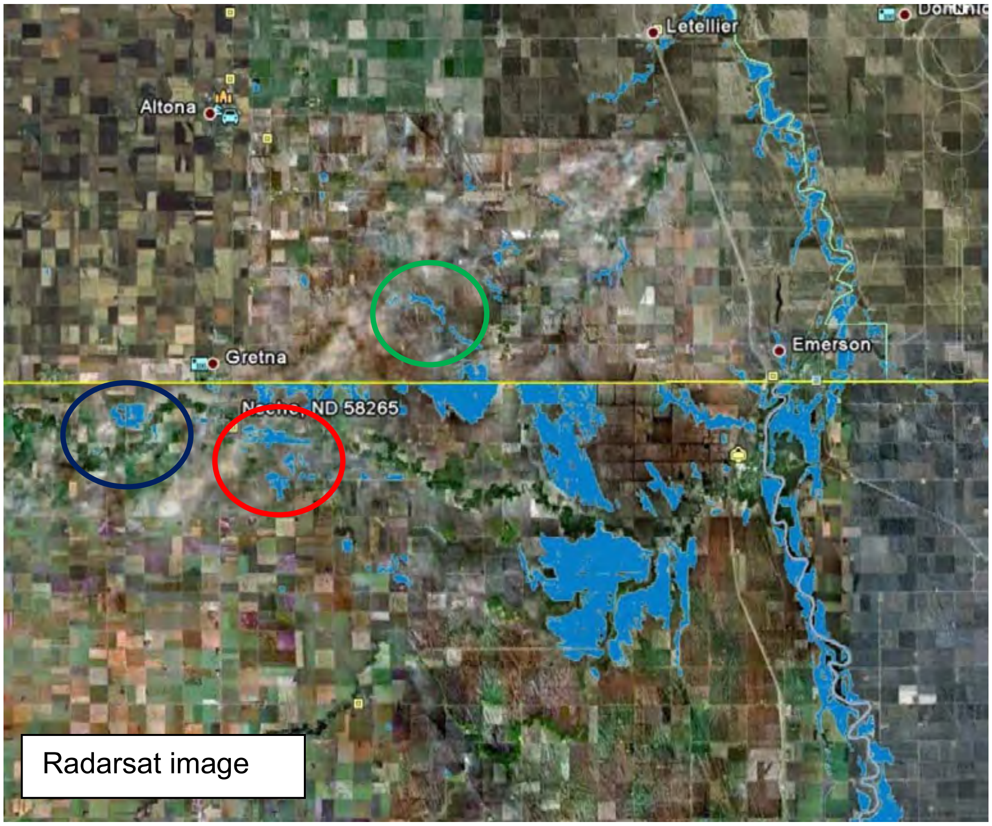
The Switzer Ridge eroded channel was left as it was in 2009.

Figure 31 show a comparison with a Radarsat image taken at the beginning of the event on April 15, 2011.

Flooding along the border and east of Switzer Ridge is very well reproduced, as well as upstream of Aux Marais (green circle), north and south of County Road 55 and north of the road located 1 mile north of CR55 (red circle).

There is a discrepancy inside the blue circle. The image shows flooding only on the north side of the Pembina River, whereas the model shows water accumulation only on the south side. Local runoff (not modelled) or ice may have caused the discrepancy between observed and modelled conditions (USGS record shows ice at Neche until 16 April 2011).

Figure 32 shows a different evaluation; it compares the Telemac inundation map with the Radarsat polygons (red lines) delineating the flood extent for April 22, 2011. The match is very good except on the Red River north of the border, where it is felt that its input flow hydrograph was not correct.



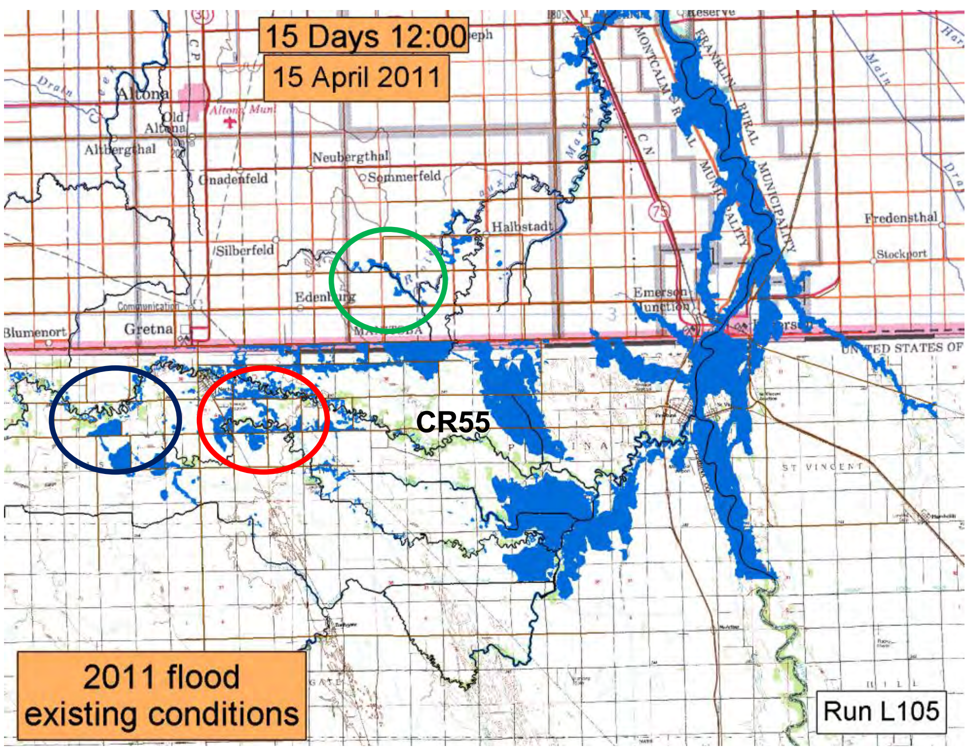


Figure 31 - Flood extent comparison with RADARSAT image - 15 April 2011

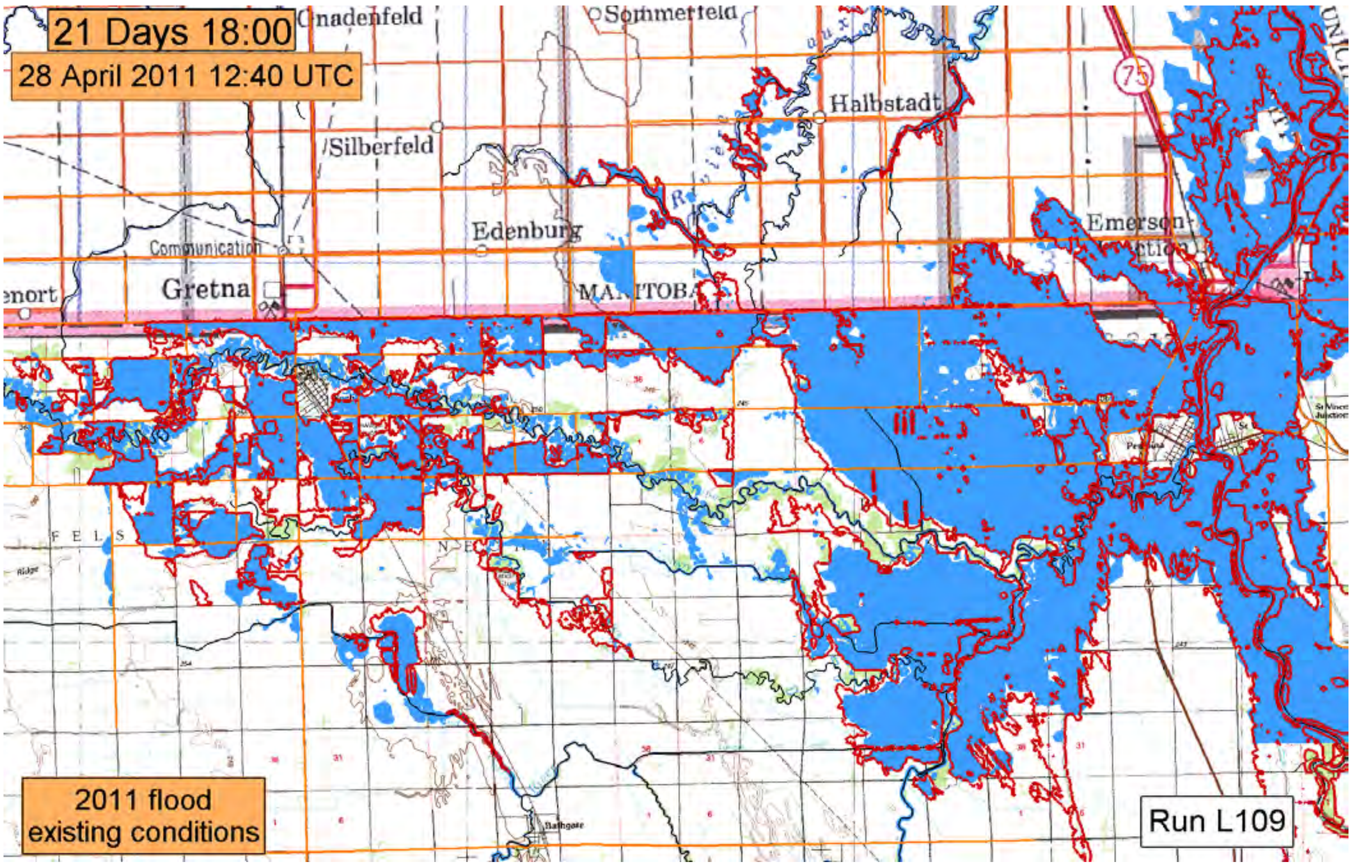


Figure 32 - Flood extent comparison with RADARSAT contour polygons - 22 April 2011

A comparison of levels is shown on Figure 33 and on Figure 34, and it is evident there is good agreement for the Pembina River with 8 cm (3.1 inches) difference with the gauge at Neche. Similar to the 2009 event, the difficulty is with the estimation of the flow on the Red River upstream of the model, in timing as well as volume. This may explain its difference in levels: 20 cm (7.9 inches) at Pembina, 10 cm (3.9 inches) at Emerson. Some of the difference is also due to the fact that the bottom friction was adjusted to match field measurements only at the peak of the 2006 flood, and no attempt was made to adjust the friction during the rising or falling limbs of the event hydrograph.

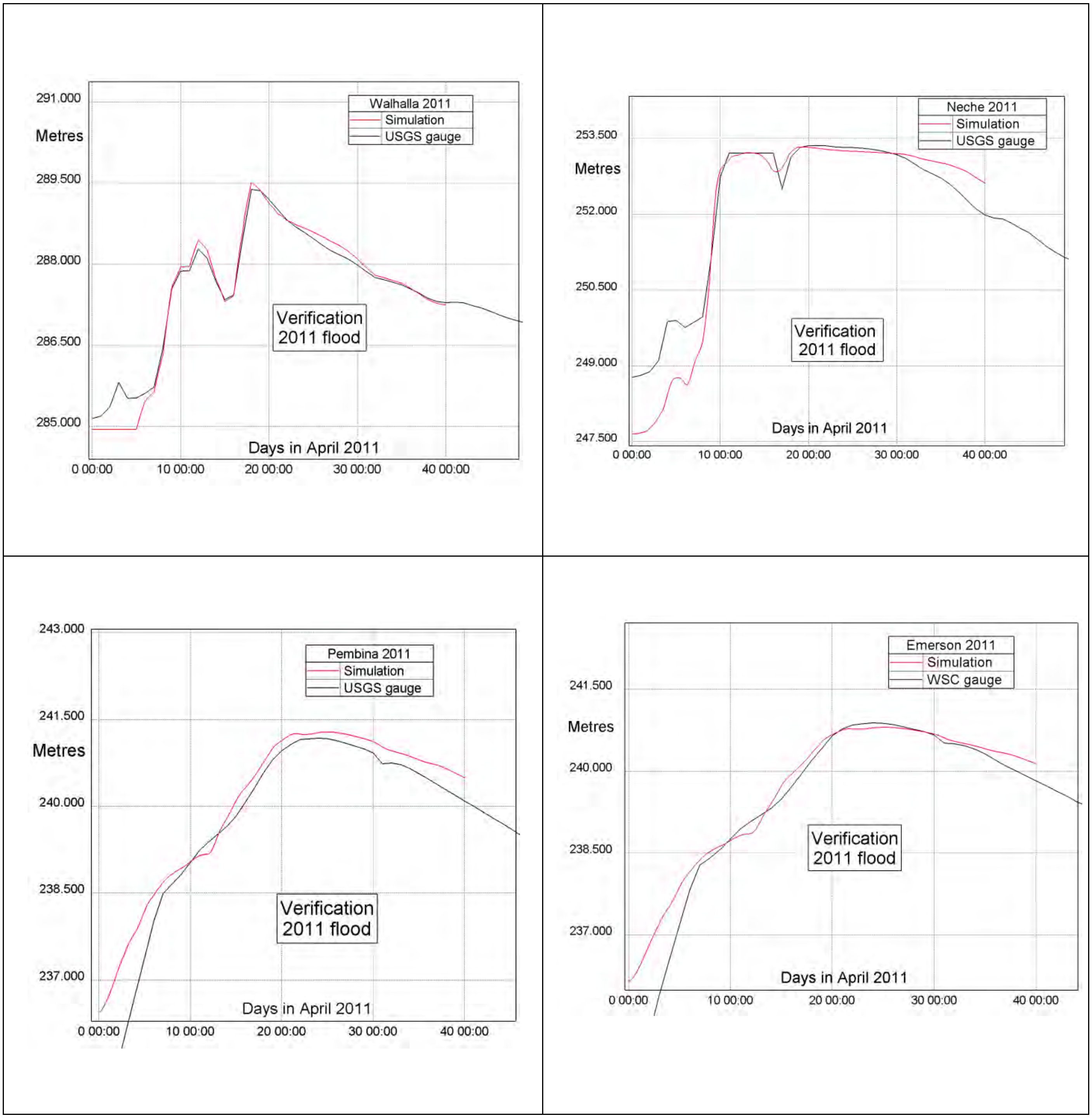


Figure 33 - Verification water levels – 2011 flood

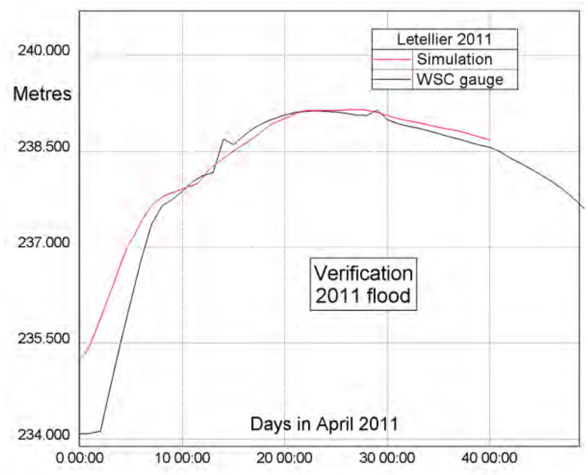


Figure 34 - Verification water levels at Letellier – 2011 flood

6.1 Verification of flows during 2011 flood

Figure 35 shows the comparison between the simulated discharge at Neche and its estimate from the USGS during the 2011 spring flood. The model reproduces very well the gauge measurements during the rising limb of the flood but shows a 1.5 day delay on the recession limb corresponding to the approximate travel time from Walhalla. The difference in peak flow is substantially different. At Neche the response in 2011 is similar to the 2006 and 2009 floods as more water propagates south of the river (150 m^3/s (5300 cfs) peak) compared to the overland flow north of it between Neche and the border (62 m^3/s (2190 cfs) peak).

The flow distribution is different a short distance downstream, similarly to 2006. As seen on Figure 36, the north flow, downstream from Neche, is strongly increased due to the breakouts moving north and then along the border.

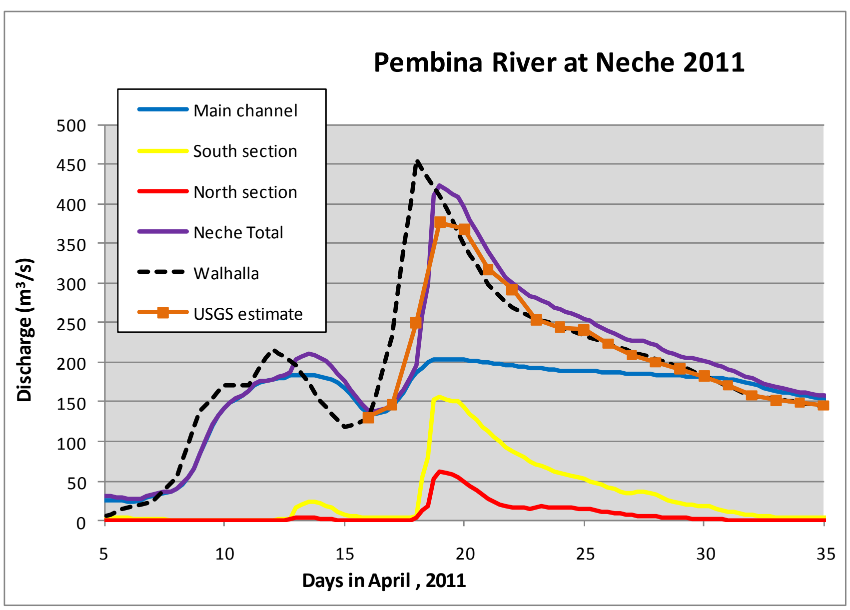


Figure 35 - 2011 event- Discharge at Neche through a north–south section across the whole model domain - comparison with gauge estimate

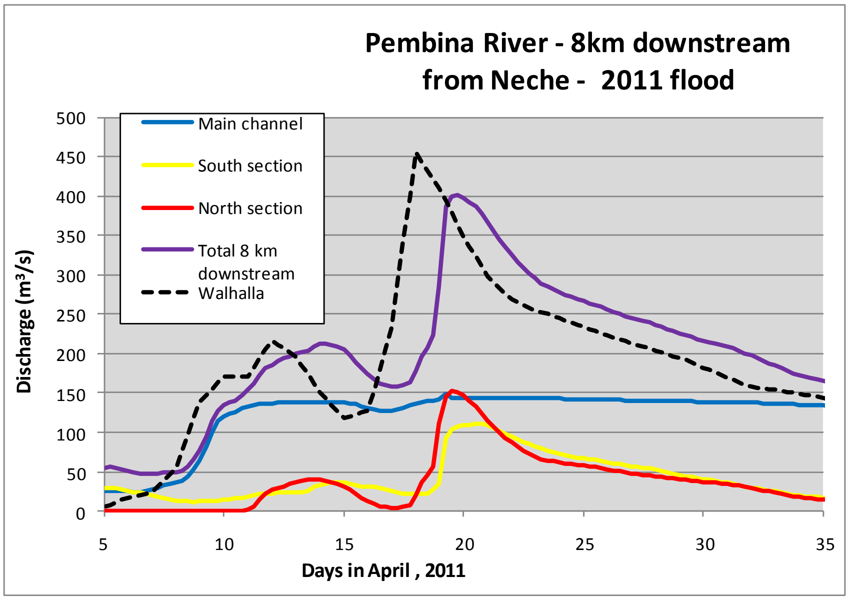


Figure 36 – 2011 event - Discharge through a north–south section 8 km downstream from Neche, across the whole model domain

The flows through the Aux Marais River and Buffalo Creek are plotted on Figure 37. The Buffalo flows are mostly local runoff and are well reproduced by the model.

For Aux Marais, it was local runoff until day 12, at which time Border Crossing 6 started to discharge through the four large culverts, water started to go over Switzer Ridge and eventually progressed north towards the Aux Marais drainage. The same phenomenon occurred in 2009, where the model, again, did not reproduce this propagation over the roads, rendering less flow at the Christie Gauge (13 m^3/s (459 cfs) instead of 28 m^3/s (989 cfs)).

This phenomenon was thoroughly investigated by refining the road definition in all the places where overtopping was felt it should occur (i.e. the low points), and by adding a minor road. The elevations of all the roads were not changed from the 2006 Lidar survey, and we feel that this may be one of the causes of the problem: local erosion.

It is felt that this discrepancy, which results from the overtopping of local roads north of the border, five miles east of Border Crossing 6 (shown on Figure 23) does not affect the results of this study which is looking at changes in flood behaviour, rather than absolute flow propagation.

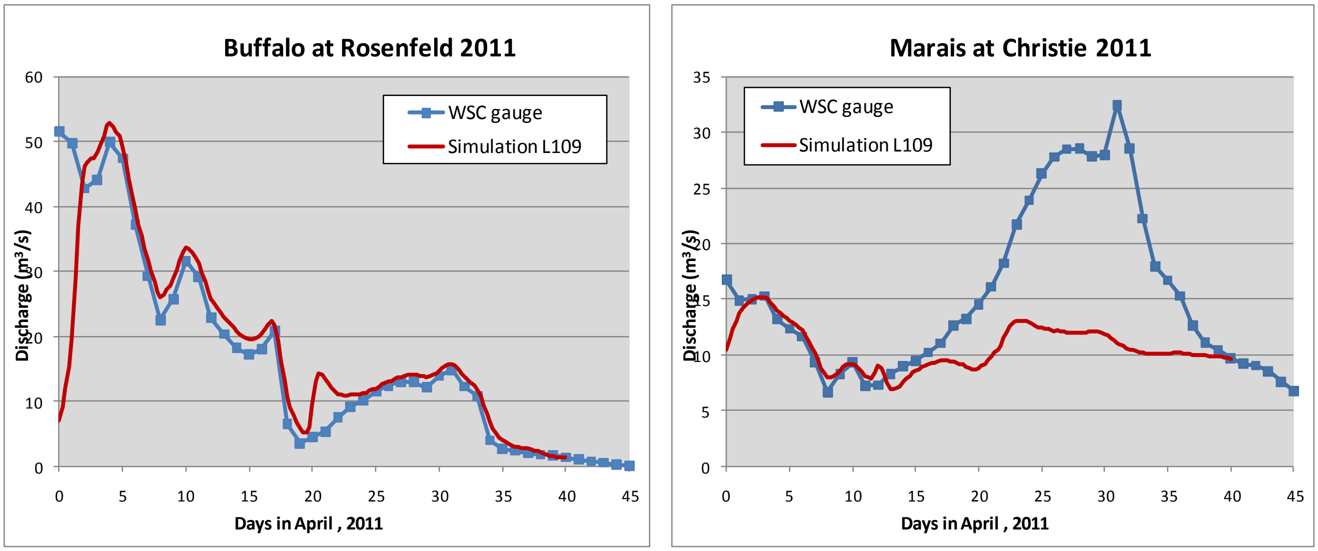


Figure 37 - Flow in Aux Marais and Buffalo River during the 2011 event

7. Red River East overflow - 2006, 2009 events

In addition to some of the major roads and railway embankments east of Pembina and Emerson, the new model included three more bridges under railway tracks at St. Vincent and Emerson, in order to provide a better representation of the landscape for the breakout flows going east and west of Pembina and Emerson.

During the 2006 and 2009 floods, WSC and USGS estimated the east flow breakouts by measuring the flow under the bridges on Hwy 200 and Hwy 171. These measurements are compared to the simulated flows in Tables 4 and 5.

It must be noted that these tables are presented as an indication only. The flow (in volume and timing) at the upstream end of the model was estimated based on the 2006 event. For the 2009 event these flows are not necessarily correct. This was seen on Figure 19 where the levels could be off by 20 cm, causing different bypass flows east or west of Pembina and Emerson.

These tables show that the model closely replicates the measured flows in the Red River's main channel at Pembina. At Emerson, the model overestimates the flow in 2009, on 20 April around the peak of the flood, indicating that the model is not allowing enough flow to break out east or west.

In 2006, before the bridge under Hwy 171 was constructed, a large 8.2 m (27 ft) wide culvert existed under the road, the model had as high as a 65.1 % error, due to he difficulty in estimating flow in a semi-dry large culvert.

In 2009 the model shows insufficient flow east of Pembina and Emerson. This indicates that the bridges under the railway embankments may not be sized properly. We did not attempt to improve these breakouts since the main concern of the project was flooding in Pembina flood plain and along the border. This discrepancy in discharge along the Red River should have negligible effects on the flood simulation in the Pembina flood plain.

Flow at Pembina - 2006									
Date	**Red Main Channel (cfs) USGS**	**Red Main Channel (m3/s) USGS**	**Telemac estimate run L40 (m3/s)**	**Difference %**		**Q East overflow (cfs) USGS**	**Q East overflow (m3/s) USGS**	**Telemac estimate run L40 (m3/s)**	**Difference %**
10/04/2006	62,200	1761	1746	-0.9		2640	75	123	65.1
14/04/2006	62,400	1767	1791	1.4		8460	240	212	-11.7

Table 4 - East flow during 2006 event

Pembina Flow - 2009 event									
Date	**Red main channel Pembina (cfs) USGS**	**Red main channel Pembina (m^3/s) USGS**	**Telemac estimate run L42 (m^3/s)**	**Difference %**		**Q East overflow Pembina (cfs)**	**Q East overflow Pembina (m^3/s)**	**Telemac estimate run L42 (m^3/s)**	**Difference %**
07/04/2009	60,100	1,702	1695	-0.4		13,300	377	279	-25.9
11/04/2009	69,700	1,974	1907	-3.4		16,900	479	380	-20.7
18/04/2009	67,100	1,900	1930	1.6		17,700	501	417	-16.7
Emerson Flow - 2009 event									
		Red main channel Emerson WSC (m^3/s)	**Telemac estimate run L42 (m^3/s)**	**Difference %**			**Q East overflow Emerson WSC (m^3/s)**	**Telemac estimate run L42 (m^3/s)**	**Difference %**
07-Apr		1847	1861	0.8			111	43	-61.0
09-Apr		2081	2064	-0.8			159	100	-37.0
11-Apr		2076	2146	3.4			177	111	-37.2
16-Apr		2145	2428	13.2			203	142	-30.0
22-Apr		2054	2284	11.2			162	126	-22.2

Table 5 - East flow during 2009 event

8. Preparation of Red River and Pembina River Hydrographs with Specific Return Periods

In phase 3 of this project, four different design flood hydrographs were prepared, so that a progression in flood severity - and consequently flood protection - could be envisaged for the various scenarios tested. They were the 1:10, 1:50 and 1:100 year annual flood return periods, and the 1:20 year summer flood. For simplicity they will be referenced in the report as: “1:10 event” etc.

The corresponding design hydrographs for the Red River and the Pembina River were provided to CHC by the USACE for spring events and MWS for summer floods (Appendix 2 and 3). For some gauged and ungauged local channels, a special hydrological study was undertaken by CHC and is described in Section 9 and Appendix 4.

8.1 Red River Hydrographs – Spring events

Manitoba Conservation Water Stewardship had derived the peaks of the floods at Morris for specific return periods, as indicated in Table 6.

Water Level (m)	Discharge (m^3/s)	return period (years)
238.72	3874	100
238.55	3123	50
238.38	2744	33
237.93	2285	20
236.42	1739	10
234.82	1257	5
238.76	4170	1997 flood
238.24	2580	2009 flood
237.40	2151	2006 flood

Table 6 – Peak parameters for various annual design events at Morris; (past floods indicated for reference)

From these design events, the water level curves at Morris were derived by assuming they would have the same shape as the 2006 flood. These curves are shown on Figure 38. They were used as the model downstream boundary conditions.

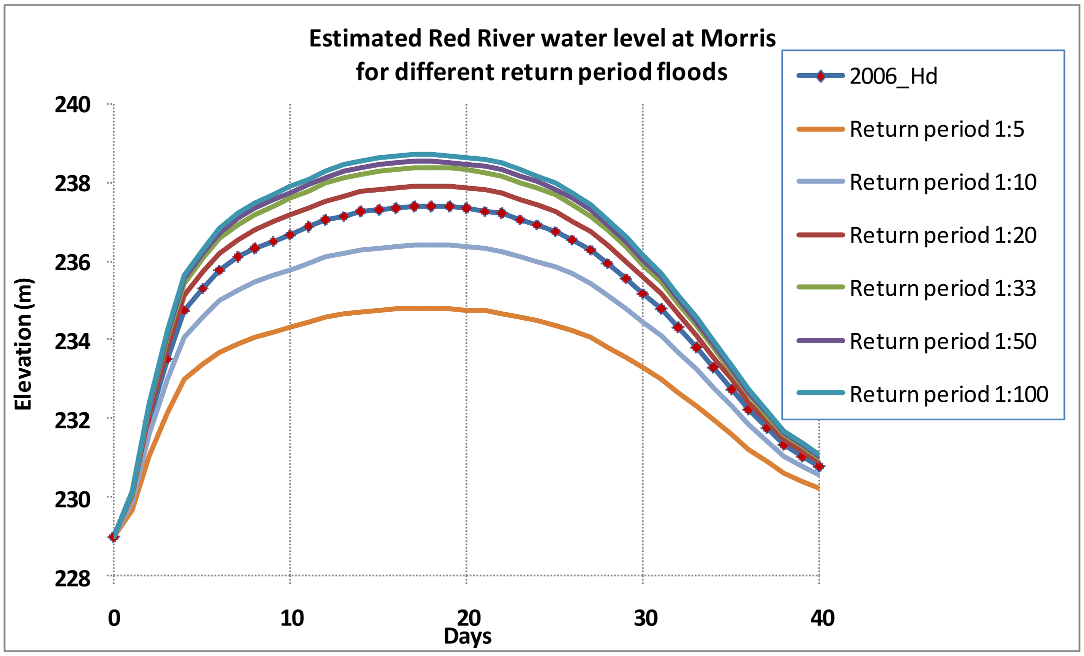


Figure 38 - Water level graph at Morris on the Red River

In this modelling effort, it was necessary to prepare the Red River hydrographs, at its upstream boundary. This was not an easy task since the only information available was the data from Table 6, downstream at Morris, and the hydrographs at Morris and at Emerson for the major spring events.

The derivation of the Red River hydrographs at the upstream boundary for the return periods was prepared in several steps:

The model had been run for the 2006, 2009 and 1997 events, which required the estimation of hydrographs at the upstream boundary. This estimate was based on an assumed time shift between the peak at Walhalla and the peak at Emerson, and the shape of the hydrographs at Emerson. A relationship between the peak at both boundaries (downstream at Morris and upstream) was therefore possible for these 3 events. This relationship is shown as the blue square on Figure 39.

Through these blue squares, the black line was drawn and extended toward small flows.

Using this relationship, from the known downstream peaks at Morris (Table 6), an estimate of the upstream peaks was calculated, (round red circles for the 100 year, 50 year, 33 year and 20 year, and red triangles for the 10 year and 5 year).

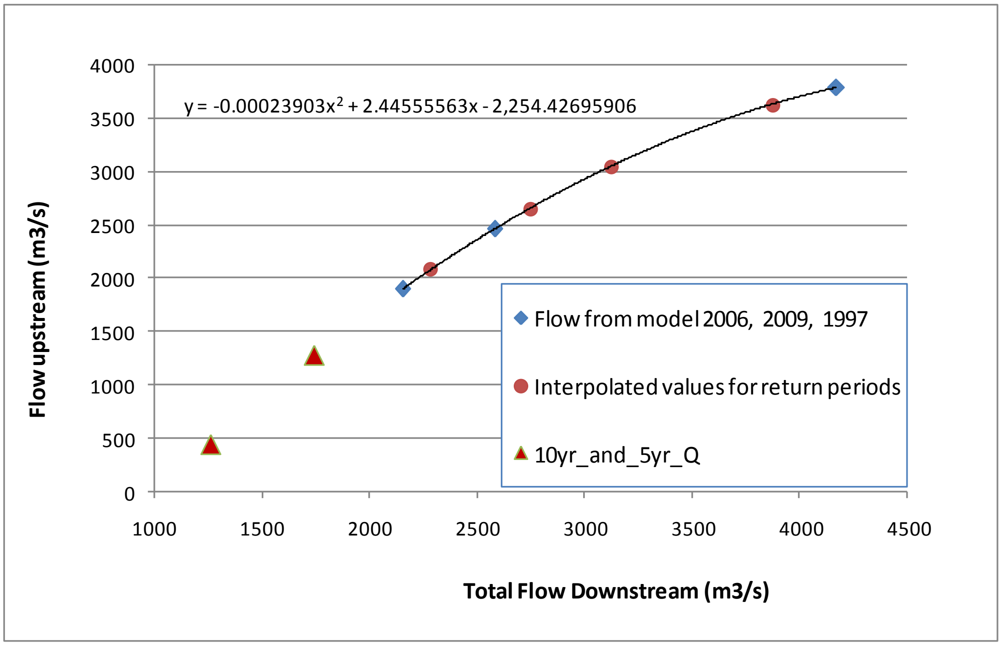


Figure 39 – Relationship between upstream and downstream peak flows on Red River

Using the shape of the modelled 2006 discharge curve, the upstream discharge hydrographs for the various return periods were prepared. These estimates are shown on Figure 40.

The downstream discharge at Morris is the combination of the upstream flow and all the flows from the many tributaries: Pembina, Roseau, DeadHorse, Aux Marais, Buffalo, Tongue, Louden and Rosebud. These rivers did not necessarily behave the same way during the 2006 event and events with specific return periods. Therefore injecting the upstream Red River flows from Figure 40 and the individual tributaries flows did not produce the target peak flows at Morris shown on Table 6. An iteration process was required where the upstream flow was adjusted until the discharge at Morris was close to the target. Table 7 shows the final discharges at the upstream and downstream (Morris) ends of the Model.

Target and model peak flows at Morris are very close because the modelling system was designed as such. The large differences in model peak flows upstream on the Red River come from the fact that all local tributaries were also assumed to exhibits hydrographs having the same return period as the Red and the Pembina, which was not the case is 2006.

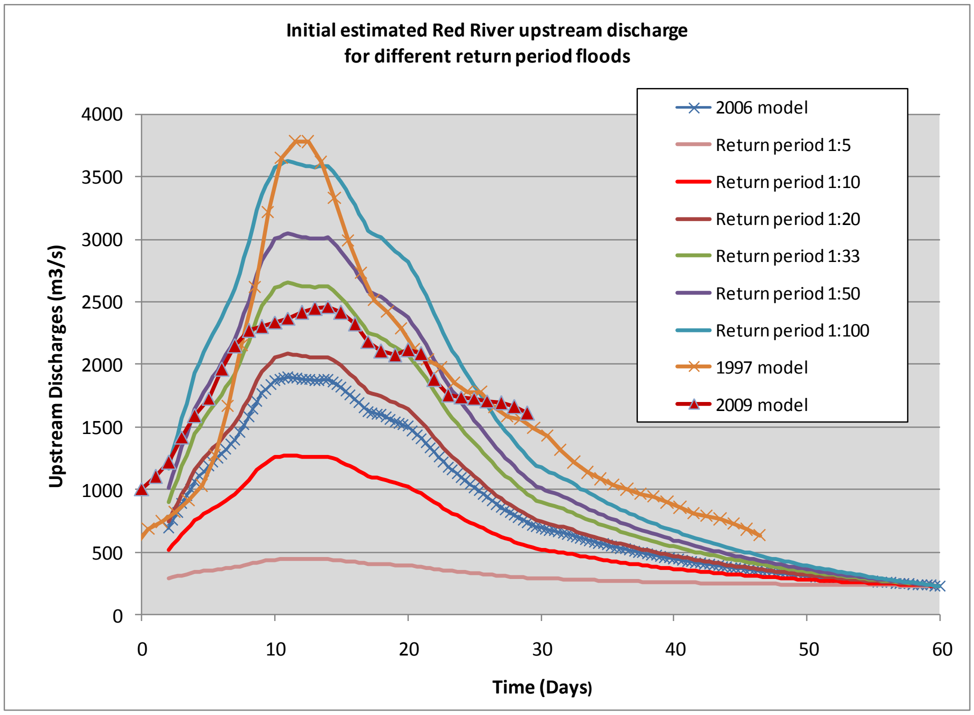


Figure 40 – Shape of target Red River upstream discharge curve, as derived from the 2006 event

Return period (years)	Model peak flow upstream (m^3/s)	Target peak flow upstream (m^3/s)	Model peak flow downstream (m^3/s)	Target peak flow downstream at Morris (m^3/s)
1:10	1347	1275	1733	1739
1:50	2381	3052	3098	3123
1:100	2909	3632	3847	3874

Table 7 – Upstream and downstream peak flows in the Red River

The timing of the peak flow at Emerson was derived from an analysis of the last 60 years of historical records. On average the Red River at Emerson peaked 6 days after the Pembina at Walhalla, based on 34 events recorded during March-April.

8.2 Red River Hydrographs - Summer events

During spring events, Red River and Pembina River overflow their banks during the same time period and the Red always enters the Pembina River channel, moving upstream.

For a summer flood, this may not be the same, as a strong localized rainfall may affect the Pembina, while the Red is still flowing with its normal flow. It was then important to simulate an event where the local flooding in the Pembina is resulting from local runoff, and not the Red River. To model this, a constant upstream flow of 1000 m^3/s (35 314 cfs) was chosen on the Red River, with a constant level at Morris set at 236.3 m (775.25 ft), a flood stage that is slightly lower than that caused by the 1:10 event.

8.3 Pembina River Hydrographs – Spring and Summer events

The hydrographs for the spring floods were prepared by USACE and are shown on Figure 41. Their shapes were based on the 2006 curve at Walhalla.

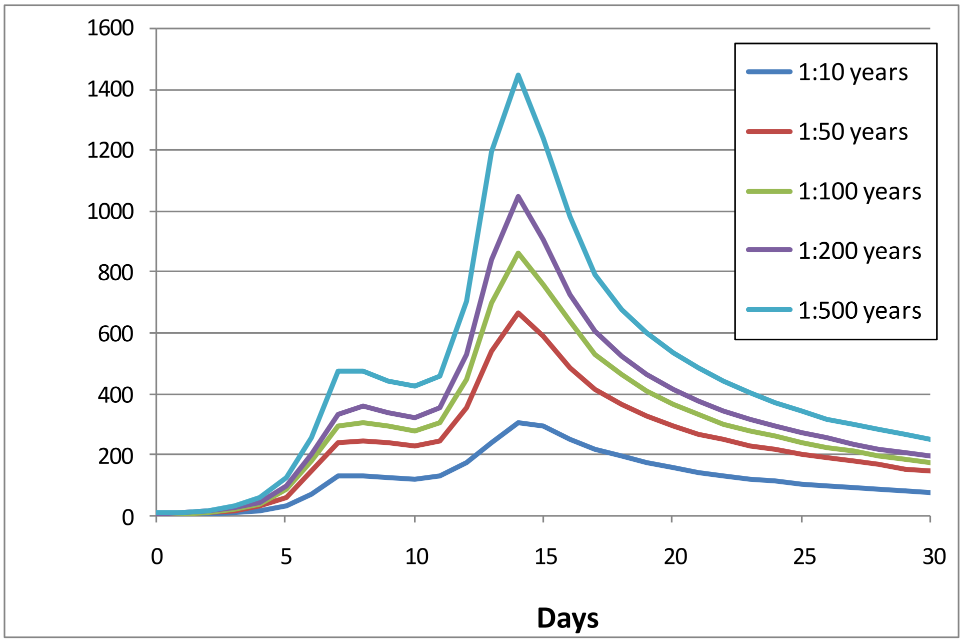


Figure 41 - Spring hydrographs for Pembina River at Walhalla

The hydrographs for the summer floods are much shorter, lasting only a week instead of a full month. They were prepared by Manitoba Conservation Water Stewardship and are shown on Figure 42.The 1:5 and 1:10 year summer design floods do not cause the Pembina to overtop its banks. The 1:20 year return period was chosen as the “summer event” to establish the degree of flooding that occurs along the Pembina River. The derivation of these hydrographs is outlined in Appendix 2 and 3 at the end of the report.

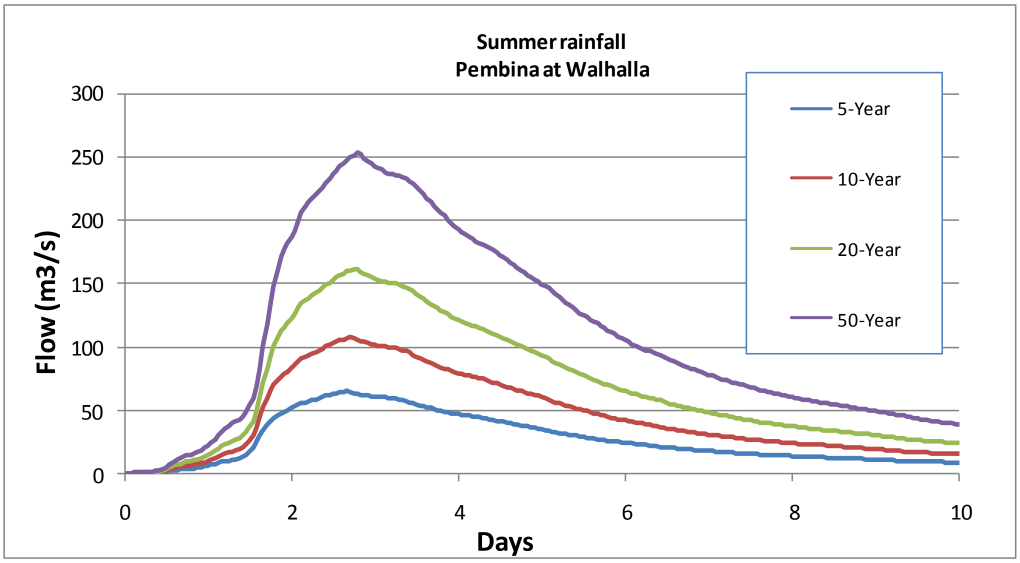


Figure 42 - Summer hydrographs for Pembina River at Walhalla

9. Hydrological Analysis

The hydrologic study was conducted to estimate the flood hydrographs for input into the Pembina River hydrodynamic model. A list of gauged stations were provided for analysis, both within the US managed by the USGS and within Canada manage by Environment Canada and the Water Survey (WSC). A list of the stations identified and analyzed is included in Table 8 along with location, upstream drainage area, the available period of record and the number of summer events identified (see below for analysis of summer events). A map of the gauges locations and the mapped upstream areas is shown in Figure 43, below.

	Agency	Station Code	Latitude	Longitude	Description	Drainage Area (km2)	Record Length (yrs)	Years with Summer Events
1	USGS	5101000	48.778	-97.746	Tongue at Akra	414	47	40
2	USGS	5100000	48.989	-97.551	Pembina at Neche	8832	59	56
3	USGS	5099600		-97.917	Pembina at Walhalla	8676	42	40
4	USGS	5099400	48.865	-98.006	Little South Pembina	471	33	29
5	WSC	5OB007	49.031	-98.272	Pembina at Windygates	7500	20	19
6	WSC	5OC016	49.251	-97.550	Deadhorse creek	926	32	30
7	WSC	5OC019	49.191	-97.403	Buffalo at Rosenfeld	927	28	25
8	WSC	5OC022	49.075	-97.305	Aux marais at Christie	195	29	20
9	WSC	5OD001	49.191	-96.985	Roseau River Near Dominion City	5020	61	61

Table 8 - Hydrometric Gauge Information

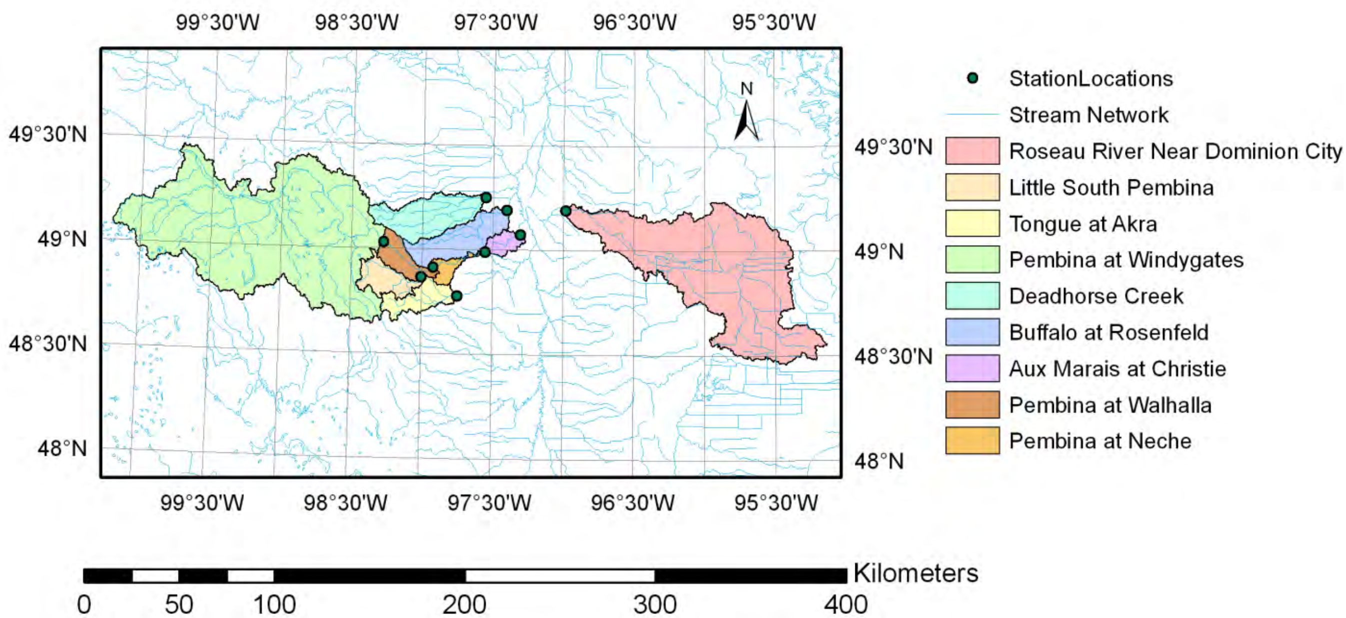


Figure 43 - Hydrometric Station Locations and Drainage Areas

A number of ungauged drainage areas were identified as possible significant contributors to flow the Pembina River hydrodynamic model. These areas represent incremental drainage areas within the model domain that will contribute significantly to the Pembina stream flow and flooding. A list of the identified areas along with the drainage areas are shown in Table 9. The drainage areas were calculated by delineating the upstream areas using the National Elevation Data (NED) digital elevation model and the Green Kenue hydrological software tool (NRC-CHC 2010). A map of the contributing drainage areas is shown in Figure 44.

Description	**Drainage Area (km2)**
Louden Coulee	57
Rosebud	266
Bathgate to Akra	64
Pembina River - Walhalla to Neche	167
Pembina River - Neche to Pembina	80
Tongue River - Louden to Pembina River	95

Table 9 - Ungauged Drainage Areas

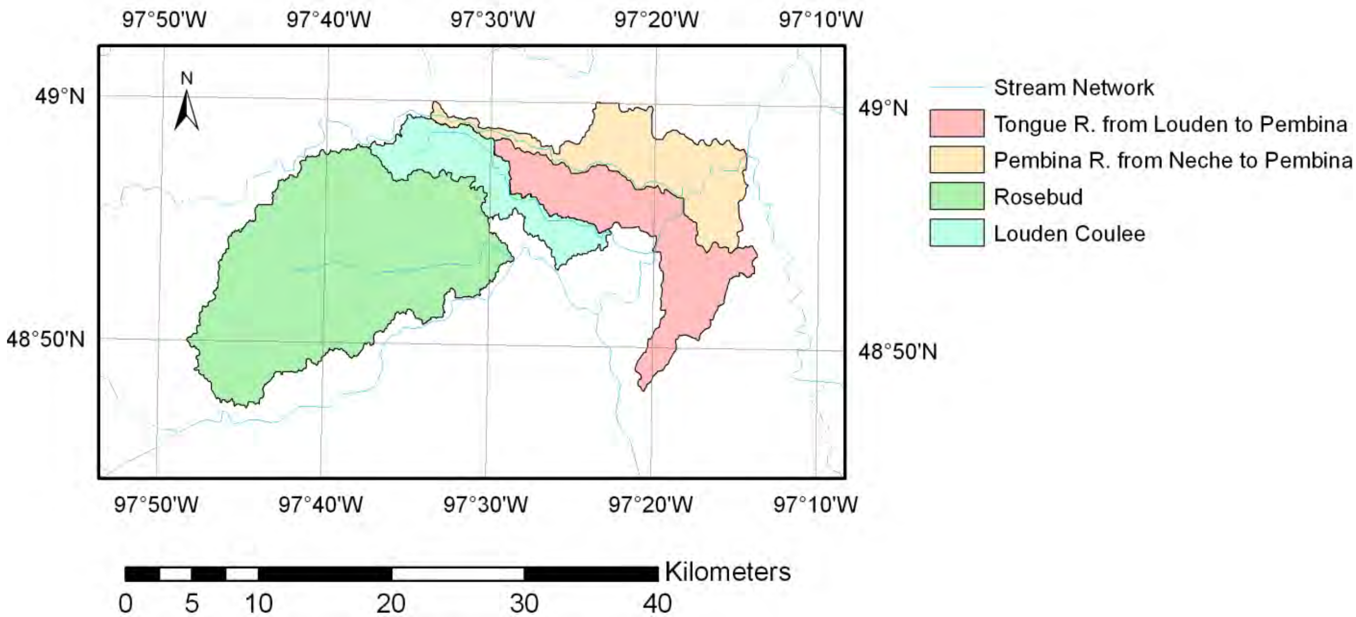


Figure 44 - Pembina River ungauged areas

This section outlines the approaches taken to calculate the design floods and timing. The detailed results for each station and ungauged basin are located in Appendix 4.

9.1 Flood Frequency Analysis

Daily data was collected for each of the stations from 1960 to 2010 as available from USGS and EC sources. Maximum annual daily flows were extracted and extreme value distributions were applied to the datasets using L-Moment techniques (Hosking, 2012). Probability distributions considered included GEV, Log Normal (LN3), Person Type III (PE3), and Log Person III (LP3). It was originally found that the GEV distribution provided the best fit (by visual inspection) most consistently for the stations analysed. However, it was later indicated by the IJC boards that the Bulletin 17B methodology (USGS 1981) should be used to be in compliance with USACE practices. Consequently the GEV distribution was abandoned in favour of the Bulletin 17B (B17) approach for all stations. Where appropriate we used the regional skewness of -0.4 which seems to best represent the region as identified by Panel 1 of the bulletin. We found some slight differences between the Bulletin 17 approach and the LP3 fitting employed using L-moments.

Flood frequency analysis using the B17 methodology was employed for each station for the recorded peak flows and the average 1-day, 3-day, 5-day, 7-day, 11-day, 15-day, 21-day and 31-day flows. This number of ordinals was required to properly balance the shape of the hydrograph. For each station four return intervals were considered: 1:100, 1:50; 1:20 and 1:10. And for each station two periods were considered: annual and summer. Summer was defined as May to September. In total 18 B17 flow-frequency analyses were conducted for each station to develop complete flow-duration-frequency curves as required.

9.2 Flow-Duration-Frequency (QdF) Balanced Hydrograph Methodology

The balanced hydrograph methodology was determined by the IJC boards as the most appropriate method to generate the design hydrographs and was a methodology adopted by the USACE. The involved determining the return frequency for a number of peak and averaged flow periods using a flow-frequency analysis, thus producing a flow-duration-frequency (QdF) curve. The design hydrograph for a particular return period should match the corresponding line in the QdF curve. An example of a series of design QdF curves and the matching QdF curves for a series of design hydrographs is illustrated in Figure 45.

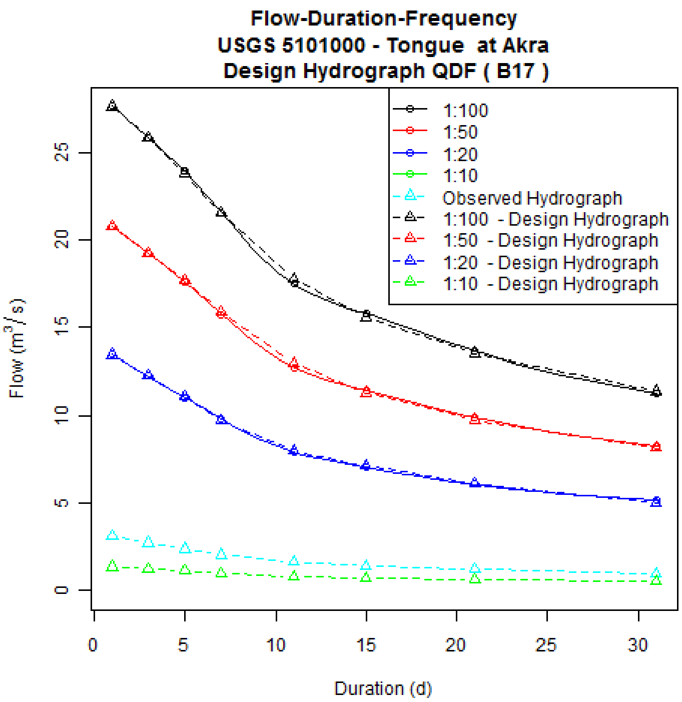


Figure 45 - QdF Curves, Tongue at Akra

The identified USACE methodology is to develop the QdF curves using the B17 flow-frequency analysis, identify a representative event and then run the event through a program such as HEC-1 or HyBART (Hydrograph Balancing and Reporting Tool, David Ford Consulting Engineers, 2010). In advance of running HyBART the QdF curves were manually adjusted to ensure they were monotonically ascending with reduced frequency. The resulting hydrographs will be adjusted to match the QdF curves but retain, in general the hydrograph shape.

The HyBART product showed some issues when applied to the design hydrographs including a jagged, unrealistic hydrograph output. It was explained that usually these jagged responses are edited manually to produce a smooth curve (Dan Reinhartz, USACE, personal communication). The HyBART code is compiled and unavailable for review (although the manual states that the HEC-1 routines have been implemented), but it is suspected that the flow matching routine adjusts segments of the hydrograph independently resulting in unrealistically jagged hydrograph response. In order to reduce the subjective and time-consuming process of manually editing the hydrographs, NRC developed a scaling algorithm that took the original hydrograph and scaled it in magnitude and duration to best match the QdF using a least-squares approach.

A comparison of the two approaches is shown in Figure 46, with the HyBART hydrograph and the scaled hydrograph identified. The result of the scaled hydrograph is a realistic hydrograph not requiring manual adjustment. The compromise is a somewhat reduced fit to the QdF matching (see Figure 47) although the manual adjustment of the hydrographs using the HyBART approach are expected cause deviations from the QdF matching as well.

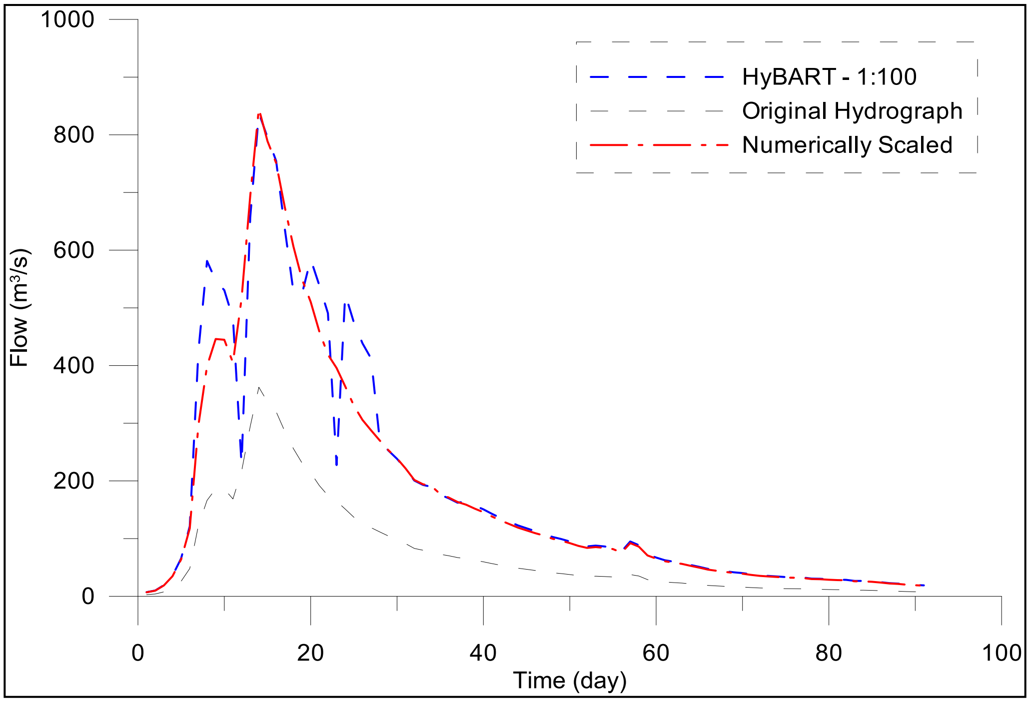


Figure 46 - USGS 5099600, Design Hydrograph Comparison

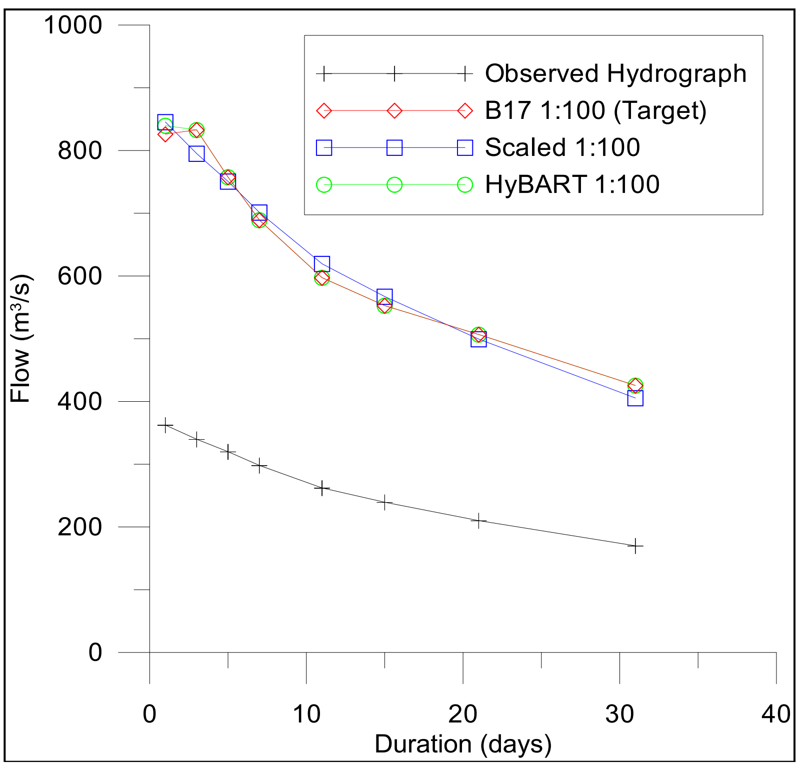


Figure 47 - USGS 5099600, Design Hydrograph QdF Comparison

The optimization routine in scaling algorithm did sometimes shift the hydrograph peak earlier or later than the original hydrograph in an attempt to provide the optimum fit with the QdF curves, but as the hydrographs were applied to the model considering the time of peak flow, the relative temporal positioning of the design hydrographs was not considered a concern. An example of a temporal shift in the design hydrographs is shown in Figure 48.

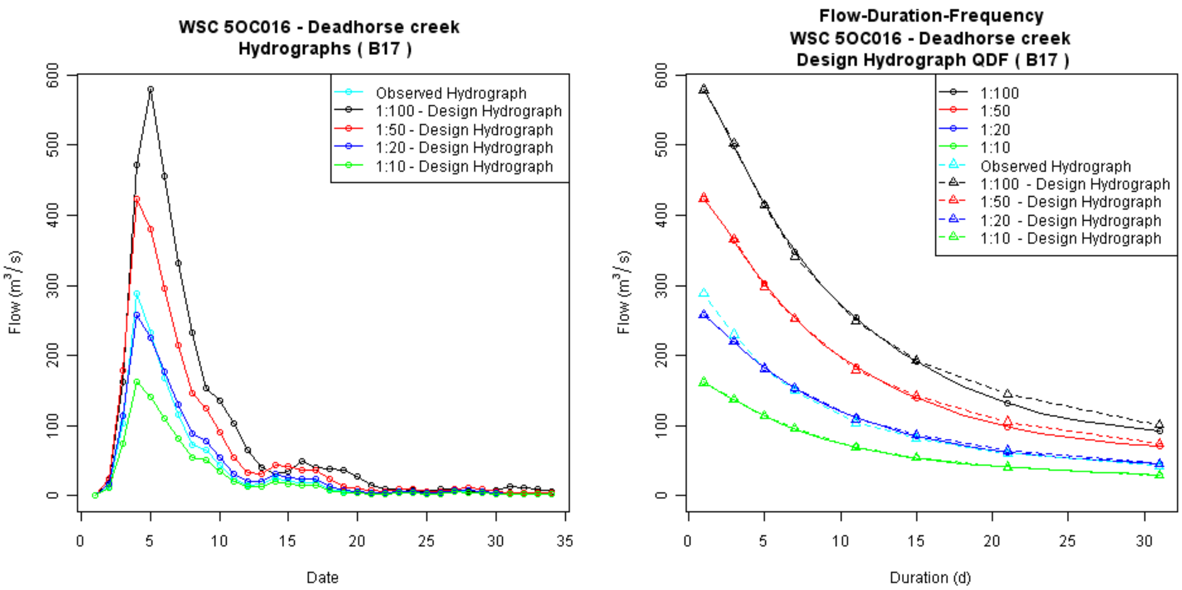


Figure 48 - Design Hydrographs - Temporal Shift

The hydrographs for each station are shown in Appendix 4.

9.2.1 Comparison of GEV and Bulletin 17B

With the adoption of the USGS Bulletin 17B methodology it was found that, in general, the estimates of the design flows was higher than when employing the GEV. As an example the differences in the QdF curves for USGS 05099600 (Pembina River at Walhalla) considering both the GEV and B17 methods are included (Figure 49). It can be seen that for this station the 1:100 produces much higher flows in general, although the higher frequency flows show less marked a difference. For this region the B17 approach seems to generally provide a conservative (higher) estimate of flow at key station locations.

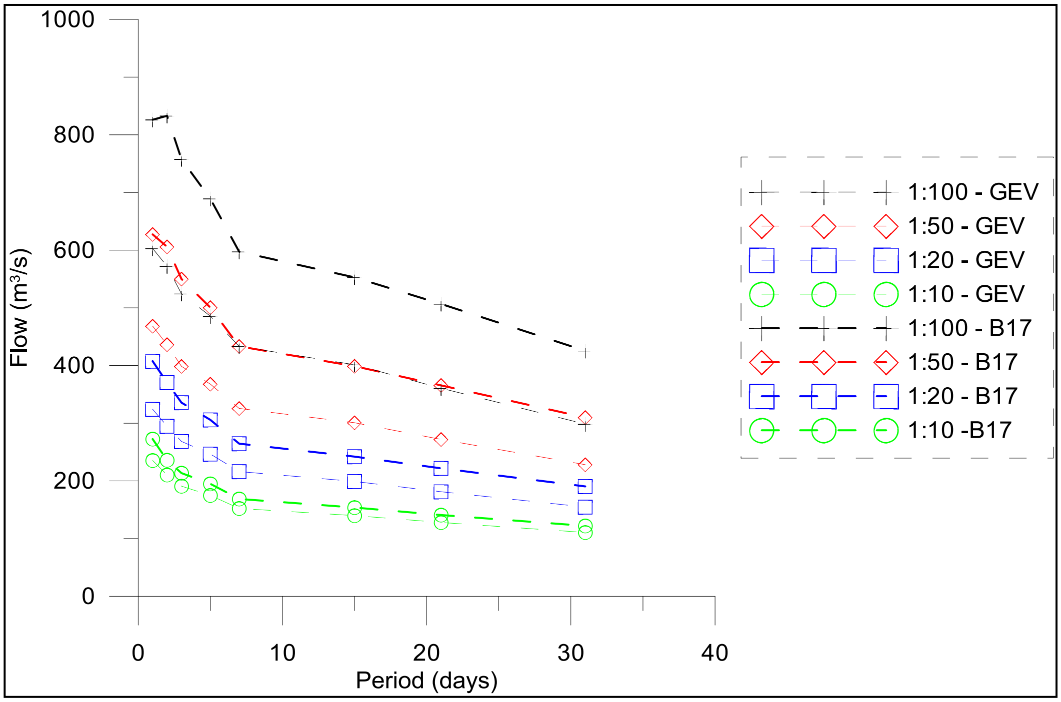


Figure 49 - QdF for USGS 05099600, GEV and LP3-Bulletin 17B (B17)

9.2.2 Event Identification

For the summer event analysis it was necessary to identify events during the prescribed summer period for frequency analysis. This preliminary identification of events was accomplished using a moving-average convergence/divergence operator to identify times when a hydrograph was beginning and when a hydrograph was ending. This population of identified events was then scrutinized visually to ensure that the start and end points of the events were consistent with the hydrographical pattern. The largest (maximum daily flow) summer event from each year for each station were then identified and employed in the frequency analysis.

The events were then each identified as a probable rainfall or snowmelt event for the events identified in May. Precipitation and temperature data were used to determine if a May event should be considered summer or winter was a combination of meteorological data from 4 separate stations. The four stations used were Gretna AUT, Emerson, Emerson Aut, and Pembina. The first three stations are from the CDCD, the last is from CDIAC (http://cdiac.ornl.gov/epubs/ndp/ushcn/access.html). The four stations were combined into a single time series as no one station had data for the complete period of analysis. The data from the station at Gretna was used primarily, and missing data was substituted from Emerson, Emerson AUT and Pembina in sequence. Events were excluded from the summer analysis if there was no rainfall precipitation signal (temperature > 0 degrees) that could have triggered the event.

In some cases the events within the summer season were influenced by the recession curve of the spring snowmelt events, producing unusually high peak flow and flow volumes for a summer-type event. To account for these influences the base flow which corresponded to these events was removed by using a straight line connection between the start of the event and end of the event. For events uninfluenced by the snowmelt period extraction of the base flow represented a negligible adjustment.

9.2.3 Characteristic Events

In order to develop the balanced hydrographs, a representative hydrograph to act as a starting point was required for each station. In order to identify a representative hydrograph to employ in the balanced hydrograph scaling methodology a semi-automated approach was employed. All the identified events for a particular station and season were collected and plotted on a single graph with the flows normalized to the observed peak flow and the time normalized to the observed time to peak (dimensionless hydrographs). A sample of one of these plots for Tongue at Akra is shown in Figure 50. On these plots the median value of all events as well as the upper and lower quartile were plotted for the time dimension, provided seven or more events were available. A routine was developed to highlight events that most closely matched the median line using a least squares approach, as well as some previously identified events that were identified as significant (high peak flows). The events that most closely represented the standard hydrograph shape were used in the balanced hydrograph approach. The median hydrographs were considered as input into the balanced hydrograph, however the median plots tended to be erratic resulting in unrealistic hydrograph shapes after balancing was conducted.

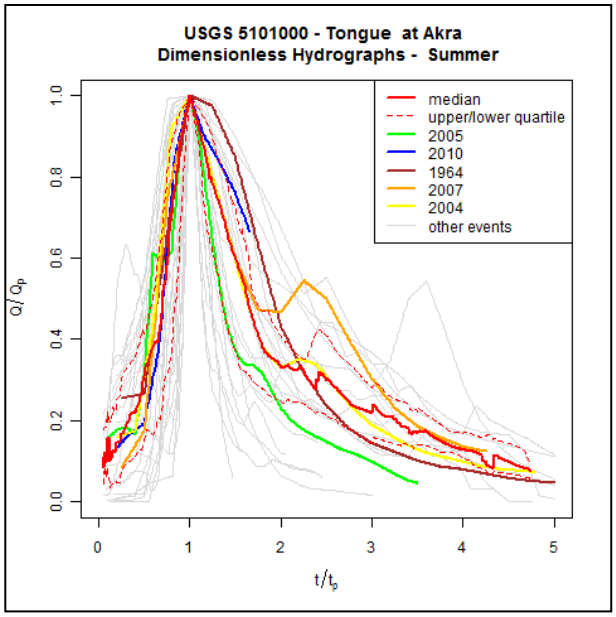


Figure 50 – Dimensionless Hydrographs – Tongue at Akra, Summer

A list of the characteristic event years identified for each station for both the annual ("winter") and summer events are listed in Table 10 below.

Station ID	Representative Event Year Winter	Summer
USGS 05101000	2006	1985
USGS 05100000	2006	1994
USGS 05099600	2006	2003
USGS 05099400	1971	1974
WSC 05OB007	2009	1982
WSC 05OC016	1979	2005
WSC 05OC019	1997	2005
WSC 05OC022	1996	2002
WSC 05OD001	1950	1998

Table 10 - List of Years of Identified Representaitve Events for Hydrometric Stations

9.2.4 Peak Flow – Max Daily Flow Relationships

Peaks flows at various return periods were calculated for the annual maximum using the available peak flow data provided by EC and USGS. For summer events where no peak flow data was available relationships were developed between the peak flows and the concurrent maximum daily flows for each station. This was done using the fitting of a linear relationship to the log-transformed flow data as indicated below in Figure 51. It was found that the least squares objective function employed was overly sensitive to outliers in cases where the residuals were clearly not normal. To provide more realistic estimates a robust regression analysis was employed instead using a Huber estimator with *k*=1.345 (Venables, 2002). The results show a more reasonable relationship with tailed outliers (USGS 0510100) and similar performance when the residuals are more normally distributed (WSC 050C019). The Peak Flow – Maximum Daily Flow relationships and associated statistics for all hydrometric stations are shown in Appendix 4.

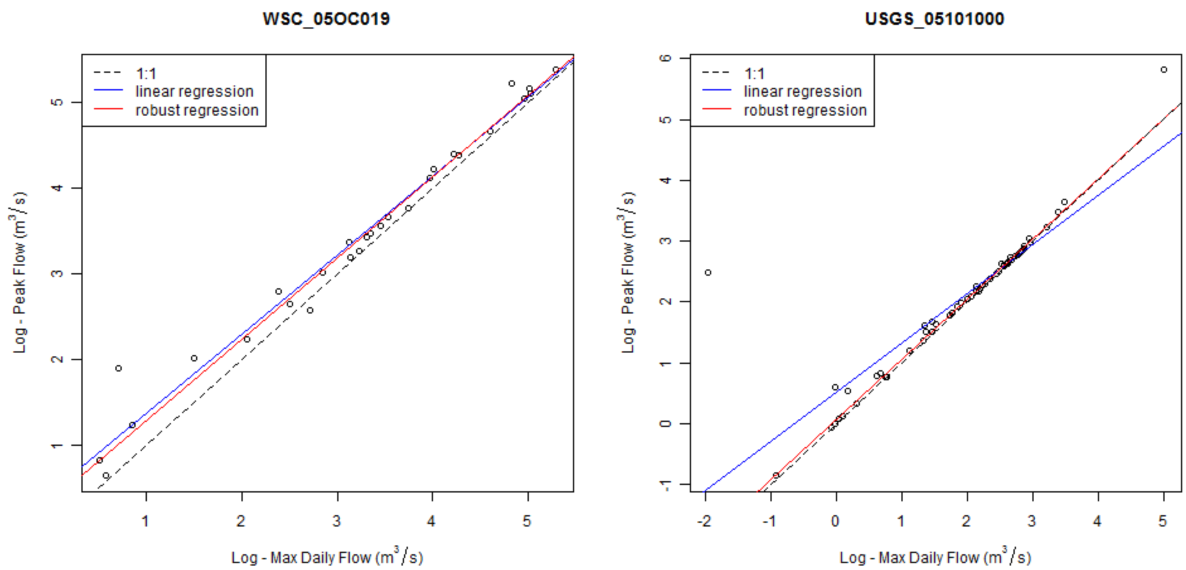


Figure 51 - Peak Flow - Max Daily Flow Linear Relationships

9.3 Conditional Probability Adjustment

For the summer flow frequency analysis, in some cases there were years with no identified summer event. In these cases the return interval frequency required a conditional probability adjustment (Maidmet et al., 1993; USGS, 1981). This is often done through graphical means but considering the number of analysis that was required (72) an automated numerical approach was developed. In this case the flow-probability plots as developed with the reduced population were plotted excluding the zero flow years. Figure 52 shows an illustrative example for station WSC 05OC019 (Buffalo creek). The probabilities were adjusted based on the ratio between the number of events and the number of years in the period of record (red triangles) and an interpolated equation was developed between these points. The new flows corresponding to the probabilities were determined by estimating the interpolated values at the appropriate target probability.

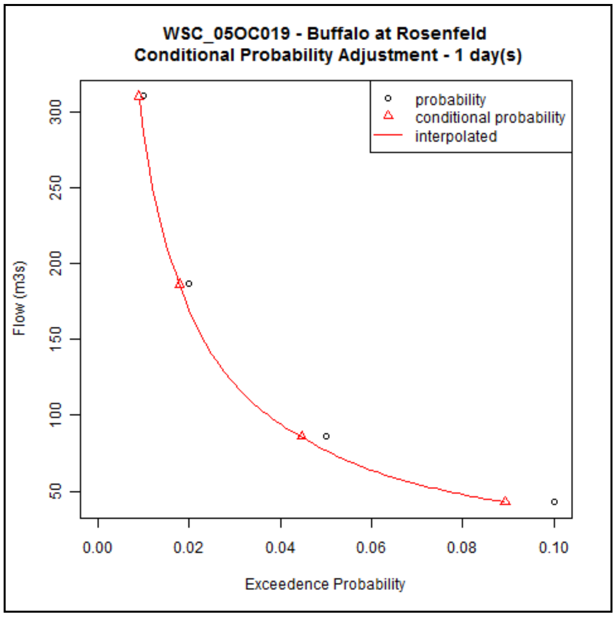


Figure 52 - Conditional Probability Adjustment - Buffalo at Rosenfeld – 1-Day Flow

To illustrate the reduction in flow the maximum flows by return period prior to conditional probability adjustment for Buffalo at Rosenfeld are shown in Table 11. After conditional probability adjustment the adjusted flows are reduced and are shown in Table 12.

The conditional probability adjustment charts for the summer flows are shown in Appendix 4.

	Maximum Average Flow - Period (days)							
Return period (years)	**1**	**3**	**5**	**7**	**11**	**15**	**21**	**31**
100	310.3	269.4	213.2	169.6	125.9	99.2	78.6	61.6
50	186.4	160.7	127.8	102.5	76.8	61.0	48.6	38.1
20	86.0	73.4	58.9	47.8	36.2	29.1	23.4	18.3

Table 11 - Maximum Average Flow by Return Period (QdF) – Buffalo at Rosenfeld

	Average Flow Period (days)							
Return period (years)	**1**	**3**	**5**	**7**	**11**	**15**	**21**	**31**
100	285.5	247.5	196.1	156.2	116.1	91.7	72.7	56.9
50	169.4	145.8	116.1	93.3	70.0	55.7	44.4	34.8
20	76.8	65.4	52.5	42.8	32.4	26.1	21.0	16.4

Table 12 - Maximum Average Flow by Return Period – Buffalo at Rosenfeld, Conditional Probability Adjusted

9.4 Drainage-Area Ratio Methodology

A methodology for estimating the design flows at the ungauged basins identified in Table 9 was required. It was determined through discussions with the IJC boards that the preferred approach would be a drainage-area-ratio methodology using the Tongue at Akra design flows as an input. Reviewing the literature, the most recent drainage area ratio equations developed in the region were found to be those produced by the USGS and reported by Emerson et al [ref 4]. This study suggests a number of area ratio curves by season for the Red river basin in North Dakota summarized below in Table 13 . The "Spring" equation was used for snowmelt events (called "Winter" in these pages) and the "Summer" equation was used for the summer events. Gross drainage area was considered in all cases.

Period	**Equation**
Winter	$Q_y = 1.24\left(\frac{A_y}{A_x}\right)^{0.85} Q_x$
Spring	$Q_y = 1.02\left(\frac{A_y}{A_x}\right)^{0.91} Q_x$
Summer	$Q_y = 1.06\left(\frac{A_y}{A_x}\right)^{1.02} Q_x$

Table 13 - Drainage Area Ratio Equations (Emerson et al., 2005)

Where A_x is the drainage area of the source station; A_y is the drainage area of the target station, Q_x is the flow at the source station and; Q_y is the flow at the target station.

The ungauged basin design hydrographs are detailed fully in Appendix 4.

9.5 Event Timing Methodology

The incorporation of the design hydrographs into the model required an estimate of relative timing of the hydrographs. The relative timing was determined using historical response in the various stations. Relative timing was compared using two methods:

- comparing the timing of peaks for common events to USGS 5099600 (Pembina River at Walhalla) and determining the mean and median differences; and
- to perform a lag cross-correlation and determine the daily lag in the time series that maximized the correlation.

For the second approach two different comparisons were made: a series of common event periods were identified and compared and; the entire seasonal period for snowmelt or summer events for the period of record were compared. All timing was compared to a reference station USGS 05099600.

The comparison of peak timing involved a direct event-by-event comparison, identifying the peak flows during the winter and summer events, determining the time of those peaks in each of the stations and comparing them to the timing to the USGS 05099600 station. For the winter period the maximum annual flows corresponding to snowmelt runoff were used. For the summer period, the summer events identified above were compared across all stations. The mean and median differences of the timing differences were calculated.

The lag cross-correlation comparison involved using comparing a flow time series from each station with that of USGGS 05099600. Two sets of time series analyses were conducted: a time series for the identified events only for both winter and summer seasons (other non-event periods excluded); as well as a complete time series for each season. For the complete seasonal time series the flow data was separated into two periods, summer and winter. The summer period was considered to be from May 1st to September 30th, and the winter period from October 1st to May 31st. A cross-correlation analysis was conducted for each station, for each year of data and compared to the equivalent time series at station USGS 05099600. The lag period for which the correlation was maximized was identified for each year and the mean and median values were calculated. The results for all these analysis are shown in Table 14 and Table 15. (Negative numbers indicate that the peak of the event occur before the peak at Walhalla)

	Peak Timing		**Lag Cross-Correlation**			
	Single Events		Single Events		Snowmelt Period	
Stations	Mean	Median	Mean	Median	Mean	Median
USGS_05101000	0.3	1.0	-0.3	0.0	0.2	1.0
USGS_05100000	1.5	1.0	0.8	1.0	1.5	2.0
USGS_05099600	*0.0*	*0.0*	*0.0*	*0.0*	*0.0*	*0.0*
USGS_05099400	-0.5	0.0	-2.4	-2.0	-1.3	-1.0
WSC_05OB007	0.3	0.0	0.3	0.0	0.7	0.0
WSC_05OC016	-0.5	-1.0	-3.8	-3.0	-1.8	-1.0
WSC_05OC019	-0.1	0.0	-2.5	-2.5	-0.9	0.0
WSC_05OC022	0.6	0.0	1.0	1.0	0.8	0.0

Table 14 - Event Timing in Days compared to USGS 05099600 (Pembina at Walhalla) – Snowmelt Events

	Peak Timing		**Lag Cross-Correlation**			
	Single Events		Single Events		Summer Period	
Stations	Mean	Median	Mean	Median	Mean	Median
USGS_05101000	1.4	1.0	1.4	1.0	0.4	1.0
USGS_05100000	1.4	1.0	1.2	1.0	1.5	1.0
USGS_05099600	*0.0*	*0.0*	*0.0*	*0.0*	*0.0*	*0.0*
USGS_05099400	-0.2	0.0	-0.1	0.0	1.0	0.0
WSC_05OB007	-0.3	0.0	-0.3	0.0	0.0	0.0
WSC_05OC016	0.8	1.0	1.0	1.0	2.2	1.0
WSC_05OC019	1.5	1.5	1.8	2.0	1.6	1.0
WSC_05OC022	2.7	2.0	-1.7	1.0	0.6	0.5

Table 15 - Event Timing in Days compared to USGS 05099600 – Summer Events

The results show that in the summer there is strong agreement between the median event delays between the three methods with differences on the order of 0.5 days. The snowmelt events show less agreement, the single event lag cross correlation somewhat different form the other two methods. However the timings tend to be within 1 to 2 days of the reference station in most cases, and of note is a one- or two-day lag to USGS 0510000 (Pembina at Neche). These timing results are in contrast to the local experience which suggests greater differences in peak timing.

Base on this local experience, it was assumed that all local rivers and creeks would peak 5 days before the peak at Walhalla

9.6 Maximum Daily Flows

The summary table of maximum daily flows for each of the stations is shown in Table 16, below.

	Coordinates		**Maximum daily flow (m^3/s)**						
	Lat	Long	1:10 years	1:50 years	1:100 years	1:20 summer	2006 flood	2009 flood	2011 flood
Aux Marais	49.075	-97.305	36.8	86.1	113.6	15.5	27.2	17.3	15.3
Buffalo	49.191	-97.403	147.5	366.7	488.7	76.8	143.0	48.7	50
DeadHorse	49.251	-97.550	161.6	424.0	579.3	89.9	109	76.7	37.8
Louden	48.912	-97.377	11.4	26.8	35.3	4.7	-	-	8(*)
Rosebud	48.893	-97.475	49.4	115.6	152.5	22.5	-	-	15(*)
Tongue at Akra	48.778	-97.746	27.8	56.1	70.2	13.5	17.3	30.6	14.3
Akra to Bathgate	48.878	-97.486	12.8	29.9	39.4	5.3	-	-	6(*)
Bathgate to Pembina	48.938	-97.306	18.6	43.6	57.5	6.6	-	-	6(*)
Roseau	49.191	-96.985	118.3	180.0	206.0	97.2	97.8	139	129
Pembina at Walhalla (peak hourly)	48.914	-97.916	304.4 (317.3)	665.4 (699.7)	863.7 (908.6)	145.2 (162.2)	362.5 (403.7)	458.7 (483)	422 (455)
Walhala to Neche	48.990	-97.557	31.7	74.3	98.0	14.0	-	-	20(*)
Neche to Pembina	48.966	-97.241	15.9	37.1	49.0	6.7	-	-	20(*)
Red at model entrance	48.896	-97.191	1347	2381	2909	1000 cnst	1900	2464	2072
Red at Morris	49.355	-97.349	1733	3098	3847	1294	2197	2780	2682
Red elevation at Morris (m)	49.355	-97.349	236.42	238.55	238.72	236.3 cnst	237.4	238.24	237.88 (*) assumed

Table 16 – Summary of maximum daily flows simulated in the various river channels

9.7 Hydrology Methodology – Sensitivity Analysis

A supplemental sensitivity analysis was conducted to examine design hydrograph sensitivity to changes in methodology including: extreme value distribution selection, drainage area ratio equation and source station selection, skew coefficient estimates, etc. The details of the sensitivity analysis are detailed in Appendix 5

10. Flood Scenarios Simulations

Several scenarios were simulated in an attempt to alter the propagation of the flood and understand better where the water is going and why. They are listed on Table 17. They were chosen after discussion with the Pembina River Basin Advisory Board (PRBAB), the Pembina River County Water Resources District, the Red River Basin Commission (RRBC) and the Lower Pembina River Flooding Task Team (task team created by the International Red River Board (IRRB).

Scenario	Geometry file	Description
Existing 2006	Geo L15 Geo L22	Calibration 2006 conditions
Existing 2009	Geo L14 Geo L21	Verification 2009 conditions
Existing 2011	Geo L21-3	Verification 2011 conditions
Existing	Geo L21-3	1:10, 1:50, 1:100 spring events 1:20 summer event
12	Geo L21-3 sce12	Remove complete border road + road West of Neche, ½ mile south of border
13	Geo L27	#Remove portion of border road West of Gretna + road ½ mile south of border + portion of road 1 mile north of border at Gretna #Improve flow passage into Buffalo River #Remove short portion of border road at Xing 6 + road 1 mile south of border East of Neche + NS roads + widen bridge under PR 243
14	Geo L21-3	Floodway
14A	Geo L21-3	Floodway short; from Border crossing 6
15	Geo L21-3	Set back dykes 400 m apart
15A	Geo L21-3	Set back dykes 400 m apart. South dyke stops at the confluence with Tongue
16	Geo L23	Natural conditions: no roads, no railway, no dykes
17	GeoL24	Remove CR 55
18	Geo L25	Remove CR 18
19	Geo L21-3	Diversion, 53% North, 47% South
20	Geo L21-3	Water storage at Walhalla 63 hm3 (51113 acft), 13% of 1997 flood in volume

Table 17 – List of scenarios tested

- In scenario 12, the entire border road was removed. At the same time, the road located ½ mile south of the border, just west of Gretna, was also removed. Generally speaking, when roads were "removed", their elevations were set to that of the surrounding area, as defined by the LiDAR surveys, about 25 m (82 feet) away from the roads.
- For scenario 13, the intent was to let water move more freely north of the border in a semi-controlled manner, by opening the border road only at two short locations and to guide the propagation of the water.
- For the floodway simulations two scenarios were tested, Scenario 14 and Scenario 14A. In the later case, the floodway was shortened to minimize its size and consequently its cost.
- In the case of the set-back dykes, two systems were tested (Scenario 15 and Scenario 15A) with the south dyke having different lengths.
- For scenario 16 all roads and railways embankments were removed and set to the elevations of the surrounding area, and the dykes along the Tongue bypass, Buffalo Creek and DeadHorse Creek near Rosenfeld were also removed.
- For scenario 20, peak water is hypothetically "stored" upstream of the Pembina River (in the model it was at Walhalla) before being released later into the river after the occurrence of the peak flows in the Red River. The amount of storage has been suggested by Charlie Anderson from WSN Engineering) [ref. 3].

Because of the large number of maps, the figures describing the flood scenarios are presented on Plates included at the end of this report in Appendix 6.

10.1 Flood-Duration Factor

Model produced flood maps which are provided in Appendix 6, represent very well the maximum extent of the flood waters over the landscape; but, as they are a depiction of the maximum extent, they cannot convey any information on the timing or the duration of flood event. Several such maps could be produced at different time steps through the event, and an analysis of these could provide information on timing and duration; however, this is quite cumbersome to so. Another method is to show in a series of graphs the variation of the water level as a function of time. This could be done at a few selected locations, but would still not provide an overall view of how different one scenario is with another over the model's domain.

The "flood-duration" factor was developed as a new parameter to characterize the combination of extent of the flood and its duration. It is calculated as the area flooded, times the duration of the local flood so that it characterizes both flood extent and duration. For instance, if 20 km^2 are flooded for 5 days, it will have a factor of 100; the same factor as an area of 10 km^2 flooded for 10 days. Its units are km^2-days.

The flood-duration factor has been calculated for each triangular numerical element within the model and added together. A numerical element was considered "flooded" if its depth was more than 15 cm (5.9 inches). Factors were computed for two separate areas, one north and one south of the border, so that when water flows from one to the other, their factors would change accordingly. The extents of both areas are shown on
Figure 53, drawn on the flood map for the 1:50 event. The delineated areas were chosen to exclude as much as possible the influence of flooding from the Red River. They do not include:

- the confluence with Tongue River
- The lower portion of Aux Marais
- The lower portion of Buffalo Creek

In most cases the simulations were run for 25 days. The 2006 event was run for 30 days and the summer event for 18 days only.

The comparison of two scenarios can be done be comparing their flood-duration factors. Within the same area, a positive difference between two scenarios indicates that the first one exhibits more flood extent, or is flooded for a longer period, than the second scenario. A negative difference indicates that the first scenario provides less flood extent within the same area. The factors are shown on Table 18.
.

As a reference, the two areas have the following total surface area:

Canadian: 666 km^2
US: 388 km^2

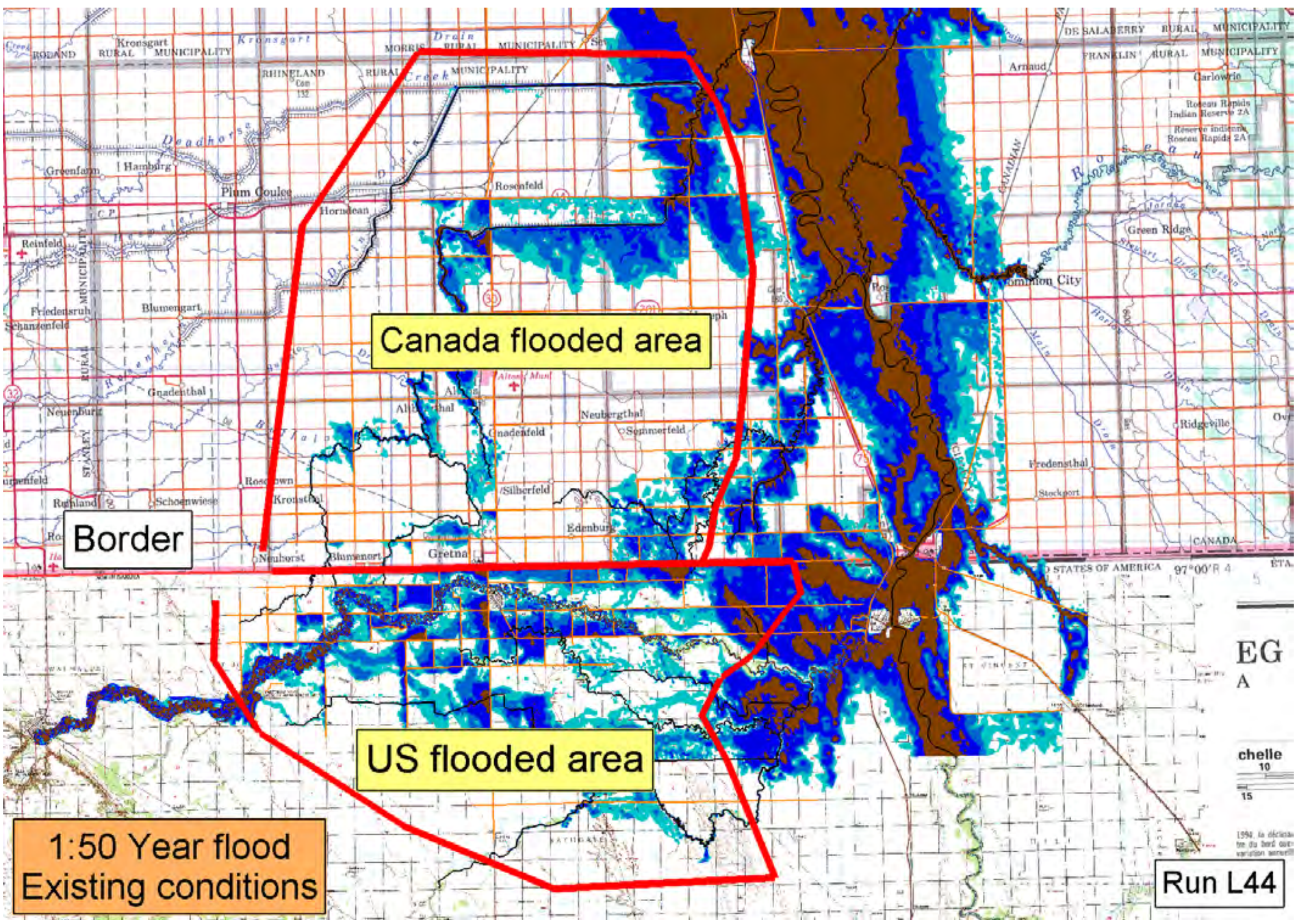


Figure 53 - Delineation showing the two areas used to calculate the flood-duration factor

In a separate series of runs, the simulations were prepared with no local runoff. This was to be able to discharge the diversion flows into empty channels, as it had been the case in 2006. In these cases, the model would show the effects of only discharging diverted flows, and provide information on the conveyance of the channels. The list of these simulations and their corresponding flood-duration factors are shown in Table 19.

Scenario	Run	Plate number	Event	Description	Maximum flooded		Flood-duration factor		Change from Existing		Change from Natural	
					km^2	km^2	km^2-days	km^2-days	km^2-days	km^2-days	km^2-days	km^2-days
					Canada	US	Canada	US	Canada	US	Canada	US
Existing	L40 L56	1	2006 flood	2006 flood	10.6	55.0	223.8	692.1	0.0	0.0	-276.3	313.7
Existing	L55	2	2009 flood	2009 flood								
Existing	L74	3	1:10 year	With all runoff	9.4	49.3	157.2	534.6	0.0	0.0	-449.0	119.2
Existing	L44P	4	1:50 year	With all runoff	55.4	103.8	821.1	1413.1	0.0	0.0	-782.0	355.1
Existing	L46Q	5	1:100 year	With all runoff	80.0	129.1	1186.6	1758.0	0.0	0.0	-1110.0	303.5
Existing	L95	6	Summer	Constant flow constant level	7.1	8.3	88.9	68.3	0.0	0.0	-1.8	0.6
12	L67	7	1:10 year	Lower border rd + 1/2 mile rd	28.2	34.8	343.0	368.6	185.8	-166.0	-263.2	-46.7
12	L68	10	1:50 year	Lower border rd + 1/2 mile rd	119.0	88.0	1422.6	1193.8	601.5	-219.3	-180.5	135.8
12	L119	13	1:100 year	Lower border rd + 1/2 mile rd	138.4	116.7	1780.7	1525.4	594.0	-232.6	-516.0	70.9
13	L96	17	Summer	Lower portions of border road	7.1	7.9	89.6	66.7	0.8	-1.6	-1.1	-1.0
13	L98	23	2006	Lower portions of border road	27.3	41.0	397.2	533.5	173.3	-158.6	-103.0	155.1
13	L102	20	1:10 year	Lower portions of border road	19.3	37.6	256.8	388.2	99.6	-146.4	-349.5	-27.2
14	L87	26	1:10 year	Floodway 50 m3/s	9.4	37.2	143.8	375.2	-13.4	-159.3	-462.4	-40.1
14	L121	28	1:10 year	Floodway 74 m3/s	9.4	30.5	142.1	322.3	-15.1	-212.3	-464.1	-93.1
14	L122	30	1:10 year	Floodway 96 m3/s	9.4	23.7	140.7	271.6	-16.5	-263.0	-465.6	-143.8
14	L123	32	1:10 year	Floodway 119 m3/s	9.4	18.0	139.2	218.8	-18.0	-315.8	-467.0	-196.5
14	L75	34	1:10 year	Floodway 142 m3/s	9.4	14.1	135.9	182.1	-21.3	-352.5	-470.3	-233.3
14	L80	36	1:10 year	Floodway 190 m3/s	9.4	12.0	130.9	155.4	-26.3	-379.2	-475.4	-259.9
14	L81	38	1:10 year	Floodway 210 m3/s	9.4	12.0	131.1	151.7	-26.1	-382.9	-475.1	-263.7
14A	L128	40	1:10 year	short Floodway 50 m3/s	9.4	40.2	145.9	409.2	-11.2	-125.4	-460.3	-6.1
14A	L129	42	1:10 year	short Floodway 67 m3/s	9.4	39.7	142.2	405.2	-14.9	-129.4	-464.0	-10.2
14	L125	44	1:50 year	Floodway 74m3/s	55.0	101.2	765.6	1209.3	-55.5	-203.8	-837.5	151.3
14	L126	46	1:50 year	Floodway 96 m3/s	54.9	99.8	756.1	1143.0	-65.0	-270.2	-847.0	84.9
14	L127	48	1:50 year	Floodway 119 m3/s	54.8	98.0	745.2	1072.4	-75.9	-340.8	-857.9	14.3
14	L84	50	1:50 year	Floodway 210 m3/s	54.5	88.2	714.6	860.6	-106.5	-552.6	-888.5	-197.5

Scenario	Run	Plate number	Event	Description	Maximum flooded		Flood-duration factor		Change from Existing		Change from Natural	
					km^2	km^2	km^2-days	km^2-days	km^2-days	km^2-days	km^2-days	km^2-days
					Canada	US	Canada	US	Canada	US	Canada	US
14A	L130	52	1:50 year	short Floodway 50 m3/s	55.3	103.4	788.0	1367.1	-33.1	-46.1	-815.1	309.0
14A	L131	54	1:50 year	short Floodway 74 m3/s	55.2	103.1	768.1	1352.3	-53.0	-60.9	-835.0	294.2
14A	L132	56	1:50 year	short Floodway 96 m3/s	55.2	102.6	755.8	1348.2	-65.3	-65.0	-847.3	290.1
14A	L133	58	1:50 year	short Floodway 142 m3/s	55.1	101.2	747.8	1340.3	-73.3	-72.9	-855.3	282.2
14A	L135	60	1:50 year	short Floodway 181 m3/s	55.2	100.3	746.4	1337.0	-74.7	-76.2	-856.7	278.9
15	L70	62	1:10 year	Set back dykes	9.4	17.2	131.4	234.5	-25.8	-300.1	-474.8	-180.9
15	L71	66	1:50 year	Set back dykes	54.4	36.8	693.1	738.0	-128.0	-675.1	-910.0	-320.0
15A	L112	64	1:10 year	Set back dykes - shorter	9.4	17.3	131.4	235.2	-25.8	-299.4	-474.9	-180.2
15A	L120	68	1:50 year	Set back dykes - shorter	54.4	33.7	694.8	650.6	-126.3	-762.5	-908.3	-407.4
15	L34	70	2006	Set back dykes	9.5	15.8	196.0	251.3	-27.8	-440.8	-304.1	-127.1
16	L69	78	1:100 year	Natural conditions	159.6	99.7	2296.7	1454.5	1110.0	-303.5	0.0	0.0
16	L85	75	1:50 year	Natural conditions	129.6	80.2	1603.1	1058.1	782.0	-355.1	0.0	0.0
16	L73	72	1:10 year	Natural conditions	38.5	35.4	606.2	415.4	449.0	-119.2	0.0	0.0
16	L99	81	summer	Natural conditions	10.0	7.4	90.7	67.8	1.8	-0.6	0.0	0.0
16	L100	84	2006	Natural conditions	35.2	31.0	500.1	378.4	276.3	-313.7	0.0	0.0
17	L72	86	1:10 year	Remove CR55	9.4	52.7	156.8	527.5	-0.3	-7.1	-449.4	112.2
17	L117	88	1:50 year	Remove CR55	55.4	100.6	839.0	1413.9	17.9	0.7	-764.1	355.8
17	L118	90	1:100 year	Remove CR55	87.8	124.0	1270.1	1764.4	83.5	6.4	-1026.6	309.9
18	L76	92	1:10 year	Remove CR18	9.4	48.2	156.8	524.0	-0.4	-10.6	-449.4	108.7
19	L78	94	1:10 year	Diversion 210 m3/s	26.8	28.4	350.7	387.2	193.5	-147.4	-255.5	-28.2
19	L82	97	1:10 year	Diversion 190 m3/s	25.0	27.7	343.6	379.0	186.4	-155.6	-262.6	-36.3
19	L83	99	1:10 year	Diversion 142 m3/s	20.8	28.8	327.2	379.4	170.0	-155.2	-279.0	-35.9
19	L86	102	1:10 year	Diversion 50 m3/s	13.4	41.8	222.7	475.9	65.5	-58.7	-383.5	60.5
19	L97	105	Summer	Diversion 50 m3/s	7.6	8.3	100.2	74.6	11.4	6.2	9.5	6.8
20	L103	107	1:10 year	Water storage	9.4	13.3	137.2	183.4	-20.0	-351.2	-469.0	-232.0
20	L104	109	1:50 year	Water storage	55.0	73.3	743.0	1195.9	-78.1	-217.2	-860.1	137.9

Table 18 – List of simulation runs with their maximum extent and flood-duration factor

				No runoff in Aux Marais, Buffalo, Rosebud, Louden								
					Maximum flooded area		Flood-duration factor		Change from Existing		Change from Natural	
					km^2-days	km^2-days	km^2-days	km^2-days	km^2-days	km^2-days	km^2-days	km^2-days
Scenario	**Run**	**Plate**	**Event**	**Description**	CDN	US	CDN	US	CDN	US	CDN	US
Existing	L92	111	1:10 year	Existing conditions	4.9	46.8	68.6	478.3	0.0	0.0	-140.6	219.1
13	L93	113	1:10 year	Lower portions of border road	17.4	35.2	182.2	332.8	113.6	-145.6	-27.1	73.5
15	L101	115	1:10 year	set back dykes	3.1	14.5	44.1	180.1	-24.5	-298.2	-165.1	-79.1
16	L94	122	1:10 year	Natural conditions	22.9	28.4	209.2	259.2	140.6	-219.1	0.0	0.0
19	L90	117	1:10 year	Diversion 142 m3/s	17.3	24.6	225.6	298.7	157.0	-179.6	16.3	39.5
19	L89	120	1:10 year	Diversion 50 m3/s	9.7	39.7	143.2	407.2	74.6	-71.1	-66.0	148.0

Table 19- List of simulation runs with their maximum extent and flood-duration factor – Case of no runoff

10.2 Existing configurations

The maximum flood extent maps are shown on Plates 1 to 6 for all six events: 2006; 2009; 1:10; 1:50; 1:100; and the 1:20 Summer. It is to be noted that all these maps, and the subsequent maps in the report, show the extent of the water when each model grid element was at its maximum depth, noting that their maximum depths may not have occurred at the same time over the entire region.

The peak discharges that occurred within the local drainage areas during these simulations are indicated on Table 20. They will be used as references for comparing results from other scenarios. The cross sections in which these flows were estimated are indicated on Plate 78 which shows the 1:100 event in Natural Conditions and therefore gives a good indication of the length of the cross sections.

	Summer	1:10 event	1:50 event	1:100 event	2006 flood
Main Pembina River channel, 8 km downstream from Neche	133	147	152	156	146
Aux Marais River (m^3/s)	9	10	59	81	10
Aux Marais River with Red River runoff (m^3/s)	9	19	84	195	10
Buffalo Creek west (m^3/s)	0	0	23	57	0
Buffalo Creek east (m^3/s)	15	30	79	106	29
Rosebud Coulee (m^3/s)	6.7	33	164	300	30
Louden Coulee (m^3/s)	5.3	34	80	112	44

Table 20 – Existing conditions - Peak discharges in the channels going north and south of Pembina River

10.3 Scenario 12: Removal of border road and road located half mile south of the border

In this scenario the border road was removed over its entire length from 1.5 km west of Crossing 4, (see its location in Figure 14), to 1.3 km west of Hwy 75 near Emerson. The east-west road located ½ mile south of the border, just west of Gretna was also removed because, as seen on Plates 1 to 5, it held the floodwaters and carried them east towards Neche. By removing this short road, it relieved some of the water west of Neche by allowing it to run north. Plates 7 to 9 show the 1:10 year event, Plates 10 to 12 show the 1:50 event and Plates 13 to 15 show the 1:100 event.

(The short road ½ mile south was removed in Phase 2 of the study while keeping the border road [Ref 2]. It was found then, that the presence of the road increased levels West of CR18. In this Phase 2, the border road alone was also removed. Comparing flood extent and flow propagation for this previous case and the new Scenario 12 shows very identical maps, except for the storage on the south side of the short road).

In this scenario the water is now stopped by PR 243 in Canada, one mile north of the border. This road is eventually overtopped at several places, and the water continues north towards Aux Marais and

Buffalo. One will notice 3 km west of PR 30 that the water cannot reach the main channel of Buffalo Creek because of its high banks. Instead it flows between its right bank and PR 30.

This scenario slightly lowers the levels held back on the north side of CR 55, but does not change levels held back by CR18 south of the Pembina River. For the 1:100 event this scenario does not affect flooding south of CR 55.

On the Red River, this scenario reduces levels as compared with existing conditions by about 5 cm (2 inches) between Pembina and Letellier for the 1:10 event and 7 to 13 cm (3 to 5 inches) for the 1:50 event. There is also a very important reduction in water levels occurring east of Switzer Ridge for the 1:10 year event (Plate 8). This area is now almost dry.

During the 1:50 event, Buffalo Creek capacity is largely exceeded, and water overflows towards Rosenfeld, but it is stopped by the dykes along Buffalo channel, south of Rosenfeld, creating a large pool south of PR14. East if Switzer Ridge there is still a large area full of water, which is filled from Red River overflow and not from the overtopping of the Ridge.

The simulated peak discharges going North through Aux Marais and Buffalo are shown in Table 21. They are significantly increased from the existing conditions. On the south side, note that Rosebud Coulee is not affected by the removal of the border road, and that the flows in Louden Coulee are slightly decreased from existing conditions. This is due to the fact that the road ½ mile south of the border is not holding water, therefore lowering the levels within the Pembina main channel and minimizing the break-outs to the south towards Louden Coulee.

	1:10 event	**1:50 event**	**1:100 event**
Aux Marais River (m^3/s)	42	86	97
Aux Marais River with Red River runoff (m^3/s)	48	209	393
Buffalo Creek west (m^3/s)	0	51	88
Buffalo Creek east (m^3/s)	43	172	200
Rosebud Coulee (m^3/s)	33	164	300
Louden Coulee (m^3/s)	32	66	95

Table 21 – Scenario 12: Peak discharges in the channels going north and south of Pembina River

When comparing this scenario to natural conditions for the 1:10 event (Plate 9), this scenario provides a similar pattern except for the storage behind many of the roads. The effect of roads on the distribution of flows is quite significant as shown on Plate 10 for the 1:50 event. The network of roads forces the water to spread horizontally in an east to west direction, much more so than occurs during natural conditions. This explains the large extent of red colours on Plate 12.

10.4 Scenario 13: Removal of portions of border road

In the previous scenario, Plate 7 showed that water crossed the border at several locations thereby creating many areas of local flooding on the north side of the border. In scenario 13, the border road was removed at only two locations over short distances:

- West of Gretna (Plate 16- left): The road ½ mile south of the border is removed as well as the local roads, to allow a passage from the Pembina River towards the East branch of Buffalo Creek. A short section of the south dyke on this Buffalo branch was also removed to allow water to reach the main channel instead of inundating the town of Gretna. These are indicated on Plate 16-left by the red circles. The passage at the border road was 1800 m wide (5900 ft), and the sill on the dyke of Buffalo Creek was set at 251.4 m, therefore lowering the Buffalo right bank dyke by 0.6 m (2 ft) over 75 m (250 ft).
- Crossing 6 (Plate 16-right): The road 1 mile south of the border was removed as well as short portions of three north-south local roads. The border road was removed over a distance of 200 m (660 ft) to allow water to flow freely toward the Aux Marais River. At the same time the passage under PR 243 bridge, just north of Crossing 6, was widened to 150 m (500 ft).

This scenario was tested with three flood events: the 1:20 summer (Plates 17 to 19); 1:10 event (Plates 20 to 22); and 2006 (Plates 23 to 25).

One of the advantages of this scenario, compared to the removal of the entire border road, is that the flood is concentrated on fewer places as can be seen when comparing Plate 7 with Plate 20. It does not change significantly flooding along the Aux Marais. This scenario decreases the flood-duration factor on the Canadian side from 343 to 257 km^2-days, but increases it slightly on the US side from 369 to 388 km^2-days.

The maximum discharges going north are shown in Table 22.

	Summer	1:10 event	2006 event
Aux Marais River (m^3/s)	9.9	59	63
Buffalo Creek west (m^3/s)	0	0	4.5
Buffalo Creek east (m^3/s)	15	30	39

Table 22 - Scenario 13: Peak discharges in the channels going north from Pembina River

10.5 Scenario 14: Floodway

In Scenario 14, a “floodway” was simulated, designed to carry a given amount of water from about 8 km upstream from Neche, to the Red River. Its intake and exit are shown by the red triangles on Plate 26. They were chosen so that the intake would be upstream of the first breakout on Pembina River and the outlet would be downstream from Pembina and Emerson. The floodway was not represented physically using a numerical grid, but rather it was modelled as a sink-source combination with the water removed from the Pembina River and reintroduced into the Red River. The berms on the south side of the floodway, preventing the flood water from moving north towards the border was modelled, and new culverts, similar to border Crossings 5 and 6 , were modelled to allow flood waters to move under the Floodway, into the channels on the Canadian sides. Siphons were also provided to allow flood waters from the Red River to flow north across the border road under the floodway (Plate 26).

Seven floodway capacities were simulated: 50, 74, 96,119,142,190 and 210 m^3/s (1770, 2600, 3400, 4200, 5000, 6710 and 7420 cfs).

The 210 m^3/s flow was derived so that Pembina River would not break out downstream of Neche for the 1:10 event.

During the 1:10 event, Plate 26 to 39 show an increasing flood reduction in the Pembina flood plains south of the border, as the floodway capacity is increased. With a capacity of 210 m^3/s (7420 cfs) the floodway takes away all the extra flows that the Pembina main channel cannot carry. But the inundations due to the local runoff are still visible on the plates, particularly along the Rosebud Coulee and the Aux Marais River (Plate 38). A capacity of 142 m^3/s is required to prevent any overtopping of Switzer ridge and the ponding of water east of the ridge. North of the border, the floodway does not have any effect on Buffalo Creek, but minimizes flooding along Aux Marais River.

Figure 54 shows the water level on the Red river at the outlet of the floodway. It indicates a raise of 24 cm (9.4 inches), occurring at the time of maximum floodway discharge, 5 days before the peak on the Red River, therefore not affecting its peak levels.

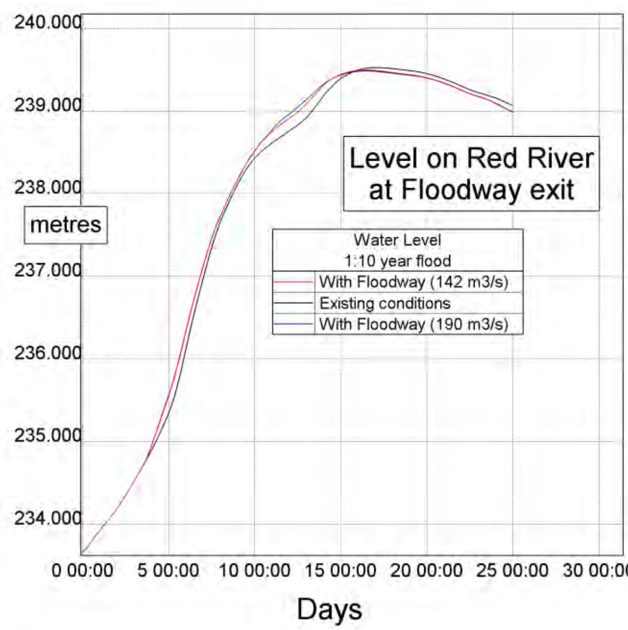


Figure 54 – Water level on the Red River at the outlet of the floodway

Figure 55 shows the water level on the US side of border Crossing 6. It indicates not only a reduction in water levels but also a significant reduction in the duration of the flood. Even for a small floodway discharge of only 50 m^3/s, the duration of the flood is reduced from 17 to 8.5 days. This is also reflected in the flood-duration factor (Table 18) which is increasingly reduced on the US side by 159 km^2-days for a floodway of 50 m^3/s, and by 383 km^2-days for a floodway of 210 m^3/s. On the Canadian side, the flood-duration factor remains at about same level, with a very slight decrease as the floodway capacity is increased.

For the 1:50 event, four capacities 74, 96,119 and 210 m^3/s (2600, 3400, 4200 and 7420 cfs) were tested, as shown on Plates 44 to 51. In this case, flood reductions are quite significant along Aux Marais River and between the border and the upstream of Buffalo Creek. On the US side, flood reduction is also significant south of the border, but has only a small impact south of CR55 for floodway capacities less than 119 m^3/s. At the outlet of the floodway on Red River, the peak levels rose from 2.0 to 3.4 cm (0.8 to 1.3 inches) for capacities between 74 and 210 m^3/s.

Simulations were not done for the 1:100 year event as it is anticipated that widespread flooding would occur even with consideration given to the larger floodway capacities of 190 m^3/s (6,710 cfs) and 210 m^3/s (7,420 cfs). This flooding results as the magnitude of the design event largely exceeds the combined capacities of the floodway and the Pembina River main channel.

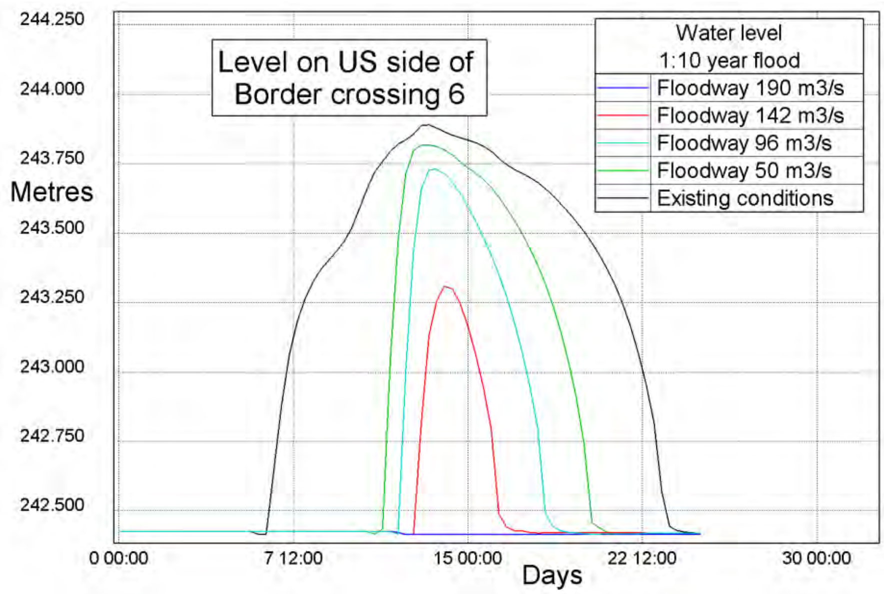


Figure 55 - Comparison the water level upstream of Crossing 6, for various floodway capacities

10.6 Scenario 14A: Short Floodway

The above floodway (scenario 14) was designed to remove some water from upstream of Neche, and carry it to the Red River. It was very long and required a bridge for crossing CR 18. A shorter floodway was proposed to remove water from further east, at Border Crossing 6 (see red triangles on Plate 42), to make it shorter, therefore less expensive, while controlling only the flows over Switzer Ridge and the large flooding east of the ridge. Flooding along the border road between Blumenort and CR18 would remain similar to the present conditions, but flooding would be improved between CR18 and Crossing 6 by removing the road ½ mile south of the border, as well as removing the short north-south roads which impede the east-west progression. These modified roads are shown on Plate 16 right. The road ½ mile south of the border, west of CR18 was not altered.

In the simulation, Border Crossing 6 remained the same with the four large culverts discharging into the Aux Marais drainage system, therefore minimizing the capacity of the floodway. Switzer Ridge was also set at its natural average elevation (243.6 m, (799.2 ft)) by filling the eroded channels.

Plates 40 to 43 show the flood extent and flood reduction in the case of the 1:10 event for capacities of 50 and 67 m^3/s (1770 and 2370 cfs). For this event, a capacity of 67 m^3/s was sufficient to avoid any overtopping of Switzer Ridge therefore preventing the creation of the large flooding east of the ridge.

For the 1:50 event, Plate 52 to 61 show that there is no change in flood extent (from the existing conditions) West of Neche, whereas, East of Neche there is an increased reduction in flood extent as the capacity of the short floodway is increased. A capacity of 181 m^3/s (6390 cfs) is needed to avoid any overtopping of Switzer Ridge. The flooding east of the ridge (seen on Plate 60) is mostly due to water coming from the Red River, and to a much smaller extent, water from the Pembina River running along the road located 1.5 mile south of the border (see red circle on Plate 60).

Figure 56 shows the water level on the US side of border Crossing 6. It differs from Figure 55 in the fact that the time of arrival of the flood waters at the crossing does not change with the capacity of the floodway, (as this was the case for the “long” floodway, scenario 14, that was removing water from the Pembina well upstream of Neche). Water levels rise, blocked by Switzer Ridge, until the floodway intake is sufficiently submerged to start removing water (day 6). As the water continues to approach from the west along the border, levels at the crossing remain at about the same level until the intake is full (day 10) and the floodway cannot absorb more flow, at which time flood water levels continue to rise, until it overtop the ridge at the elevation 243.6 m.

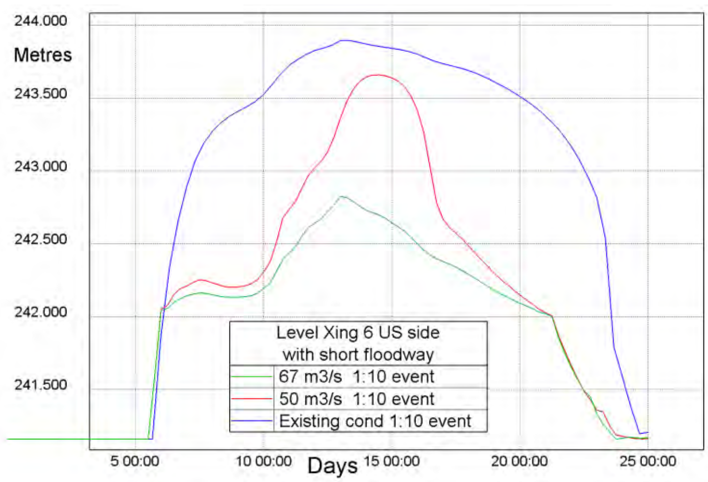


Figure 56 - Comparison the water level upstream of Crossing 6, for various capacities of the short floodway (1:10 event)

10.7 Scenario 15: Set-back dykes

In Phase 2 of the project a set-back dyke system, 800 m wide (2600 ft), had been simulated along Pembina River. It showed that for a flood similar to the 2006 event, the dykes should be mostly between 1 and 1.5 m (3.3 and 5 ft) high.

In this scenario it has been assumed that the river would be contained by a set of dykes nominally only 400 m apart (1,300 ft). In fact because of the tight meanders of Pembina River, the width between the left and right dykes varied between 398 m and 689 m (1,305 and 2,260 ft), with a mean width of 463 m (1,519 ft) .

In this scenario, the south dyke turned south at the confluence with the Tongue River, to reach the Tongue bypass. Its length was 49.96 km (31.2 mi). On the north side, it tied-in with Interstate 29, having a total length of 48.96 km (30.6 mi).

The flood maps and the corresponding change in maximum water depth are shown on Plates 62-63, 66-67, 70-71 for the 1:10 and 1:50 events and the 2006 flood. The set-back dykes are very effective in controlling the Pembina River water, but they have no effect on the local runoff (similarly to the floodway).The local flooding on the Rosebud, Louden, Aux Marais and Buffalo are clearly visible on Plates 62 and 66. On Plate 67, the increase in water level at the confluence of Louden and Tongue is due to the local runoff on the Louden which is held back by the dykes on the west side of the Tongue River. In the model, in an attempt to remove this water, two 5.9 m^3/s (207 cfs) pumps were simulated to pump the water from Louden coulee, over the dyke, into Tongue River. This pumping was sufficient for the 1:10 event but not enough for the 1:50 event. This problem did not occur for the 2006 flood (Plate 70) since there was little to no local runoff during that event.

At the confluence with Tongue River (on the east side of the dyke), since all the water is now concentrated in one location, its level was increased by up to 23 cm (9 inches) for the 1:10 event and 30 cm (12 inches) for the 1:50 event.

For these two events the peak levels on the Red River were not affected by the set-back dykes. But for a 2006 event when the timing in the peaks between Pembina and Red was different, the peak levels on Red River increased by up to 12 cm (4.7 inches) compared to existing conditions, as seen by the wide orange colour along Red River on Plate 71.

Figure 57 shows the maximum depth along the toe of the dykes as a function of the distance going from upstream to downstream. Most of the apparent oscillations in the depth come from the roads and depressions in the topography. This graph represents the height of the dykes required to contain the 1:10 event without free board.

Similarly Figure 58 shows the graph of the minimum height required for dykes to contain the 1:50 event. They are about 1 m higher than for the 1:10 event. Once again, the estimates do not include freeboard.

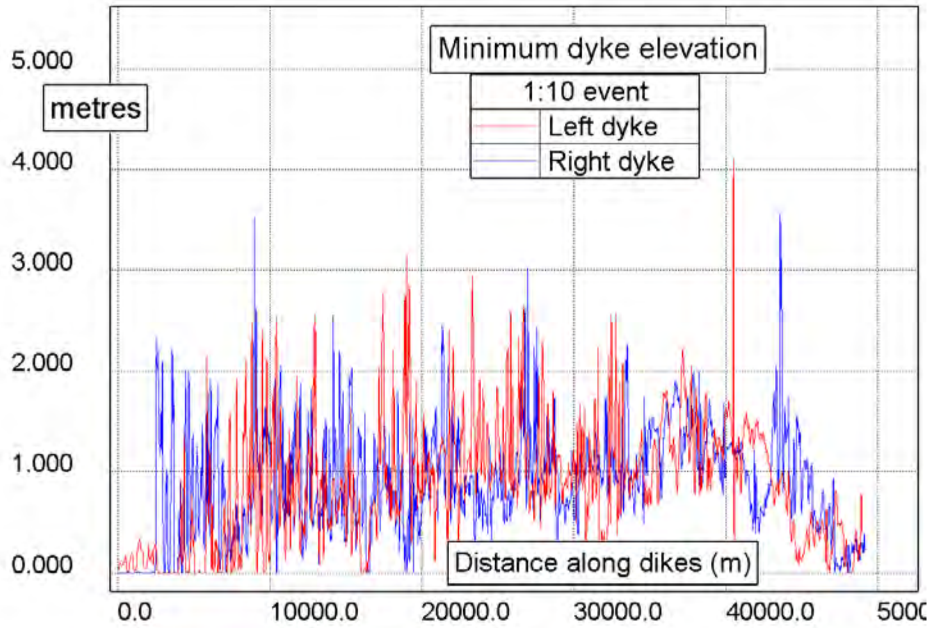


Figure 57 - Minimum dyke elevation along the entire set-back dyke from upstream to downstream, to contain the 1:10 event

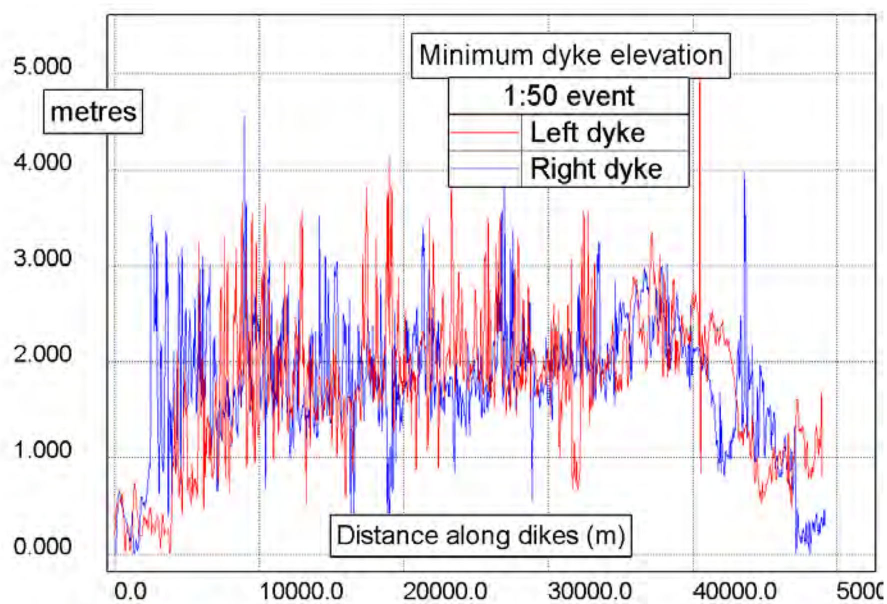


Figure 58 - Minimum dyke elevation along the entire set-back dyke from upstream to downstream, to contain the 1:50 event

10.8 Scenario 15A: Short Set-back dykes

As seen in the preceding Scenario 15, the local runoffs are trapped behind the dyke which was set along the Tongue River. In the model this water was pumped over the dyke into the Tongue River, but this required two large pumps which may be sufficient for the 1:10 event but not for the 1:50 event.

In Scenario 15A, the south dyke was shortened, ending at the confluence with the Tongue River. Its length was 42.36 km (26.5 mi). The flood maps and the corresponding change in maximum water depth are shown on Plates 64-65 and 68-69 for the 1:10 and 1:50 events. There is still a rise in maximum water levels (Plates 65 and 69) at the confluence with Tongue River due to the water being concentrated in one location: its level was increased by up to 24 cm (9.5 inches) for the 1:10 event and 33 cm (13 inches) for the 1:50 event, very slightly more than in the case of Scenario15.

It is to be noted that for the 1:10 event this change in maximum levels from existing condition is only on the west side of IS29. There is no change along the Pembina community dykes, east of IS29. For the 1:50 event, the increase in water level along these community dykes is 3 to 5 cm (1.2 to 2 inches).

Having a shorter south dyke along the Pembina River, ending at the confluence with Tongue River, did not change its minimum required height (Figure 57and Figure 58).

10.9 Scenario 16: Natural conditions

In Scenario 16 all roads and railway embankments were set at the elevations of the natural ground, as defined by the LiDAR surveys, about 25 m (82 feet) away from them. For all these simulations, the bridge piers along the Pembina River were left in place. (A verification of the influence of these piers is presented in section 10.9.1, which indicates that the piers have in fact a negligible effect on the Natural Conditions flood extent.)

Along the border, the road and the drain were set at the elevation of the natural ground as found further away from the road. Along Buffalo Creek, downstream from Altona, the dykes on both sides of Buffalo Lake Channel were also lowered. The dykes on both side of the Tongue Bypass were lowered as well. The elevations of the bottom of the rivers and coulees were left unchanged.

This scenario was simulated for all the four design events and the 2006 flood. Plates 72 to 85 show the corresponding maximum depths and their changes from the existing conditions.

In general the flood waters tend to flow in north-east and south-east directions. They follow the east and west drainage channels of Buffalo Creek, the west channels of Aux Marais River system and the local drainage channels for Louden, Rosebud and County ditch 42.

During the simulation, discharges were measured in the six cross sections where water was propagating, shown in red on Plate 78. The cross section in Aux Marais River was chosen so that it did not include the overflow from the Red River. The peaks of these discharges are indicated on Table 23.

	Summer	1:10 event	1:50 event	1:100 event	2006 flood
Main channel Pembina, 8 km downstream from Neche (m^3/s)	129	133	133	133	133
Aux Marais River (m^3/s)	4.5	29	50	52	31
Aux Marais River with Red River runoff (m^3/s)	4.5	29	134	327	31
Buffalo Creek west (m^3/s)	0	0	94	172	4
Buffalo Creek east (m^3/s)	15	64	190	226	82
Rosebud Coulee (m^3/s)	6.8	51	144	215	51
Louden Coulee (m^3/s)	15	51	127	179	55

Table 23 – Natural conditions: Peak discharges in the channels going north and south of Pembina River

This table shows a maximum flow in the Pembina River main channel of only 133 m^3/s, compared to a larger flow rate (156 m^3/s for the1:100 event). This is due to the fact that in existing conditions the water level in the channel is maintained higher by the presence of CR55 (Plate 5 and 78), and the model calculates the flow rate in the cross sections under the elevated free surface. In Natural Conditions, CR55 is removed and the model calculates the flow rate up to the top of the natural banks.

The table also shows a regular flow (30 to 52 m^3/s) into Aux Marais River, but a strongly increasing flow going into the other drainage channels. A comparison of Plates 72, 75 and 78 indicate that the breakouts from Pembina River first follow Louden Coulee and Buffalo East drainages. Then an earlier breakout on the north side, occurring upstream of the Louden breakout, flows into Buffalo West drainage. There is therefore an unbalance between the flows going north and those going south.

The peaks in Table 23 do not necessarily occur at the same time and do not give a good evaluation of the amount of water (in volume) flowing north over the border or south. To estimate this volume, the hydrographs for the 1:100 event (shown on Figure 59) were integrated over time.

Table 24 indicates the distribution of the volume of water having propagated in each of these channels during the simulation. It shows that for this event, close to 1/3 of the flow stays within the main channel, more than 1/3 (35.7 %) flows North and less than 1/3 (31.8%) flows south. If we consider only the overland flow, and not the Pembina main channel, 53 % is directed north over the border and 47 % is directed south.

In the subsequent scenario 19 that simulates the diversions along five corridors, the same flow distribution will be maintained.

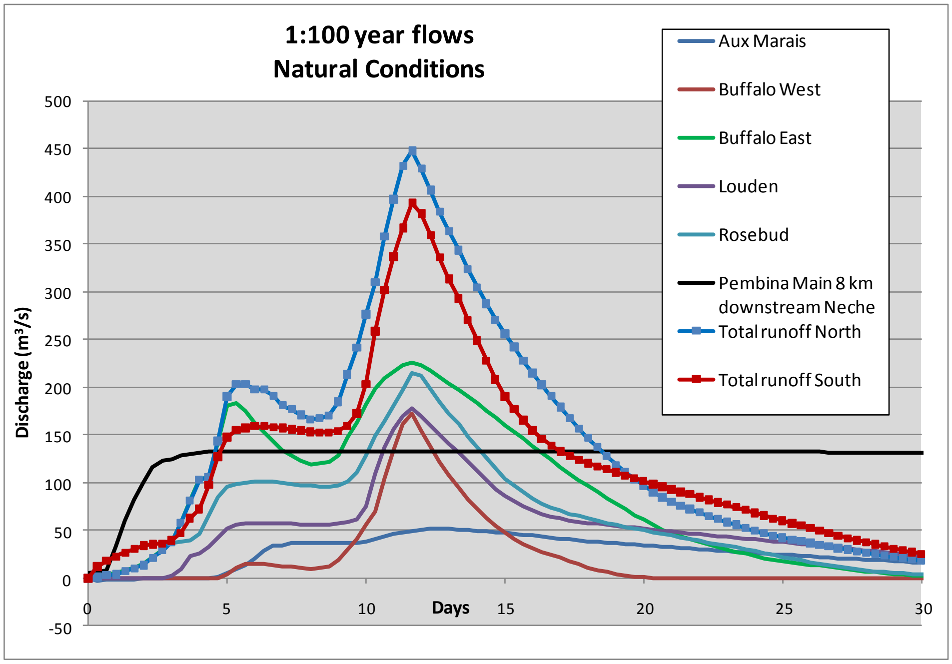


Figure 59 – Natural conditions - hydrographs in the various channels during the 1:100 event

Total volume of water after 31 days - 10^6 m^3									
100 year event									
	Total	Pembina 8 km from Neche	North	South	Buffalo West	Buffalo East	Rosebud	Lauden	Aux Marais
Volume	1043.5	339.3	372.4	331.8	61.1	236.8	182.6	149.3	74.5
%	100.0	32.5	35.7	31.8	5.9	22.7	17.5	14.3	7.1

Table 24 – Natural conditions - Distribution of flow in volume, across the various channels (1:100 event)

10.9.1 Natural conditions without bridge piers

As mentioned previously the natural conditions scenario were run without the roads but with the bridge piers along Pembina River still in place. To verify the effect of these piers, they were removed in one case (1:10 event), and the corresponding flood characteristics were compared with the run where bridge piers in place. Table 25 shows only minor differences in water levels along the river (2 cm), with negligible differences in the amount of flow travelling through flood plains, and negligible differences in the maximum flood extent. It can be concluded that all results in scenario 16, simulating Natural Conditions with the bridge piers in place are therefore reliable.

Natural conditions - all roads removed - 1:10 years event			
	With bridge piers along Pembina River	Without bridge piers along Pembina River	Difference
Run number	L73	L113	
Peak elevation at Neche (m)	253.26	253.24	-0.02
Peak elevation 8km downstream from Neche (m)	247.55	247.53	-0.02
Total volume flow Buffalo West (hm^3)	0.00	0.00	0.00
Total volumeflow Buffalo East (hm^3)	30.20	30.10	-0.10
Total volumeflow Aux Marais (hm^3)	20.10	20.10	0.00
Total volumeflow Rosebud (hm^3)	27.60	27.50	-0.10
Total volumeflow Louden (hm^3)	36.00	35.80	-0.20
Total volumeflow main channel 8 km from Neche (hm^3)	238.80	239.20	0.40
Max flood area Manitoba (km^2)	38.46	38.46	0.00
Max flood area North Dakota(km^2)	35.40	35.32	-0.08
Area-duration factor Manitoba (km^2-days)	606.22	605.37	-0.85
Area-duration factor North Dakota (km^2-days)	415.36	414.31	-1.05

Table 25 – Comparison of flood characteristics with and without the bridge piers in place

10.9.2 Comparison natural-existing conditions

When the road network was built, it has created regions of the flood plain which were not flooded prior of the construction (called natural conditions) but are now flooded in the existing conditions, and vice versa. There are also regions which are flooded in both scenarios, natural and existing conditions, but for which the duration of the flood has changed. A separate study was completed, with a comparison between the regions flooded in one case or the other, and is summarized in Table 26. This table was derived by looking at all the individual cells of the numerical mesh. If one of them was wet at any time during the 25 days of the simulation, with a depth of more than 15 cm (0.5 ft) it was considered as flooded. Note that this analysis is different from the derivation of Table 18 which was based on the maximum overall flooded area during the simulation. Therefore the surface areas estimated by both methods will be different. In Table 26 the “average duration” was a weighted average based on the surface area of the individual numerical cells. In particular, a large cell on the flood plains (several thousands of square metres) flooded only for a few days will have much more weight than a smaller cell within the river channels (several tens of square metres) always full of water.

		1:10 event		1:50 event		1:100 event		Summer	
		Manitoba	North Dakota	Manitoba	North Dakota	Manitoba	North Dakota	Manitoba	North Dakota
Region flooded in Natural conditions only	Area (km2)	68.06	8.23	115.47	16.77	112.01	17.13	10.29	2.15
	Average Duration (days)	4.52	4.57	7.97	5.95	9.41	5.75	2.56	2.18
Region flooded in Existing conditions only	Area (km2)	3.58	41.75	31.46	53.58	35.31	60.27	1.79	3.92
	Average Duration (days)	2.64	8.09	4.65	10.46	6.83	10.69	3.14	2.12
Region flooded in both conditions	Area (km2)	17.33	44.18	107.11	109.98	158.26	139.75	9.77	9.97
	Average Duration Natural conditions (days)	13.67	9.36	13.86	13.27	15.11	14.68	11.19	7.89
	Average Duration existing conditions (days)	13.06	9.91	10.13	13.93	9.72	14.07	11.35	7.72
	Difference(days)	-0.61	0.55	-3.73	0.66	-5.38	-0.61	0.15	-0.16

Table 26 - Comparison of regions flooded in Natural and Existing Conditions

For the summer event, most of the surface areas flooded in both conditions, are located within the main channels of the various rivers creeks and coulees.

Plates 74, 77, 80 and 83 provide a visual representation of the three regions for the four events.

10.10 Scenario 17: Removal of County Road 55

Plates 86 shows the flood extent for this scenario for the 1:10 event, and Plate 87 shows the change in peak levels between this scenario and the existing conditions for the 1:10 event (Plate 3).

The elevated CR 55 forces water that would naturally travel towards Rosebud and Louden Coulees, to move in an east-west direction towards the Red River. Removal of CR 55 allows the water to travel south-east towards Rosebud and Louden Coulees. As shown on Plate 87, removal of CR 55 significantly lowers water levels on the north side of the road from 5 cm to over 1 m (2 inches to over 3 feet). This water now progresses towards Louden, Rosebud and County ditch 42, which seems to be able to convey this extra flow. Rosebud Coulee, on the other hand, experiences additional overbank flooding as the waters make their way to the Tongue River.

Plate 87 shows that removing CR 55 lowers flood levels along the border only for about a mile (1.6 km) distance along the border, east of Border Crossing 6. CR 55 also significantly alters the distribution of water within the US portion of the model domain, channelling water in an east-west direction north of the road rather than allowing it to flow in a southeast direction.

The same observations are seen for the 1:50 event (Plates 88, 89) and the 1:100 events. (Plates 90, 91) where water on the north side is now released and is allowed to flow south into County ditch 42.

There is a significant difference with the 1:50 and 1:100 events: On the west side of the model, CR55 is located north of Pembina River instead of south. It will therefore affect the breakouts on the left bank of the river. For these two stronger events, the break out on the north side of Pembina River, which was contained by CR55, is now free to progress toward the border and into Buffalo Creek, causing strong flooding along its banks. At the same time since more water is released into Buffalo Creek, there is a reduction of flow into Aux Marais, as seen on Plates 89 and 91.

10.11 Scenario 18: Removal of County Road 18

CR18 is a major obstruction to the passage of flows in an east-west direction. The road's culvert and boxes do not have enough capacity to carry the flow during the 1:10 event; in the model they had a total capacity of 44.4 m^3/s (1,568 cfs), which included the box passage over Louden Coulee. (The passage under CR18 for the Rosebud Coulee is not included in this total flow capacity. Water is therefore stored on the west side of the road until the road is overtopped. The corresponding flood extent is shown on Plate 92. This extent is very similar to the existing conditions as seen on Plate 93, which shows only a local relief of flooding on the west side at Neche, with no significant new flooding on the east side, and a decrease in the local flooding in the south-west corner, between the border and CR 18. This indicates that the 2 large culverts under CR18 at the border are not discharging enough water

10.12 Scenario 19: Five diversions

In this scenario water is be diverted from the Pembina River main channel onto five neighbouring drainages. These diversions can have various capacities depending on the local slope of the land, the size of the diversion channels or the capacity of the receiving channel. To be able to compare with the other scenarios, the same four diversions capacities as in scenario 14 (Floodway), were simulated: 50, 142, 190 and 210 m^3/s (1770, 5000, 6710 and 7420 cfs). Plate 94 shows the location of these diversions while Table 27 indicates the individual flow distribution towards each of the channels. All diversions had their inlet upstream of Neche.

In all cases, as explained in Section 10.9, the total diverted flows were distributed so that 53% would be diverted north (into Buffalo and Aux Marais) and 47% south (into Rosebud and Louden). In the model these diversions were not represented physically using a numerical grid, but rather they were modelled as a sink-source combination with the water removed from the Pembina River and reintroduced into the other channels.

	1:10 event				**Summer event**
Total diversion capacity	**210 m^3/s**	**190 m^3/s**	**142 m^3/s**	**50 m^3/s**	**50 m^3/s**
Aux Marais River (m^3/s)	20	18.1	13.5	4.7	4.7
Buffalo Creek west (m^3/s)	48	43.4	32.4	11.4	11.4
Buffalo Creek east (m^3/s)	43	39	29.1	10.2	10.2
Rosebud Coulee (m^3/s)	49.5	44.8	33.5	11.8	11.8
Louden Coulee (m^3/s)	49.5	44.8	33.5	11.8	11.8

Table 27 – Flow diversion capacities simulated with the 1:10 and the summer events

The peak discharges estimated during the simulations are shown on Table 28. They are different from Table *27* because they include the local runoff, and the fact that the water, if the channel overflows, may be stored in the field or behind roads. This is accentuated for Louden that travels under several roads through large culverts or boxes which restrict the flow capacity of the coulee. Because of this, the maximum flow in Louden drainage was less than 38 m^3/s even though the amount of diverted water into it was larger.

	1:10 event					Summer event
Diversion capacity (m^3/s)	**Existing**	**210**	**190**	**142**	**50**	**50**
Main Pembina River channel, 8 km downstream from Neche	147	106	118	134	143	110
Aux Marais River (m^3/s)	10	19	17	20	14	4.7
Aux Marais River with Red River runoff (m^3/s)	19	20	20	20	18	8.5
Buffalo Creek west (m^3/s)	0	48	43	32	12	12
Buffalo Creek east (m^3/s)	30	47	44	40	40	15
Rosebud Coulee (m^3/s)	33	65	59	46	39	18
Louden Coulee (m^3/s)	34	38	35	33	36	6.5

Table 28 - Scenario 19 - Peak discharges in the channels going north and south of Pembina River

Flow diversions will have quite different effects in various parts of the flood plains as seen on Figure 60.

The graph A of this figure shows the water depth on the US side of border Crossing 6. It indicates not only a reduction in flooding but also a significant reduction in the duration of the flood. This graph is very similar to what was found in the case of the floodway (Figure 55).

The graph B is an indication of what could happen in the Aux Marais River, downstream from the diversions - its location is shown on Plate 99. Adding the diversion flows into the Aux Marais did not change significantly the duration of the inundation but increased its water depth.

The graph C represents the depth just south of Neche (see Plate 99) that is inundated by the extra water diverted into Louden Coulee. At this location flood depth is similar to the existing conditions, but its duration is much longer. This is due to the culverts and bridge restrictions in this Coulee.

It should be noted that these results should not be generalized as these three graphs show what would happen at three particular locations. Other locations may behave differently and would be influenced by the local land configuration, such as the presence of a road, bridge, culvert, small depression in the ground which may store water, or the local conveyance of the river or creek.

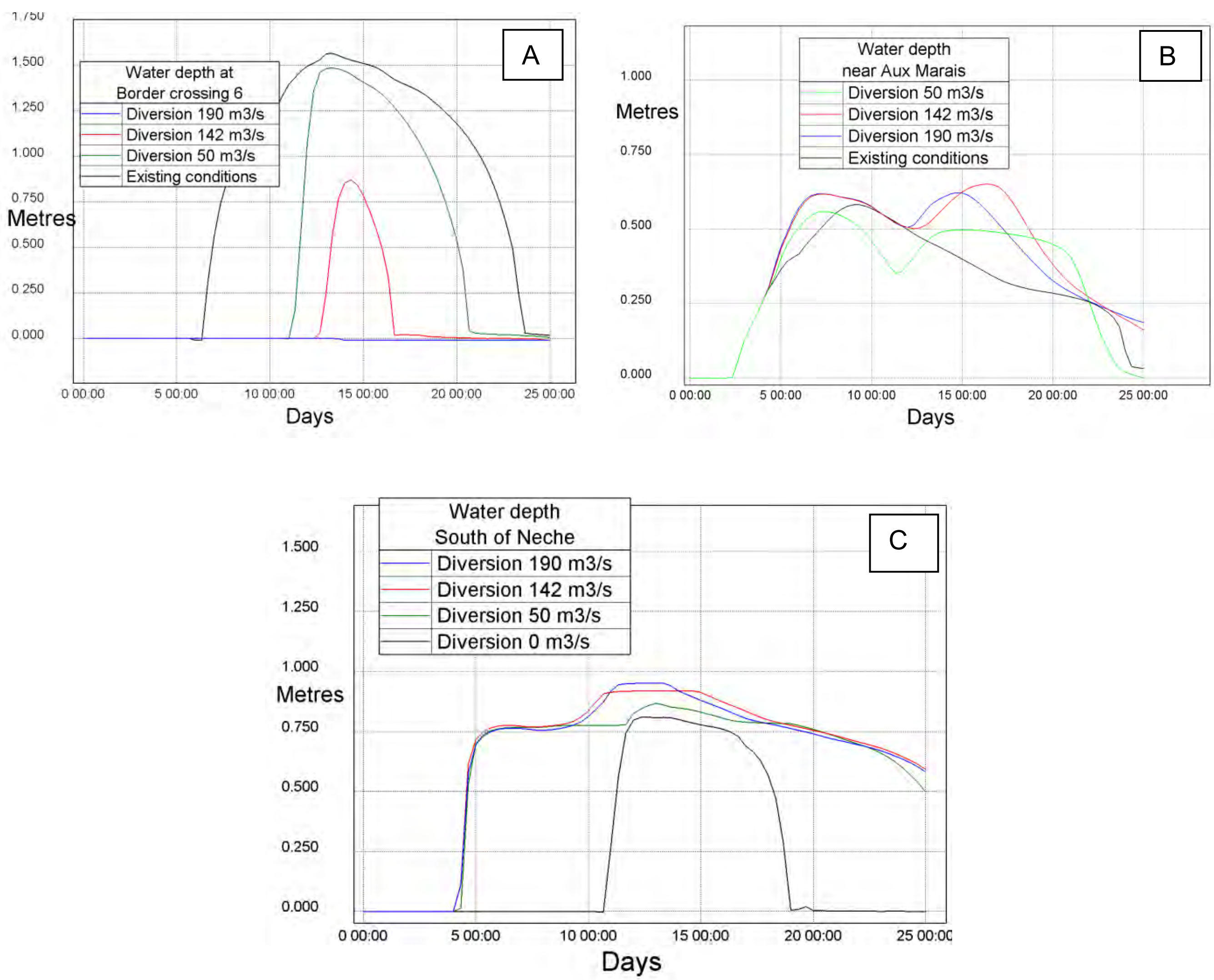


Figure 60 – Scenario 19 - Comparison of flood duration at three locations within the flood plains, for various diversion capacities

Contrary to the floodway where all the water was removed from Pembina River and diverted into one single location, in scenario 19 the water was spread over the Pembina flood plain, therefore resembling a "natural" condition. Plates 94 to 106 show the flood extent during the diversions, and indicate not only the comparison with existing conditions but also the comparison with natural conditions.

The analysis of these Plates, in general, shows that:

- In many cases flooding occurs upstream from a bridge or culverts and water would spread along its road (Figure 61). No attempt was made to increase the size of these flow restrictions.
- 210 m^3/s (7420 cfs) is needed to avoid any overbank flows of the Pembina River for the 1:10 event.

- 142 m^3/s (5,000 cfs) allows only minor flooding along the border. It creates some local flooding along Buffalo, Aux Marais, Rosebud and Louden than with the existing conditions (Plate 100).
- Larger diversions will provide less flood accumulation along the border,
- Larger diversions will provide less flood accumulation in the land between the Pembina River and the diversion outlets
- The west branch of Buffalo Creek can absorb most of the extra flow with some major local flooding
- For the east branch of Buffalo Creek, the bridge under PR 243 does not have enough capacity and water runs along the road and cannot go back to the main channel causing flooding close to Gretna. (Plate 97)
- The Aux Marais diversion causes extra flooding on the upper reach of the river but not as it approaches HWY 75 (Plate 100)
- With the 142 m^3/s (5,000 cfs) there is a reduction of 20.5 km^2 (8 mi^2) for the flood extent in the US (from 49.3 to 28.8 km^2) with a reduction for the flood-duration factor of 155.2 km^2-days from existing conditions, mainly due to the disappearance of flood along the border, along CR55 and east of Switzer ridge. (Plates 99 and 100)
- With the same 142 m^3/s the flood extent in Canada was increased by 11.4 km^2 (from 9.4 to 20.8 km^2) with an increase of the flood-duration factor of 170 km^2-days
- When compared to the natural conditions, (Plate 101) the 142 m^3/s provides less flood damage in both Canada and US (flood-duration factor reduction of 279 and 35.9 km^2-days, respectively) partly due to the fact that the land between Pembina River and the two channels, Buffalo and Louden, is not flooded.
- The small diversion of 50 m^3/s is enough to lower the peak level from existing conditions between CR 55 and the border from a few centimetres (1 inch) to 15 cm (6 inches) (Plate 103)
- With the 50 m^3/s capacity diversion, reduction in flood extent from existing conditions in the US is 7.5 km^2 (from 49.3 to 41.8 km^2), accompanied by a similar increase in Canada of 4 km^2 (from 9.4 to 13.4 km^2). In this case Switzer ridge is overtopped and the area east of the ridge is still largely flooded.
- For the summer conditions as compared with existing conditions, a small diversion of 50 m^3/s (1,770 cfs) is sufficient to prevent flooding along the border, at the expense of increased local flooding along Louden and Rosebud. (Plates 105 and 106).
- In all cases when compared to existing conditions, the flood-duration factor increases on the Canadian side and decreases on the US side.

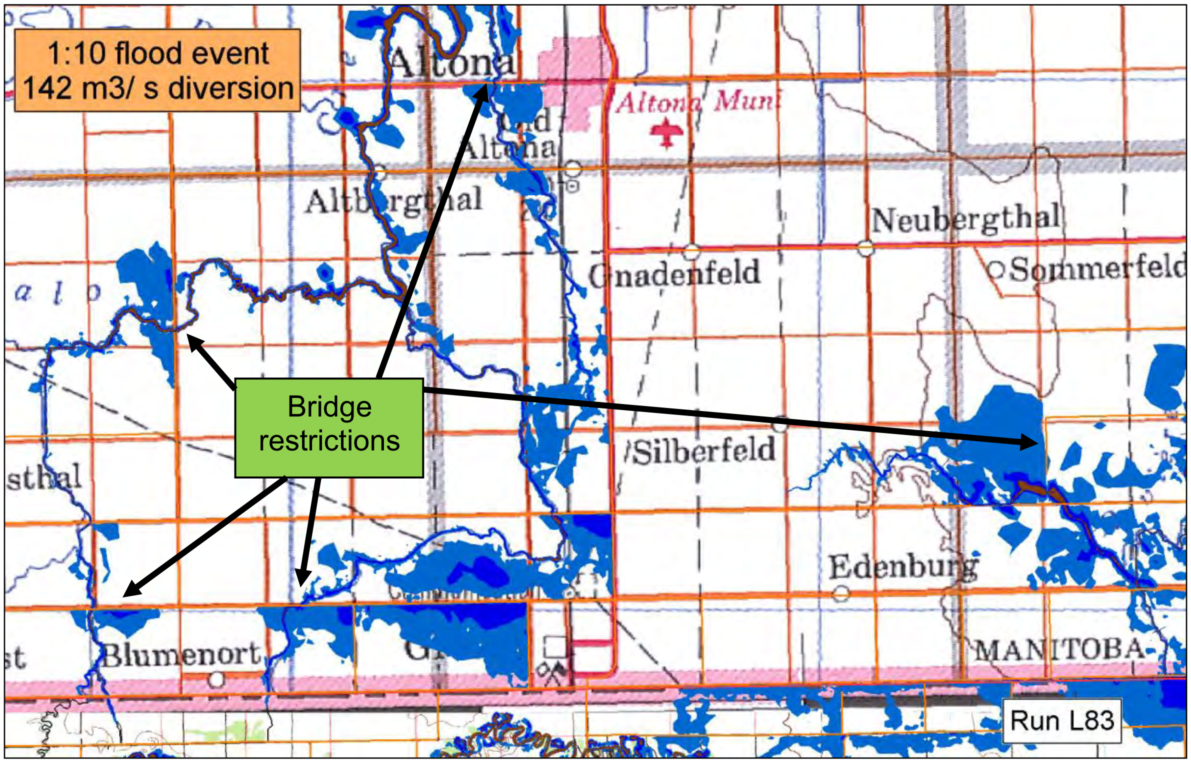


Figure 61 - Scenario 19 – Some bridges on Buffalo and Aux Marais causing flow restrictions during the diversions

10.13 Scenario 20: Storage of water at Walhalla

In order to reduce flood damage within the Red River basin during extreme spring events, it has been suggested that water could be stored upstream on the major tributaries of the Red River, and released later, therefore reducing the peak flow on the main stem of the Red River [Ref. 3]. The aim was to reduce the 1997 peak flow on the Red River by 20 %.

In this scheme, the Pembina River would have the 1997 peak flow reduced by 13% or 54 m^3/s (1900 cfs) corresponding to a volume reduction of 9% or 63 hm^3 (51,113 acre-feet). This volume would be stored upstream of Neche and released when the peak on the Red has passed.

In scenario 20, this strategy was simulated by assuming that the same amount of water 63 hm^3 (51,113 acre-feet), could be stored in the spring at or upstream of Walhalla. The peak of the hydrographs for the 1:10 and 1:50 events were truncated to values listed on Table 29, so that the corresponding volume of water would be reduced by approximately 63 hm^3. In this scenario the flow on the Red River was not reduced; the same hydrograph as in the existing conditions was kept because of the uncertainty in setting the corresponding levels downstream at Morris, and the fact that by not changing the flows in the Red, the effect of peak reduction at Walhalla would be assessed independently without the interference from the Red River.

	1:10 event	1:50 event
Existing peak flow (m^3/s)	317	700
Reduced peak flow (m^3/s)	140	400
Volume stored in Walhalla (hm^3)	63.00	60.65

Table 29 – Scenario 20 - Peak flow reduction at Walhalla

Plates 107 and 108 show that storage reduces the flooding from the 1:10 event to one that slightly exceeds the downstream capacity of Pembina River below Neche. Therefore, this scenario provides only minor flooding along the border. Local flooding along Buffalo, Rosebud and Aux Marais remained identical, since their local runoff hydrographs were not altered. Levels along Red River dropped by 10 to 15 cm (4 to 6 inches) compared to existing conditions. Upstream storage has strong reductions in flood extent and flood-duration factors, very similar to the floodway scenario having a capacity of 142 m^3/s (5,000 cfs).

In the case of the 1:50 event (Plates 109 and 110), limiting the flow at 400 m^3/s (14,126 cfs) still greatly exceeds the capacity of Pembina resulting in much flooding over the entire basin. Levels along Red River dropped by less than 5 cm (2 inches) compared to existing conditions (assuming the levels at Morris remained identical to existing conditions). A comparison of Plates 109 (storage scenario) with Plate 4 (1:50 existing conditions) indicates that storage greatly reduces the aerial extent of flooding.

11. Scenarios without local runoff

All the above scenarios were run with full hydrographs along the Pembina and the local drainages. It has been assumed that these hydrographs would simultaneously present the same return period of occurrence, which rendered the simulations very conservative. In 2006 and 2009 it was reported that there was only minor local runoff, even though these floods were somewhere between a 1:10 and 1:50 event. In scenario 19 (five diversions), diverted water was discharged into channels that had already water in them, since it was assumed that they would peak five days before the peak at Walhalla, or about six days before Pembina starts to overtop its banks in its lower floodplain.

Scenarios without local runoff would provide information on the flooding from Pembina River breakouts only, and help in assessing in the required conveyance of the channel in case of artificial diversions.

In the subsequent simulations the local rivers

- Aux Marais River,
- Buffalo Creek,
- Louden Coulee and
- Rosebud Coulee

were assumed dry at the time when Pembina started to overflow.

11.1 Existing conditions – no local runoff

There is a net decrease in local flooding (Plate 111 and 112). The small inundation along the Aux Marais comes from border Crossing 6 which discharges through its four large culverts. Rosebud Coulee is still overflowing at its usual location between CR 18 and the confluence with the Tongue River.

On the Canadian side, flood extent has decreased by about half (from 9.4 to 4.9 km^2), but remained similar on the US side (from 49.3 to 46.8 km^2). This implies that for the 1:10 year event, flooding in Manitoba can be primarily attributed to the local runoff, but in North Dakota attributed to the Pembina overflows and not to locally produced runoff, except for the Rosebud Coulee.

11.2 Scenario 13 – Removal of Portions of Boarder Road with no local runoff

Removing two short sections along the border road provides very similar flooding both north and south of the border (Plate 113 and 114), except along the Rosebud which does not breakout just before meeting the Tongue (compare Plates 113 with 20). A comparison of Plates 113, 20 and 3 show that removal of the two sections of the road causes increased flood extent along the Aux Marais and significant decreased of flood extent east of Border Crossing 6. Some flooding downstream of the Rosebud is attributed to local runoff.

11.3 Scenario 15 – Set-back Dykes with no local runoff

One of the drawbacks in raising dykes along both Pembina River and Tongue River was the inability to evacuate the local runoff that was trapped behind the dykes and could not reach the Tongue. This can be seen in this scenario by comparing Plates 62 with 115 where there is no pooling of water at the confluence between the Louden and Tongue. The local flooding along the Aux Marais which is visible on Plate 62, does not appear on Plate 115.

11.4 Scenario 19 – Five Diversions with no local runoff

Diversions with a total capacity of 142 m^3/s (5,000 cfs) for the 1:10 event were simulated assuming the local rivers would be empty. In this case the flood extent (Plate 117) is much reduced compared to flooding when local runoff is considered (Plate 99), particularly west of Gretna and downstream of Rosebud Coulee. The difference between these two cases is visualized on Plate 119 which shows the change in maximum depth when the runoff in the local rivers are considered. The assumption of diverting some of the Pembina flows into empty channels has decreased the flood-duration factor by 102 and 81 km^2-days in Canada and the US respectively.

Diverting only 50 m^3/s (1770) cfs) (Plates 120 and 121) into empty channels still shows local flooding along Aux Marais River, Rosebud Coulee and upstream of Louden Coulee. When compared with the same diversion into channels with their local runoffs, the flood-duration factor did not change in Canada, but was lowered from 475.9 to 407.2 km^2-days.

11.5 Scenario 16 – Natural Conditions with no local runoff

Plate 124 indicates the strong effect from local runoff. The green shade areas are the zones which would be inundated if runoff was considered. It shows the difference between Plate 122 and Plate 72. Of particular interest is the flooding around Rosenfeld (shown on Figure 1) which is protected by the Buffalo dykes against local runoff.

12. Conclusions

- During flood events roads have a huge temporal and spatial impact on the distribution of water within the lower Pembina River flood plains. The relationship between the lower Pembina River basin and adjacent basins, including the Red River, is quite complex. As presented in this report, the Phase 3 TELEMAC model does an exceptional job at replicating historic events, and as a result, is a useful tool for assessing the impacts of various alternative mitigation measures that are structural in nature. Such measures include evaluating the impacts on the landscape of:

    - the removal of one of more roads that act as barriers to the flow of water;
    - the upstream storage of water that reduces the magnitude of the flood peak on the lower Pembina River;
    - a floodway having various conveyance capacities;
    - multiple diversions of water from the Pembina River to various other stream systems;
    - set-back dykes along both sides of Pembina Rive to contain its spreading, and;
    - breaching the border road at two locations.

- For the 10-year event, the only location that County Road (CR) 55 impacts the level of flood waters along the Border is for about one mile (1.6 km), east of the Border Road Crossing 6 (Plate 87). It does not appear that there would be any change north of the border at this site. This location would experience reduced water levels by about 5 cm during the 10-year flooding if CR 55 did not impair the movement of water towards south-east (while all other roads remain in place).
- County Road 55 runs in an east-west direction. It crosses Pembina River 9.6 km (6 miles) west of Neche, and then runs north of the River instead of south. It therefore affects the breakouts on the left bank of the river as they move north. For the 50-year and 100-year events, if CR55 is removed, the break outs on the north side of Pembina River, which was contained by CR55, is now free to progress toward the border and into Buffalo Creek, causing strong flooding along the creek. At the same time since more water is released into Buffalo Creek, there is a reduction of flow into Aux Marais, as seen on (Plate 89).

- CR 55 also significantly alters the distribution of water within the US portion of the model domain, channeling water in an east-west direction north of the road rather than allowing it to flow in a south-eastward direction. It was overtopped during large recent flood events. Plate 87 shows that CR 55 increases water levels north of CR 55 and lowers levels south of it. Conversely, a large area north of the road and south of the Pembina River would generally experience reduced flood levels during flooding while south of the road would experience increased water levels during flooding if CR 55 did not impair the movement of water in a north-south direction. The widening of the existing bridges and culverts, or the creation of new ones under CR55 (east and west of CR18), would reduce flooding on the north side of CR55. This must be undertaken under controlled situation where water is guided downstream into channels with sufficient capacity such as the downstream portions of Louden and Rosebud Coulee, and County ditch 42.

- The Border Road reduces flooding in Canada from waters overflowing the banks of the lower Pembina River in the United States, while causing more flooding in the U.S.. (Plate 3 and 7)

- If the Border Road did not impair the south-north movement of water, there would be significant flood reduction in the United States while at the same time significant increased flooding in Canada, which would be closer to the conditions that would occur with natural conditions. Natural receiving streams in Manitoba are the east and west branch of Buffalo Creek and Aux Marais River (Plate 72, 75, 78 and Table 23). It is to be noted that the model was prepared with four east-west roads which impeded the south-north natural flow direction: the road 1/2 mile south of the border, the border road itself, PR243 located 1 mile north of the border, and the next road 1 mile north. If only the border road was removed, the flood water would still be directed by the other east-west roads towards Aux Marais (Plate 10), even though Aux Marais is not the main natural drainage (Plate 75). With the border road removed, peak flow in Aux Marais (42 m^3/s, for the 1:10 event) would be higher than in natural conditions (29 m^3/s) and than in existing conditions (10 m^3/s)

- The removal of the border road must be accompanied by the modification of the other east-west roads (widening of the existing bridges and culverts, creation of new ones), to create passages (along with new drains and channels) which would guide water towards its natural drainage: mostly Buffalo Creek and to a lesser extent Aux Marais River.

- Upstream basin storage (above Walhalla), as promoted by the Red River Basin Commission to reduce the Red River main stem 1997 peak flows by 20%, would have broad benefits to the lower Pembina flood plain. It is evident from a comparison of Plate 107 that upstream storage reduces overall flooding more than what occurs under either existing conditions (Plate 3) or natural conditions (Plate 72), for the 10-year event. Upstream storage also has dramatic reductions in flood extent and flood duration factors similar to the floodway scenario having a capacity of 142 m^3/s (5,000 cfs) (Plate 34). The timing and quantity of the water placed in storage was assumed to be nearly perfect, taking the entire volume off the highest portion of the flood hydrograph even for the small 1:10 event.

- Floodways having seven capacities between 50 m^3/s (1,770 cfs) and 210 m^3/s (7,420 cfs) were analyzed for the 1:10 event, while the 1:50 event was run for capacities between 74 m^3/s (2,610 cfs) and 210 m^3/s (7,420cfs). Results on the inundated area and flood duration factors are provided below for the 1:10 event (termed metrics) and illustrate the effect of the various floodway capacities on them. A shorter floodway having its intake at Crossing 6 is also shown. It should be noted that there is associated reductions in aerial extent of flooding and flood reduction factors not captured below due to the two zones wherein these metrics were calculated. There are reductions in these metrics outside the zones evident in the various Plates contained in this report. It is evident from the numerical results below that for the 1:10 event, within the American portion, there are decreasing reductions in the metrics as the size of the floodway increases (Plate 27, 29, 31, 33, 35, 37, 39), with minor changes on the Canadian side. Any significant increase in capacity would provide a level of protection that would have occurred without roads impairing the movement of water for the 1:10 magnitude design event. Larger capacities would also provide protection for events exceeding the 1:10 level and would have benefits associated with the additional capacity. Results from the 50 m^3/s (1,770 cfs) floodway capacity provide metrics that are very close to what would have occurred under natural conditions within the American portion of the analyzed area (Plate 45, 47, 49, 51).

Run	Description all 1:10 event	Inundated area (km^2)		Flood-duration factor (km^2-days)	
		Canada	US	Canada	US
L74	existing conditions	9.4	49.3	157.2	534.6
L73	Natural conditions	38.5	35.4	606.2	415.4
L87	Floodway 50 m3/s	9.4	37.2	143.8	375.2
L121	Floodway 74 m3/s	9.4	30.5	142.1	322.3
L122	Floodway 96 m3/s	9.4	23.7	140.7	271.6
L123	Floodway 119 m3/s	9.4	18.0	139.2	218.8
L75	Floodway 142 m3/s	9.4	14.1	135.9	182.1
L80	Floodway 190 m3/s	9.4	12.0	130.9	155.4
L81	Floodway 210 m3/s	9.4	12.0	131.1	151.7
L128	short Floodway 50 m3/s	9.4	40.2	145.9	409.2
L129	short Floodway 67 m3/s	9.4	39.7	142.2	405.2

- For events 1:50 and larger, the floodway does not reduce significantly the extent of flooding coming from Red River overflow, East of Switzer Ridge and at the Tongue confluence, but it reduces their depth, and their duration.
- In the conditions tested , there is a local increase in water level of up to 24 cm (9.4 inches) on Red River at the floodway outlet, but no increase in the peak levels on the Red for the 1:10 event, and minor peak level increase (2 to 4 cm) for the 1:50 event. But this effect is very sensitive to the timing of the peaks between Red River and Pembina River.

- A shorter floodway starting at Border Crossing 6 with a capacity of only 67 m^3/s (2370 cfs), during 1:10 event, would prevent any overtopping of Switzer Ridge and the subsequent filling from Pembina River of the area east of the ridge. A capacity of 181 m^3/s (6390 cfs) would be required for the 1:50 event. Much work is required to determine the most effective floodway route and the required capacity. As well, the capacity of the floodway could potentially decrease in an upstream direction if branching into it were included in the design.

- The diversion of some of Pembina River flow into five adjacent channels provides a scenario closest to natural conditions where water is allowed to flow according to natural slope of the land, in both northern and southern directions, instead of one artificially set direction (such as set-back dykes or floodway). It would provide additional flow capacity to the existing flood relief infrastructures such as border crossings 4, 5 and 6, and county ditch 42. The diversions being independent from each other could be designed according to the planned capacity of the receiving channels. Their implementation could be gradual, compared to the floodway or set-back dykes which must be fully completed before being operational. The model was run with one location for the diversion intake and outlet. Other locations, which may require less infrastructure modification (bridges) or channel improvement (where the model identified insufficient conveyance), should be investigated.

- There are issues around the mitigation approach of such diversions. They relate to the already estimated significant local flows that can occur during flood events along the receiving waterways, their timing, existing channel capacities and existing infrastructure (e.g. bridge restrictions, homes and farmyard locations). Cost of increasing channel capacities and undertaking bridge enhancements or extra culvert installations, should be undertaken on a one to one basis; Buffalo Creek has more capacity than Aux Marais, and some of the impeding bridges identified by the model may require only minor modifications. One must also consider the impact that channel enlargement (deepening or widening or simply clearing), would have on the environment. A constructed channel adjacent to the coulees, rivers and creeks, or additional drains, may reduce some of the environmental concerns, but may still have several of the other issues.

- Set-back dykes may be an absolute mitigation alternative. It prevents any flooding for a given design event. Right-of-way and compensation would be issues to consider (there are many farmsteads located close to the river edge). With the south side of the dyke along Pembina River ending at the confluence with the Tongue River, there is a dramatic reduction in both inundation area and flood-duration factor for both American and Canadian portions. The increase in water level is localized at the confluence with Tongue River and is minimal on the Pembina community dykes. This increase is sensitive to the timing of the peaks between Pembina River and Red River. (For an event similar to the 2006 flood, an increase along the Red River of up to 12 cm (4.7 inches) would be observed). This downstream impact must be mitigated for this option to be further considered, such as raising the dykes around the communities.

- Breaching the Border Road at the two identified locations, one located west of Gretna and the other at Border Road Crossing 6, is seen as only a partial solution. It must be accompanied by infrastructure enhancements such as the creation of channels which would guide water towards Buffalo and Aux Marais, bridge widening and farm yard protection. The scenario with diversions would be a better choice of mitigation.

13. Acknowledgment

We would like to thank the members of the International Red River Board and the Lower Pembina River Flood Task Team, for all their valuable comments and suggestions during the course of this project.

Also we would like to recognize the extremely valuable help from two CHC colleagues, Dan Pelletier and Martin Serrer, without whom this project would have been very difficult, if not virtually impossible.

Dan always made sure that the cluster of computers was in excellent working order - even during a power outage – and kept updating them, ensuring the processors were always the latest on the market. Martin, who developed Blue Kenue, kept adding new features to this software each and every time the need occurred, greatly facilitating the preparation of the model, its visualization and its analysis.

14. List of Simulated Runs with Runoff

The lists of runs simulated during this project are presented here and in the next section, as a reference. They include more runs than the ones mentioned in Table 18 and Table 19, and mention the files describing the topography (geometry file). All the output files of the Telemac model have been provided to IJC and these tables will help in identifying the various conditions.

Scenario	Run	Geometry file	Event	Description
	L40 L56	Geo L15 Geo L22	2006 flood	Calibration 2006 conditions Verification 2006 with latest L22 grid - see L56 culvert
	L42 L55	Geo L14 Geo L21	2009 flood	Verification existing conditions
	L109	Geo L29	2011 flood	Verification existing conditions
12	L33	Geo L11	2006 flood	Remove border road + road ½ mile south of border
15	L34	Geo L10	2006 flood	Set back dykes 400 m wide
13	L98	Geo L27	2006	Remove portion border road West of Gretna + road ½ mile south of border + portion road 1 mile north border at Gretna Remove portion border road Xing 6 + road 1 mile south of border East of Neche
16	L100	Geo L23	2006	Natural conditions no roads
Existing conditions	L74	Geo L21-3	1:10 year	With two extra runoff in Neche to Pembina + Tongue Peak inflow on Red:1347 m3/s; Peak outflow Morris:1733 m3/s Peak Walhalla:304; Peak Walhalla hourly 317 Peak level Morris:236.42
Existing conditions	L116	Geo L31	1:10 year	Add two short roads identified by MIT
Existing conditions	L44P	Geo L21-2	1:50 year	Peak inflow on Red:2381 m3/s; Peak outflow Morris:3098 Peak Walhalla:665; Peak Walhalla hourly :699 Peak level Morris:238.55
Existing conditions	L124	Geo L31	1:50 year	Add two short roads identified by MIT
Existing	L46Q	Geo L21-3	1:100 year	Peak inflow on Red:2909 m3/s; Peak outflow Morris:3847

conditions				m3/s Peak Walhalla:864; Peak Walhalla hourly :908 Peak level Morris:238.72
Existing conditions	L95	Geo L21-3	Summer 1:20	Constant 1000 m^3/s in Red, constant level 236.3 at Morris
12	L67	Geo L21-3 sce12	1:10 year	Remove border road + road ½ mile south of border
12	L68	Geo L21-3 sce12	1:50 year	Remove border road + road ½ mile south of border
12	L119	Geo L21-3 sce12	1:100 year	Remove border road + road ½ mile south of border
13	L102	Geo L27	1:10 year	Remove portion border road West of Gretna + road ½ mile south of border + portion road 1 mile north border at Gretna Remove portion border road Xing 6 + road 1 mile south of border East of Neche
13	L96	Geo L27	Summer 1:20	Remove portion border road West of Gretna + road ½ mile south of border + portion road 1 mile north border at Gretna Remove portion border road Xing 6 + road 1 mile south of border East of Neche Constant 1000 m^3/s in Red, constant level 236.3 at Morris
14	L87	Geo L21-3	1:10 year	Floodway 50 m3/s
14	L75	Geo L21-3	1:10 year	Floodway 142 m3/s
14	L81	Geo L21-3	1:10 year	Floodway 210 m3/s
14	L80	Geo L21-3	1:10 year	Floodway 190 m3/s
14	L121	Geo L21-3	1:10 year	Floodway 74 m3/s
14	L122	Geo L21-3	1:10 year	Floodway 96 m3/s
14	L123	Geo L21-3	1:10 year	Floodway 119 m3/s
14A	L128	Geo L32	1:10 year	Short Floodway from Crossing 6: 50 m3/s
14A	L129	Geo L32	1:10 year	Short Floodway from Crossing 6: 67 m3/s

14	L84	Geo L21-3	1:50 year	Floodway 210 m3/s
14	L125	Geo L21-3	1:50 year	Floodway 74 m3/s
14	L126	Geo L21-3	1:50 year	Floodway 96 m3/s
14	L127	Geo L21-3	1:50 year	Floodway 119 m3/s
14A	L130	Geo L32	1:50 year	Short Floodway from Crossing 6: 50 m3/s
14A	L131	Geo L32	1:50 year	Short Floodway from Crossing 6: 74 m3/s
14A	L132	Geo L32	1:50 year	Short Floodway from Crossing 6: 96 m3/s
14A	L133	Geo L32	1:50 year	Short Floodway from Crossing 6: 142 m3/s
14A	L135	Geo L32	1:50 year	Short Floodway from Crossing 6: 181 m3/s
15	L70	Geo L21-3	1:10 year	Set back dykes 400 m
15	L71	Geo L21-3	1:50 year	Set back dykes 400 m
15A	L112	Geo L21-3	1:10 year	Set back dykes 400 m, Not along Tongue River
15A	L120	Geo L21-3	1:50 year	Set back dykes 400 m, Not along Tongue River
16	L69	Geo L23	1:100 year	Natural conditions no roads
16	L85	Geo L23	1:50 year	Natural conditions no roads
16	L73	Geo L23	1:10 year	Natural conditions no roads
16	L99	Geo L23	Summer 1:20	Natural conditions no roads
16	L113	Geo L23	1:10 year	Natural conditions no roads, no bridge piers along Pembina
16	L115	Geo L23	Summer 1:20	Natural conditions no roads, no bridge piers along Pembina
17	L72	GeoL24	1:10 year	Remove Hwy 55
17	L117	GeoL24	1:50 year	Remove Hwy 55
17	L118	GeoL24	1:100 year	Remove Hwy 55
18	L76	Geo L25	1:10 year	Existing without CR18
19	L78	Geo L21-3	1:10 year	Diversion 210 m3/s 53% 47%
19	L82	Geo L21-3	1:10 year	Diversion 190 m3/s 53% 47%
19	L83	Geo L21-3	1:10 year	Diversion 142 m3/s 53% 47%

19	L86	Geo L21-3	1:10 year	Diversion 50 m3/s 53% 47%
19	L97	Geo L21-3	Summer 1:20	Diversion 50 m3/s 53% 47%
20	L103	Geo L21-3	1:10 year	Storage at Walhalla
20	L104	Geo L21-3	1:50 year	Storage at Walhalla

15. List of Simulated Runs with no Runoff

Scenario	Run	Geometry file	Event	Description
Existing conditions	L92	Geo L21-3	1:10 year	Existing conditions with No runoff No Aux Marais Buffalo Louden Rosebud
13	L93	Geo L27	1:10 year	Remove portion border road West of Gretna + road ½ mile south of border + portion road 1 mile north border at Gretna Remove portion border road Xing 6 + road 1 mile south of border East of Neche No runoff No Aux Marais Buffalo Louden Rosebud
16	L94	Geo L23	1:10 year	Natural conditions no roads No runoff No Aux Marais Buffalo Louden Rosebud
19	L90	Geo L21-3	1:10 year	Diversion 142 m3/s 53% 47% No runoff No Aux Marais Buffalo Louden Rosebud
19	L89	Geo L21-3	1:10 year	Diversion 50 m3/s 53% 47% No runoff No Aux Marais Buffalo Louden Rosebud
15	L101	Geo L21-3	1:10 year	Setback dykes 400 m No runoff No Aux Marais Buffalo Louden Rosebud

16. References

[1] National Research Council of Canada, Canadian Hydraulics Centre (CHC), *Preparation of a two-dimensional Hydrodynamic Model of the Lower Pembina River Flood Plains,* Controlled Technical Report CHC-CTR-093; July 2009.

[2] National Research Council of Canada, Canadian Hydraulics Centre (CHC), *Simulation of Flood Scenarios on the Lower Pembina River Flood Plains with the 2D Hydrodynamic Model,* Controlled Technical Report CHC-CTR-106; June 2010.

[3] Charlie Anderson, *20% Flow Reduction Strategy Implementation Planning.* Paper presented at the RRBC Annual Conference, Fargo ND, January 2011; WSN Engineering

[4] Emerson, D. G.; Vecchia, A. V. and Dah, A. L. *Evaluation of Drainage-Area Ratio Method Used to Estimate Streamflow for the Red River of the North Basin, North Dakota and Minnesota* USGS, 2005

[5] United States Geological Survey (USGS), *Guidelines for Determining Flood Flow Frequency*, Bulletin 17B, USGS, 1981

[6] Maidment, D. *Handbook of Hydrology* McGraw-Hill New York, 1993

[7] Burkley, K *Log Pearson Flow Frequency Using USGS 17B* MATLAB, 2009

[8] Hosking, J. and Wallis, J. *Regional frequency analysis: an approach based on L-moments* Cambridge University Press, 1997

[9] Hosking, J. R. M., *L-moments*, R package, version 1.6, http://CRAN.R-project.org/package=lmom, 2012

[10] Venables, W. N. and Ripley, B. D.; *Modern Applied Statistics* with S Springer, 2002

[11] National Research Council Canada - Canadian Hydraulics Centre (CHC)*; Green Kenue Reference Manual*, Ottawa, Ontario, September 2010