Comparison of One-Dimensional and Two-Dimensional Hydrodynamic Modeling Approaches For Red River Basin

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Comparison of One-Dimensional and Two-Dimensional Hydrodynamic Modeling Approaches For Red River Basin

Final Report to

International Joint Commission
Commission mixte internationale

December 1999

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EXECUTIVE SUMMARY

A devastating flood in Red River valley in 1997 emphasized the need to study the flood control measures in the Red River basin using state of the art modeling tools. The Red River and its floodplains can be modeled using one-dimensional, quasi two-dimensional or fully two-dimensional hydrodynamic models. Each modeling approach has its own advantages and limitations. The main purpose of this report is a comparison between one-dimensional (or quasi two dimensional) and fully two-dimensional hydrodynamic modeling approaches for modeling floods in the Red River basin.

A two-dimensional hydrodynamic model, MIKE 21, coupled with Geographic Information System (GIS) has been used in this study to capture the hydraulic response of the Red River and its floodplains in extreme flooding conditions. The focus area for modeling is a section of the Red River basin from south of Winnipeg floodway to the town of Ste. Agathe in Manitoba, Canada. For comparison purposes, results produced by Klohn-Crippen (1999), using one-dimensional unsteady flow model, MIKE 11, have been used.

MIKE 21, two-dimensional unsteady-flow model, is based on the finite-difference solution scheme and uses rectangular grids to resolve bathymetry. Simulations are made, using MIKE 21, to study the impact on flooding by modifying the operation of floodway. Problems associated with the application of a 2-D model are discussed especially with respect to the representation of terrain.
Based on comparison of results, produced in this study using two-dimensional model MIKE 21 and reported by Klohn-Crippen (1999) using one-dimensional model MIKE 11, it is recommended that:

1. The whole Red River basin should be modeled using a combination of quasi two-dimensional and fully two-dimensional models.

2. Fully two-dimensional model should be used only for areas of interest where detailed description of flow field is required, for example in the vicinity of major structures, communities and sensitive areas with high flood damage potential.

Using combined modeling approach boundary conditions for two-dimensional model can be derived from results produced by one-dimensional model.
1. INTRODUCTION

1.1 Motivation

Occurrence of floods is a natural phenomenon all over the world and the Red River valley is no exception. With approximately 1.25 million inhabitants in the Red River basin in USA and Canada, the basin is highly productive agricultural area serving local, regional and international food needs. Several devastating floods have occurred in the valley during this century, causing damages to business and property worth millions of dollars. The latest flood event in 1997 clearly emphasized the need to study the flood control measures in the basin using state of the art modeling tools.

The Red River valley in Manitoba and North Dakota is very flat with direction of flow from south to north. The Red River is main drainage channel in the valley carrying water to Lake Winnipeg. When the capacity of the channel is exceeded, the river overtops the banks and floods a very large area including adjacent roads and railway embankments. The overland flow occurs over the floodplain area in a general south to north direction. The road and railway embankments in the floodplain obstruct the flow when perpendicular to the direction of flow. The overland flow is often prevented from returning to the main river by intervening roads and rail embankments. During 1997 flood the overland flooding was further complicated by sudden washout of roads and rail embankments and road cuts that were made by government personnel to reduce flood levels in some areas. Areas like Grand Forks, Pembina and Ste. Agathe were flooded during 1997 flood event. Due to the presence of complex
infrastructure and very flat floodplains, the task of numerical modeling to capture the overland flooding is very challenging in case of the Red River basin.

Most of the studies in the Red River basin for flood modeling and flood management, so far, have been done using either one-dimensional or quasi two-dimensional models. As the Red River valley is remarkably flat, one-dimensional modeling approach might not be the best way to simulate the progression of flood wave over a complex topography. Fully two-dimensional hydrodynamic modeling approach has not been used in the Red River basin so far for flood management. Main objective of this research is to explore the possibly of applying a fully 2-D hydrodynamic model and comparing the results with previous studies that used 1-D model. In this study a two dimensional hydrodynamic model has been used to capture the response of river and floodplains to an extreme hydrological event. The advantage of using the 2-D approach is that it provides information on variable velocities and depths at any point of interest in the model domain. The computation of velocity profiles in two dimensions provides a better prediction of the effects of river training, scouring and sediment transport processes.

Two-dimensional hydrodynamic modeling approach, using finite difference solution scheme, is presented in this report for modeling flows in a floodplain that is flat and have complex topographic features. Present study deals with simulation of flow in river and floodplains for flood management purposes.

1. Modeling approach to simulate lateral flows in the floodplain using 1-D model. Pre-defined paths (branches) are used to exchange lateral flow between river and floodplain at selected points.
This report outlines a general framework for modeling of river and floodplains using two-dimensional hydrodynamic modeling approach. Theoretical background of 1-D and 2-D modeling approaches is given. Data requirements and problems with data processing especially topographic data are discussed. The benefits of proposed approach are demonstrated by application to a case study in river environment. Details on modeling approach are given. Finally, a discussion on results is presented and conclusions are drawn. Results from 1-D and 2-D modeling approaches are compared. The suggestions for use of 1-D or 2-D models in the context of Red River conclude the report.
2. THEORETICAL BACKGROUND

Based on objectives of the study, available data, computational resources, accuracy requirement and real-time operational efficiency both 1-D (quasi-2-D) or fully 2-D modeling approaches may be used for hydrodynamic modeling. The theoretical bases of both 1-D and 2-D modeling approaches, their salient features, data requirements and limitations are discussed in this section.

2.1 1-D Hydrodynamic Modeling

The theory of 1-D hydrodynamic modeling is based on following assumptions:

1. The water is incompressible and homogeneous, i.e., without significant variation in density.
2. The bottom slope is small.
3. The water-lengths are large as compared to water-depths. This ensures that the flow everywhere can be regarded as having a direction parallel to the bottom, i.e., vertical accelerations can be neglected and a hydrostatic pressure variation along the vertical can be assumed.

The basic equations for 1-D hydrodynamic modeling are derived considering conservation of mass and momentum. Considering the hydraulic resistance and the lateral inflow, the equations can be written as:
\[
\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q
\]  \hspace{1cm} (1)

\[
\frac{\partial Q}{\partial x} + \frac{\partial}{\partial x} \left( \frac{\alpha Q^2}{A} \right) + gA \frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2AR^*} = 0
\]  \hspace{1cm} (2)

where

\[A = \text{Flow area (m}^2\text{)}\]
\[R^* = \text{Resistance radius (m)}\]
\[C = \text{Chezy’s Resistance coefficient (m}^{1/2}/\text{s)}\]
\[g = \text{Acceleration due to gravity (m/ s}^2\text{)}\]
\[h = \text{Stage above horizontal reference level (m)}\]
\[Q = \text{discharge (m}^3/\text{s)}\]
\[\alpha = \text{momentum distribution coefficient}\]
\[q = \text{Lateral inflow (m}^2/\text{s)}\]

In 1-D modeling approach simplified equations of continuity and momentum allow the use of large spatial resolution \((dx)\) thus making solution scheme more efficient.

The typical data requirements to setup a 1-D hydrodynamic model can be divided into two categories that are boundary conditions and topographic data. Series of discharges and water levels at upstream and downstream model boundaries are required to satisfy the
boundary conditions of the model. Cross sections of main river and floodplains are required to define the topographic setup of the model. Model output consists of water levels and discharges at each cross-section.

Results (water levels and discharges) can only be obtained at points where cross-section information is available. This requires, at model setup stage, strategic selection of areas of interest where results are required.

1-D models can provide discharges and velocities in one dimension only i.e., along the direction of flow. Using 1-D models it is not possible to track the exact path of flood wave propagation over floodplains with complex topographic features.

1-D model can not produce good simulation of floods in areas where lateral flow in and out of floodplains plays an important role in flood wave propagation. However, with quasi-2-D modeling approach lateral inflows and outflows can be modeled with reasonable accuracy. Quasi 2-D modeling approach requires pre-determined pathways, to be defined by the modeler, for flood wave propagation. That is a limitation especially in cases where main interest is in finding the exact path of flood wave and where multiple paths are possible (case of very flat area).

However, capacity to consider lateral flows using quasi 2-D modeling approach, ability to perform well with limited topographic data and time efficient solution algorithm makes 1-D modeling approach a favorable choice for real-time operational models.
2.1.1 MIKE 11

The tool used by Klohn-Crippen (1999) for 1-D hydrodynamic modeling is MIKE 11. The MIKE 11 is an implicit finite difference model for one dimension unsteady flow computation and can be applied to looped networks and quasi two-dimensional flow simulation on floodplains. The model has been designed to perform detailed modeling of rivers, including special treatment of floodplains, road overtopping, culverts, gate openings and weirs. An add-on geographic information system (GIS) module provides an interface for display of river modeling results for floodplain management.

MIKE 11 is capable of using kinematic, diffusive or fully dynamic, vertically integrated mass and momentum equations (the “Saint Venant” equations). The Solution of continuity and momentum equations is based on an implicit finite difference scheme. This scheme is structured so as to be independent of the wave description specified (i.e. Kinematic, Diffusive or Dynamic). For solution, a computational grid of alternating Q (discharge) and h (water level) points is used as illustrated in Figure 1. The Q points are placed mid way between neighboring h points and at structures, while h points are located at cross sections, or at equidistant intervals in between (if the distance between cross sections is greater than the space step used for hydrodynamic model computations dx-max).

Hydrodynamic model operates on the basis of information about the river and floodplain topography, including man-made flood control measures as embankments, dredging schemes and flood retention basins.
2.2 2-D Hydrodynamic Modeling

The following basic equations for the conservation of mass and momentum are used to describe the flow and water level variations in two-dimensional models:

(a) Continuity

\[
\frac{\partial \zeta}{\partial t} + \frac{\partial p}{\partial x} + \frac{\partial q}{\partial y} = 0
\]  

(b) X-Momentum

\[
\frac{\partial p}{\partial t} + \frac{\partial}{\partial x} \left( \frac{p^2}{h} \right) + \frac{\partial}{\partial y} \left( \frac{pq}{h} \right) + gh \frac{\partial \zeta}{\partial x} + \frac{gp\sqrt{p^2 + q^2}}{c^2 \cdot h^2} - \frac{1}{\rho_w} \left[ \frac{\partial}{\partial x} (h \tau_{xx}) + \frac{\partial}{\partial y} (h \tau_{xy}) \right]
\]

\[- \Omega q - \frac{f v v_x}{\rho_w} + \frac{h}{\rho_w} \frac{\partial}{\partial x} (p_n) = 0\]
(c) Y-Momentum

\[ \frac{\partial p}{\partial t} + \frac{\partial}{\partial y} \left( \frac{q^2}{h} \right) + \frac{\partial}{\partial x} \left( \frac{pq}{h} \right) + gh \frac{\partial \zeta}{\partial y} + \frac{gq}{c^2} \frac{p^2}{h^2} - \frac{1}{\rho_w} \left[ \frac{\partial}{\partial y} (h\tau_{yy}) + \frac{\partial}{\partial x} (h\tau_{xy}) \right] \]

\[ + \Omega q - f\nu_v + \frac{h}{\rho_w} \frac{\partial}{\partial y} (p_a) = 0 \]  

(5)

Where:

\( h(x,y,t) \) = water depth (m);
\( \zeta(x,y,t) \) = surface elevation (m);
\( p,q,(x,y,t) \) = flux densities in x and y-directions (m\(^3\)/s/m) = (uh, vh); (u, v) = depth averaged velocities in x- and y- directions;
\( c(x,y) \) = Chezy resistance (m\(^{1/2}\)/s);
\( g \) = acceleration due to gravity (m/s\(^2\));
\( f(v) \) = wind friction factor;
\( \Omega(x,y) \) = Coriolis parameter, latitude dependent (S\(^{-1}\));
\( P_a(x,y,t) \) = atmospheric pressure (kg/m/s\(^2\));
\( \rho_w \) = density of water (kg/m\(^3\));
\( x,y, \) = space coordinates (m);
\( t \) = time (s);
\( \tau_{xx}, \tau_{xy}, \tau_{yy} \) = components of effective shear stress;
\( V, V_x, V_y (x,y,t) \) = wind speed and components in x- and y- direction (m/s);
In very flat floodplains with complex topographic features due to the presence of infrastructure, flood wave propagation is not an one-dimensional phenomenon. To accurately capture the lateral flows, a two-dimensional modeling approach is required.

In 2-D modeling equations of continuity and momentum are written in two dimensions and results are calculated at each grid point in the solution domain. Thus only a fine spatial resolution (dx) can be used that makes computing slow and requires a lot of computer memory.

Contrary to 1-D modeling approach where results (water levels and discharges) can only be obtained at points where cross-section information is available, 2-D modeling results are available at every grid point in the solution domain. Moreover discharges and velocities are available in two dimensions i.e. along the flow and in lateral direction.

The main advantage of using the 2-D approach is that it provides information on variable discharges and velocities in both x and y direction at each grid point at each computational interval. The computation of velocity profiles in two dimensions allows the accurate representation of flood wave propagation and better prediction of the effects of river training, scouring and sediment transport processes. The assessment of impacts of any proposed change in the river such as dikes, training walls and dredging, begins by determining the local changes in velocities. Velocity predictions can also be used for navigation purposes. The information on distribution of average velocity at any section can also assist fisheries research and management. Thus a 2-D model can be used to asses the impact of proposed
changes, such as dredging and changes in channel configuration, and addition or removal of flood control structures.

Contrary to quasi-2-D approach in full two-dimensional modeling the path followed by flood wave is computed from the topographic information. Thus this approach provides correct description of the flood wave propagation over floodplains.

Water level accuracy may be improved by using a two-dimensional model that can better resolve bathymetry and flow features. Water levels are required primarily for flood forecast, floodway operation and river management. Water level information can be used to verify existing floodplain contingency preparations. Since the 2-D model provides an accurate flow field, it can serve as the basis of a contaminant model to assess the effects of predicted or present loading.

In terms of data the most important requirement for 2-D modeling of a river system is accurate description of topography and bathymetry of river and floodplains. Prediction of water levels depends heavily on accurate representation of floodplain. Other necessary data can be divided into three groups i.e. basic model parameters, calibration parameters, and boundary conditions. Basic model parameters include model grid size and extent, time step and length of simulation and type of output required and its frequency. Bed resistance and wind friction factors are required parameters for calibration. Hydrographic boundary conditions can be specified as a constant or variable (in time and space) level or flux at each open model boundary, as a constant or variable source or sink anywhere within the model, and
as an initial free surface level map applied over the entire model. Basic output of model is water surface elevation and flux densities in x- and y-directions. The derived output includes water particle velocity and flow direction. Output results are computed at each grid point for each time step.

Due to significant requirements of topographic data and computational time for two-dimensional modeling, one-dimensional approach was a preferred choice for modeling floods, especially in very large basins. However, with advances in topographic data capturing and processing techniques and advancement in parallel computing, two-dimensional modeling applications are increasing in river environments. A detailed literature review of 2-D model applications for flood management is reported in Ahmad and Simonovic (1999).

2.2.1 MIKE 21

The tool used for 2-D hydrodynamic modeling in this study is MIKE 21, developed by Danish Hydraulic Institute. MIKE 21 is a comprehensive modeling system for 2-D free-surface flows and can be used for the simulation of hydraulic and related phenomena in rivers, lakes, estuaries, and coastal areas where stratification can be neglected. Typical application areas are modeling of tidal hydraulics, wind and wave generated currents, storm surges, dam-break and flood waves.

The model simulates the water level variations and flows in response to a variety of forcing functions. The water levels and flows are resolved on a rectangular grid covering the area of interest when provided with the bathymetry, bed resistance coefficients, wind field,
and hydrographic boundary conditions. The modeling tool is capable of handling convective and cross momentum, bottom shear stress, wind shear stress at the surface, barometric pressure gradients and Coriolis forces. Momentum dispersion is handled using the Smagorinsky formulation. Different sources and sinks (mass and momentum) can be described in the model and the model is also capable of handling flooding and drying.

The modeling system solves the fully time-dependent non-linear equations of continuity and conservation of momentum. The solution is obtained using an Alternating Direction Implicit ADI finite difference scheme of second-order accuracy. The outcome of a simulation is the water level and fluxes in the computational domain. Model simulates unsteady 2-D flows in one layer (vertically homogeneous) fluids. The continuity and momentum equations are solved by implicit finite difference techniques with the variables defined on a space staggered rectangular grid as shown in Figure 2.

![Difference grid in x,y-space](image-url)
A 'fractioned-step' technique combined with an Alternating Direction Implicit (ADI) algorithm is used in the solution to avoid the necessity for iteration. Second order accuracy is ensured through the centering in time and space of all derivatives and coefficients. The ADI algorithm implies that at each time step a solution is first made in the x-momentum equations followed by a similar solution in the y-direction.

The equations are solved in one-dimensional sweeps, alternating between x and y directions (Figure 3). In the x-sweep the continuity and x-momentum equations are solved, taking \( \varsigma \) from \( n \) to \( n+1/2 \) and \( p \) from \( n \) to \( n+1 \). For the terms involving \( q \), the two levels of old, known values are used, i.e. \( n-1/2 \) and \( n+1/2 \). In the y-sweep the continuity and y-momentum equations are solved, taking \( \varsigma \) from \( n+1/2 \) to \( n+1 \). And \( q \) from \( n+1/2 \) to \( n+3/2 \), while terms in \( p \) use the values just calculated in the x-sweep at \( n \) and \( n+1 \). Adding these two sweeps together gives time centering at \( n+1/2 \) i.e. the time centering is given by a balance sequence of operations.

![Fig. 3 Time centering](image-url)
The application of the implicit finite difference scheme results in a tri-diagonal system of equations for each grid line in the model. The solution is obtained by inverting the tri-diagonal matrix using the Double Sweep algorithms, a very fast and accurate form of Gauss elimination.

2.3 Selecting a Model for Red River Basin

Both one-dimensional and two-dimensional hydrodynamic models can be used to simulate floods in the Red River basin. However, each modeling approach has its own advantages and limitations. Two most commonly used one-dimensional modeling tools are HEC-2 and MIKE 11. The main objective of HEC-2 program is to compute water surface elevation at locations of interest for a given flow value (Hydrologic Engineering Center, 1991). The basic assumptions underlying HEC-2 are that flow is steady, flow is gradually varied, flow is one-dimensional i.e. velocity component in direction other that the direction of flow is not accounted far and river channel has small slope. Main inputs to the model include flow regime, starting elevation, discharge, loss coefficient, cross-section geometry and reach length. Model can handle bridges, weir flow, ice covered streams and split flow options. The computational procedure is based on solution of the one-dimensional energy equation using the standard step method. This is a shareware program available without any technical support. One main limitation with HEC-2 is that it can only handle steady flow problems. HEC-2 can not communicate with GIS hence providing topographic information for very large watersheds like Red River basin is cumbersome. MIKE 11 has a GIS interface and can handle unsteady flows. Cost of MIKE 11 is high but it comes with very good technical support. Considering the size of Red River floodplains and characteristics like flat slopes and complex
topography due to the presence of infrastructure, MIKE 11 is the probably the best available one-dimensional modeling tool.

However, there are certain aspects of modeling that can not be resolved using a 1-D model e.g. determining the flow path of flood wave, velocity and flow in floodplains perpendicular to the flow in the main river, and determination of flooded areas based on topography. In these particular situations a fully two dimensional model is required. For Red River basin areas around floodway inlet, Ste. Agath town and Grand Point are locations where use of 2-D model will produce better description of flow field.
3. CASE STUDY PRESENTATION

The proposed 2-D hydrodynamic modeling approach is applied to a part of the Red River basin in Manitoba. The main characteristics of the study area are discussed in this section.

3.1 Description of Study Area

The Red River originates in the North-Central United States in Minnesota and flows north. It forms the boundary between North Dakota and Minnesota and enters Canada at Emerson, Manitoba. It continues northward to Lake Winnipeg. From origin to its outlet in Lake Winnipeg, the river is 350 miles long. The Red River basin covers 45,000 square miles (exclusive of Assiniboine River and its tributary, the Souris) of which nearly 40,000 square miles are in US. The remaining 5,000 square miles are in Canada. In the city of Winnipeg, the Red River is joined by its major tributary, the Assiniboine River from the west (IJC, 1997).

The Red River basin has a sub-humid to humid continental climate with moderately warm summers, cold winters, and rapid changes in daily weather patterns. On average the Red River basin mean monthly temperature range from –15 to +20 degree Celsius. The flow records show that some 80% peak flows at Redwood Bridge in Winnipeg come from the main stem of the Red River. Further, a very large portion of these peak flows, some 80 percent or more, originates in the United States.
The basin is remarkably flat. The drainage area of Red River has two basic types of topography. The central portion of the area, extending east and west of the river is a broad, flat plain with very gentle slopes. As a result, once the river leaves its banks very extensive areas are subject to flooding. Surrounding the plain is a rougher and higher upland region. Because of the gentle slopes the Red River and the lower end of its tributaries have never developed sufficient velocity to cut channels adequate to carry the higher flows. Between Emerson and Winnipeg the slope is especially flat, averaging only about one-quarter of a foot per mile.

The soil covering the Red River plains consists of a highly plastic clay which is able to hold large quantities of water and possesses high swelling and shrinking characteristics with change in moisture content. These qualities make it a very poor foundation material and make the riverbanks in many areas unstable and subject to slides.

The St. Agathe town was completely flooded during the 1997 flood event. Focus area for the study is from south of the Winnipeg floodway to the town of St. Agathe. This is a very flat area where flow is predominantly beyond the x-section of the river once the flood arrives. There are pockets formed by highways 330, 305, 75 and CN railway line as shown in Figure 4. A 2-D hydrodynamic modeling approach is used in this study to accurately capture the flow in this flat and complex topography.
Fig. 4 Schematic diagram of study area
3.2 Data Requirements

The data required for 2-D hydrodynamic modeling can be divided into two categories i.e. hydrological data and topographic data. Details on data type, resolution and its processing are given in this section.

3.2.1 Hydrologic Data

The discharge and water level data used in this study was collected from:

1. Manitoba Department of Natural Resources, Water Resources Branch.
2. Water Survey of Canada (WSC)

The gauging stations used in the study are listed in Table 1.

<table>
<thead>
<tr>
<th>Location</th>
<th>Station No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red River near Ste. Agathe</td>
<td>05OC012</td>
</tr>
<tr>
<td>Red River Floodway near St. Norbert</td>
<td>05OC017</td>
</tr>
<tr>
<td>Red River near St. Norbert</td>
<td>05OC008</td>
</tr>
</tbody>
</table>

Hourly discharges at Red River near Ste. Agathe and hourly water levels below floodway were used as upstream and down stream boundaries respectively. Hourly wind data recorded at the Winnipeg airport was also used.
3.2.2 Topographic Data

Two different topographic data sets were used for this study. Initially, topographic data for the floodplain covering area from inlet of Winnipeg floodway to Morris was obtained from Land Information Division, Manitoba Department of Natural Resources. This data was based on surveys carried out in early fifties at a scale of 1:60,000 with five feet contour interval. To improve the resolution, preliminary DEM produced from this data was interpolated to get a 500 m grid. Cross-section data collected and its sources are shown in Table 2.

Table 2: Data sources for cross section data

<table>
<thead>
<tr>
<th>River</th>
<th>Reach</th>
<th>Data Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red</td>
<td>Floodway Inlet to Ste. Agathe</td>
<td>1979 Manitoba Department of Natural Resources</td>
</tr>
<tr>
<td>Red</td>
<td>Ste. Agathe to Morris</td>
<td>1951 Red River Hydrographic Survey</td>
</tr>
</tbody>
</table>

The second set of topographic data, with significantly better resolution, became available in September 1999, through IJC (Laser Images Plus, 1999). This data set was in the form of ArcInfo GRID files and was derived from LIDAR airborne survey. Data set consists of a 5m by 5m grid with elevation value available at the center of each grid. The area covered by this data was 688 km², from south of the Winnipeg floodway to Ste. Agathe. This area is covered by 43 sheets where each sheet covers an area of 16 square kilometers (4 km X 4 km). The projection used for this data set is UTM NAD83 Zone 14 (North) and Vertical Datum
used is CGVD1928. The digital elevation model produced from LIDAR data set is shown in Figure 5.

![Fig. 5 DEM of study area created from LIDAR data](image)

**3.3 Data Issues With Respect to Red River Basin**

The data issues related to two-dimensional hydrodynamic modeling in the Red River basin can be discussed under four categories.

**Topographic Data:** The use of high quality and fine resolution LIDAR data has demonstrated that results of two-dimensional hydrodynamic model are very much influenced
by resolution of topographic data. However, high quality data is only available for a portion of the Red River basin covering 688 km² from south of Winnipeg floodway to Ste. Agathe town. Moreover, the communication between MIKE 21 and GIS is not efficient, every change in topography requires reprocessing of data.

**X-section data:** LIDAR data capturing technique can not collect topographic information for areas under water, so cross-section information for main Red River is missing from latest topographic data set. Cross-section data from old surveys has been merged with LIDAR data. Results show that old cross-section data is good enough to describe the topography of the river.

**Boundary Conditions:** Currently available discharge and water level data is of good quality and is sufficient to set up the boundary conditions of the model. However, a gauging station at floodway inlet for direct measurement of velocity and flow is recommended (also recommended by Klohn-Crippen, 1999)

**Calibration Data:** The data required for calibration includes Manning’s roughness coefficient for river and floodplains, eddy’s viscosity and wind friction coefficient. Calibration is carried out by trial and error method until a reasonable match between observed and modeled water levels, discharges and velocities is achieved. Sufficient data is available on the main Red River for calibration purposes however, this is not the case in floodplains. During flood events there should be measurement of overland flow, water levels and velocities at key locations in the floodplain such as along Z-dike, Avonlea corner, near Emerson, Morris and Ste. Agathe. This will be a valuable information for calibrating the model in floodplain areas.
4. MODELING APPROACH

The schematic diagram of modeling approach is shown in Figure 6. The topographic data is processed through ArcInfo and ArcView. This processed topographic data is provided as an input to hydrodynamic model MIKE 21. Hydraulic data i.e. boundary conditions and calibration parameters are directly provided to MIKE 21. After model is calibrated, different flooding scenarios are explored. The results of MIKE 21 can either be directly viewed or can be displayed over topographic data through GIS.

![Fig. 6 Schematic diagram of modeling approach](image)

This report only discusses calibration of model and simulation of 1997 flood event. Future work will include the modifications in floodway operation, dike modifications and simulation of flood events of 1979, 1826 and parameter generated floods (Warkentin, 1999).
4.1 Model Application and Discussion on Results

Initially the topography for the model was defined using a course topographic information available at that time with 500 m grid size and 1 m contour interval. Morris and Winnipeg floodway were used as upstream and downstream boundary points respectively. Daily discharge and water level data was used.

Normally, model requires calibration using trial and error procedure by changing bed resistance. In order to provide for better comparison with MIKE 11 it was decided to use the Manning’s roughness coefficient (n value) reported by Klohn-Crippen (1999). They have derived, through calibration of 1-D model, the roughness coefficient value 0.067 for the floodplains.

Drying and flooding banks are of paramount importance in the floodplains as well as along the main river channels. The modeling tool used in this study incorporates drying and flooding banks in a robust manner, without excessive smoothing of bathymetry. Whenever water level at a particular grid falls below user defined value, this grid is taken out of the calculation of flooded area. That particular cell is added into the flooded area calculation when water level in that cell reaches a certain threshold value.

Model simulations were made for 1997 flood event. Model results are shown in Figures A-1 to A-13 in Appendix A. These figures show the movement of flood wave at selected time steps. Some interesting patterns of flood wave movement can be observed in Figure A-10 and A-11. Both figures show discharge vectors, with Figure A-10 showing
discharge vectors drawn over bathymetry while Figure A-11 showing discharge vectors only. It can be observed that the discharge vectors in circles are representing lateral flow between main stem of river and floodplains. Flow from floodplains that is joining back the flow in the main Red River can also be noticed in these figures. The impact of Z-dike on movement and direction of flood wave is also evident in Figure A-10 and A-11 where flow is changing direction after hitting the dike. Similar trends can be noticed in velocity vectors shown in Figure A-13. A close look at Figure A-12, showing discharge vectors drawn over depth of water, reveals that discharge activity is concentrated in deep pockets of water. It can also be noticed in Figure A-13 that water is approaching the Ste. Agathe town from the west.

Two important conclusions were drawn from these initial model results, based on coarse topographic data. First, model is capable of providing a fairly reasonable estimate of the paths followed by flood wave while moving from Morris to floodway. The second important conclusion was that the available resolution of topographic data is not sufficient enough to obtain meaningful results on water levels at different locations in the model domain.

In September 1999 topographic data of very good resolution became available through International Joint Commission (Lasermap Images Plus, 1999). This data was collected using LIDAR technology. However, data was only available for the area from the Winnipeg floodway to Ste. Agathe town. Based on new topographic data a revised model was developed with upstream and downstream model boundaries at Ste. Agathe and north of Winnipeg floodway respectively. Boundary conditions for the model were specified as variable flux at upstream boundary and variable water levels at downstream boundary. Hourly data for 1997 flood event was used at boundaries. Wind effects were also considered while simulating the
flood wave. For this purpose data on wind speed and direction recorded at the Winnipeg airport was provided to the model. Wind profile used in this study is shown in Figure A-14. Operation of floodway was incorporated in the model as sink and inflow series at floodway was supplied to the model as input. Grid size of 25 m with 15 cm vertical accuracy was used to describe the topography of the floodplain. One tile of the LIDAR data covering the inlet of Winnipeg floodway is shown in Figure A-15. As LIDAR data does not include the cross-section information on main branch of the Red River this information was taken from old data sets. A digital elevation model (DEM) was generated from this topographic data set by processing the data in ArcInfo and finally a 25 m grid was produced by merging the cells. This grid was converted to an ASCII format (x,y,z coordinates), using a scripts written in AML (ArcView Macro Language). This conversion is required to import data in MIKE 21 model. DEM after importing into MIKE 21 is shown in Figure A-16. The model output includes water surface elevation and velocity and flux densities in x- and y-directions. Output results are computed at each grid point for each time step. Discharge vectors computed by model are shown over the topography in Figure A-17. Same discharge vectors without topography are shown in Figure A-18. In this figure it can be observed how Z-dike is influencing the pattern of flow in the area along the dike. This model was used for all subsequent analyses.

To satisfy the Courant number requirement, the time step corresponding to spatial discretization of 25 m is 30 seconds. There are 1.07 million solution points in the model domain. This imposes enormous burden on computational resources. Model takes about three hours to simulate 24 hours of flooding on a 450 megahertz computer running under Windows
NT. Thus it takes about 60 hours to simulate 20 days of flooding from April 20 to May 10 1997.

Observed and model simulated water levels at Ste. Agathe and inlet of Winnipeg floodway are shown in Figure A-19 and A-20 respectively. A comparison of results obtained through this study using MIKE 21 and results reported by Klohn-Crippen (1999) using MIKE 11 is shown in Table 3.

Table 3. Comparison of Recorded and Modeled Peak Water Levels (ft) for 1997

<table>
<thead>
<tr>
<th>Location</th>
<th>Recorded Peak (ft)</th>
<th>Modeled Peak MIKE 11 (3)*</th>
<th>Modeled Peak MIKE 21 (4)</th>
<th>Difference 3-2 (ft) (5)*</th>
<th>Difference 4-2 (ft) (6)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Main River</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ste. Agathe</td>
<td>776.5</td>
<td>776.2</td>
<td>776.3</td>
<td>-0.3</td>
<td>-0.2</td>
</tr>
<tr>
<td>Ste. Adolphe</td>
<td>772.5</td>
<td>772.5</td>
<td>771.6</td>
<td>0</td>
<td>-0.9</td>
</tr>
<tr>
<td>Floodway Inlet</td>
<td>771.5</td>
<td>771.5</td>
<td>770.4</td>
<td>0</td>
<td>-1.1</td>
</tr>
<tr>
<td><strong>Floodplain</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Near Ste. Agathe</td>
<td>779.6</td>
<td>779.6</td>
<td>779.3</td>
<td>0</td>
<td>-0.3</td>
</tr>
</tbody>
</table>

* MIKE 11 results are taken from Klohn-Crippen (1999) with factoring of inflows.

The error in peak water levels generated through MIKE 21 ranges between 0.2 to 1.1 ft. Similarly MIKE 21 has simulated a peak flow of 129,200 cfs at the floodway inlet compared to 138,000 cfs observed. The difference in recorded and observed peak is 8,800 cfs.
i.e. under prediction of 6.4 % of the recorded flow. Observed and simulated velocities in the floodplain are shown in Table 4. The maximum error in velocities is 0.15 ft/s. This can be noted that the velocities are under-predicted.

Table 4. Comparison of Recorded and Modeled Velocities (ft/s) for 1997 using MIKE 21

<table>
<thead>
<tr>
<th>Location</th>
<th>Date and Time</th>
<th>Recorded Velocity</th>
<th>Modeled Velocities</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Over Flow Over PTH # 210 East of St. Adolphe</td>
<td>May 9, 1997/1200</td>
<td>0.42</td>
<td>0.36</td>
<td>-0.06</td>
</tr>
<tr>
<td>Ste. Adolphe Coulee at St. Adolphe</td>
<td>Apr. 27, 1997/1200</td>
<td>1.19</td>
<td>1.12</td>
<td>-0.07</td>
</tr>
<tr>
<td>Red River Overflow West of St. Adolphe</td>
<td>May 9, 1997/1500</td>
<td>1.51</td>
<td>1.38</td>
<td>-0.13</td>
</tr>
<tr>
<td>Flow Over P.R. 205 Near St. Adolphe</td>
<td>May 9, 1997/1200</td>
<td>1.58</td>
<td>1.43</td>
<td>-0.15</td>
</tr>
<tr>
<td>Flow West of CPR Tracks</td>
<td>May 9, 1997/1315</td>
<td>1.11</td>
<td>0.98</td>
<td>-0.13</td>
</tr>
<tr>
<td>East Side of St. Adolphe Ring Dike</td>
<td>May 9, 1997/1400</td>
<td>0.73</td>
<td>0.62</td>
<td>-0.11</td>
</tr>
<tr>
<td>Flow Through St. Adolphe Coulee Bridge</td>
<td>May 9, 1997/1330</td>
<td>1.01</td>
<td>0.96</td>
<td>-0.05</td>
</tr>
</tbody>
</table>

As model is consistently under predicting the discharges and water levels this may be attributed to un-gauged inflow from tributaries joining the Red River between Ste. Agathe and Winnipeg floodway inlet. In order to account for un-gauged inflow to the Red River and model its effects at floodway inlet, MIKE 11 study (Klohn-Crippen, 1999) has factored up the inflow north of Morris. However, no explanation has been given in the report on first selecting the inflow factors and then changing the selected inflow factor values twice (Klohn-Crippen, 1999, page 9, 16 and 17). Similar approach by factoring up the inflows with a certain percentage can improve the accuracy of MIKE 21 model simulated water levels.
4.2 Comparison of Models Based on Use in Red River Basin

<table>
<thead>
<tr>
<th>1-D Model (MIKE 11)</th>
<th>2-D Model (MIKE 21)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model can perform well with limited topographic information. The type and resolution</td>
<td>Detailed topographic description of river and floodplain is required e.g. information</td>
</tr>
<tr>
<td>of topographic data currently available for Red River basin is good enough to setup a</td>
<td>on surface elevation at each grid point is must. Currently this type of data is not</td>
</tr>
<tr>
<td>1-D model.</td>
<td>available for entire Red river basin.</td>
</tr>
<tr>
<td>Lesser topographic data requirement to define model along with continuity and</td>
<td>Due to detailed description of topography and fully two-dimensional equations of</td>
</tr>
<tr>
<td>momentum equations written in one dimension only makes the task of setup and running</td>
<td>continuity and momentum, 2-D models require significantly more time to setup and run.</td>
</tr>
<tr>
<td>the model efficient.</td>
<td></td>
</tr>
<tr>
<td>Output of model is water levels and discharges in one dimension i.e. along the</td>
<td>Output of model is water levels, discharges and velocities in two dimensions i.e.</td>
</tr>
<tr>
<td>direction of flow in Red River. There is significant flow in the Red River</td>
<td>along and perpendicular to the flow in Red River. Description of flow field in 2-D</td>
</tr>
<tr>
<td>floodplains during large flood events like 1997 that can not be described accurately</td>
<td>provides accurate representation of flood wave propagation and better prediction of</td>
</tr>
<tr>
<td>(possible by using quasi-2-D approach). Moreover 1-D model is not capable of</td>
<td>the effects of river training, scouring and sediment transport process. Model can</td>
</tr>
<tr>
<td>providing information such as the velocity with which flow is striking the dike.</td>
<td>assess the impacts of proposed changes, such as dredging or addition or removal of</td>
</tr>
<tr>
<td></td>
<td>flood control structures.</td>
</tr>
<tr>
<td>Operation of gated structure can be captured in the model using either a rating</td>
<td>Model can not explicitly handle operation of gated structures however using sink</td>
</tr>
<tr>
<td>curve or specifying discharges as a function of time. However, operating rules</td>
<td>function along with specification of discharge as a function of time can serve the</td>
</tr>
<tr>
<td>can not be incorporated in the model. This poses a challenge in case of Red River</td>
<td>purpose. This is a limitation while applying model to the Red River basin as floodway</td>
</tr>
<tr>
<td>basin where operation of Winnipeg floodway is based on water levels at James Avenue.</td>
<td>plays an important role in modeling floods in the basin. Model can not handle</td>
</tr>
<tr>
<td></td>
<td>operating rules for gated structures.</td>
</tr>
<tr>
<td><strong>1-D Model (MIKE 11)</strong></td>
<td><strong>2-D Model (MIKE 21)</strong></td>
</tr>
<tr>
<td>-------------------------</td>
<td>-------------------------</td>
</tr>
<tr>
<td>Water level and discharge information is only available at points where cross-sections are defined. This is a limitation since distance between cross sections varies between 1 to 5 km at different locations in Red River basin.</td>
<td>Water levels, discharges and velocities can be obtained at each grid point in the model domain i.e. practically every where since grid size used in the model is 25 m. (only for a part of Red River basin i.e. south of Winnipeg floodway to Ste. Agathe town where LIDAR data is available)</td>
</tr>
<tr>
<td>1-D models do not perform well in areas where lateral flow in and out of floodplains plays an important role in flood wave propagation. However, with quasi-2-D modeling approach lateral flows can be modeled with reasonable accuracy. Quasi 2-D modeling approach requires predetermined pathways, to be defined by the modeler, for flood wave propagation. That is a limitation especially in cases where main interest is in finding the exact path of flood wave and where multiple paths are possible (case of very flat area like Red River basin).</td>
<td>Contrary to quasi-2-D approach in full two-dimensional modeling the path followed by flood wave is computed from the topographic information. This approach provides correct description of the flood wave propagation over floodplains. For Red River basin 2-D models can describe flow more accurately around dikes or in the vicinity of floodway inlet.</td>
</tr>
<tr>
<td>Model is not capable of dealing with flooding and drying. Which means that user has to identify the areas before simulation that would be allowed to be flooded or areas that would remain dry. This poses a limit whenever a flood scenario larger than calibration flood is simulated (e.g. 1826), as it is not known which area would be flooded.</td>
<td>Model can handle flooding and drying. The areas where water depth reaches zero are considered dry and are automatically taken out of calculation. These areas are brought back into the calculation when water reaches in those areas again.</td>
</tr>
<tr>
<td>Capacity to consider lateral flows using quasi 2-D modeling approach, ability to perform well with limited topographic data and time efficient solution algorithm makes 1-D modeling approach a favorable choice for real-time operational models.</td>
<td>In 2-D modeling equations of continuity and momentum are written in two dimensions and results are calculated at each grid point in the solution domain. Thus only a fine spatial resolution (dx) can be used that makes computing slow and requires a lot of computer memory.</td>
</tr>
</tbody>
</table>
5. CONCLUSIONS

A two-dimensional hydrodynamic model was developed for the Red River covering area from south of Winnipeg floodway to Ste. Agathe. The flood event of 1997 was simulated using the model. The difference between observed and modeled peak water levels at different locations is in the range of 0.2 to 1.1 ft. The difference between the peak observed and simulated discharges at the floodway inlet is 8,800 cfs i.e. 6.4 % of recorded flow. This can be attributed to additional flow joining the Red River between Ste. Agathe and the Winnipeg floodway inlet.

Despite tremendous developments in computer software the interoperability of different software components is still a major area of concern. There is no direct communication between GIS and hydrodynamic model MIKE 21. All topographic data processed in GIS needs to be converted to ASCII format prior to importing it to MIKE 21. With every change in topography e.g. size, height or location of dike the whole process has to be repeated.

LIDAR airborne survey was used to collect the topographic information in the study area. Lidar technology is not capable of penetrating the water surface so this method does not provide any information on river cross-sections. So cross-section data is adopted from a different source that requires datum correction and geo-referencing. This merging of data sets introduces some error in high resolution LIDAR data as cross-section data is from old surveys carried out in 1979 and 1950’s.
One important conclusion after experimenting with different resolution of topographic data is that model is only as good as the data is. The large area to be modeled in Red River Basin pose a challenge for the application of the two-dimensional model.

Some important points derived from the comparison of 1-D and 2-D hydrodynamic modeling approaches are:

(a) Mike 21 can not explicitly model the operation of floodway. There is an indirect way to incorporate floodway operation i.e. by using sink function in the model and providing observed series of inflows at floodway as input to sink. However, there is a limitation with this approach since the sink function only affects the continuity equation. The momentum term is unaffected in the Saint-Venant equation. Thus, backwater effects due to operation of floodway can not be captured accurately.

(b) Two-dimensional models, compared with 1-D models, require a significant amount of additional data (especially topographic data) and time to set up and run. Any change in topography like addition of dike or road will require a change in topographic data and incorporating such changes, in general, is more time consuming compared to 1-D modeling. However, two-dimensional models provide a description of flow path and velocities. Lack of this feature makes quasi 2-D modeling inferior to fully 2-D modeling.

(c) Due to detailed description of topography and additional terms in mass and momentum equations the 2-D models require more time and computational resources to
simulate same hydrological event. MIKE 11, with a time step of 5 minutes and spatial resolution of 500 m, takes 15 minutes to simulate the 1997 flood event (Klohn-Crippen, 1999). The time required for the same event by MIKE 21, with time step of 30 seconds and spatial resolution of 25 m, is around 48 hours. However with rapid advancement in computing power this will not be an issue in the near future.

(d) Model setup is complete and now it can be used for analysis of different scenarios by changing height and location of dikes. Impacts of newly constructed ring dike around Ste. Agathe town can also be studies using the existing model setup.

(e) Considering the current cost of acquiring good resolution topographic data and computing resources involved in setting-up and running the model, it is not economical to setup the 2-D model for whole Red River basin. Moreover, two-dimensional hydrodynamic modeling for the whole Red River basin will not provide better results compared to 1-D model with currently available topographic data.

(f) Therefore, it is recommended to model the whole Red River basin using a one-dimensional or quasi two-dimensional model. Fully two-dimensional model can be used in areas of special interest like the floodway inlet, around Ste. Agathe town and Grand Point area. Another advantage of using this combined modeling approach is that the boundary conditions required for 2-D model can be obtained from results of 1-D model.
6. REFERENCES


APPENDIX – A

Figures
Fig. A-1  Discharge vectors drawn over bathymetry, Time step 1

Fig. A-2  Discharge vectors drawn over bathymetry, Time step 2
Fig. A-3 Discharge vectors drawn over bathymetry, Time step 3

Fig. A-4 Discharge vectors drawn over bathymetry, Time step 4
Fig. A-5 Discharge vectors drawn over bathymetry, Time step 5

Fig. A-6 Discharge vectors drawn over bathymetry, Time step 6
Fig. A-7 Discharge vectors drawn over bathymetry, Time step 7

Fig. A-8 Discharge vectors drawn over bathymetry, Time step 8
Fig. A-9 Discharge vectors drawn over bathymetry, Time step 9

Fig. A-10 Discharge vectors drawn over bathymetry, Time step 10
Fig. A-11 Discharge vectors, Time step 10

Fig. A-12 Discharge vectors drawn over water depth, Time step 10
Table showing water levels west of Ste. Agathe simulated by MIKE 21

<table>
<thead>
<tr>
<th>Date</th>
<th>Water Level (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 27, 1997</td>
<td>774.8</td>
</tr>
<tr>
<td>April 28, 1997</td>
<td>775.1</td>
</tr>
<tr>
<td>April 29, 1997</td>
<td>776.2</td>
</tr>
<tr>
<td>April 30, 1997</td>
<td>777.8</td>
</tr>
<tr>
<td>May 1, 1997</td>
<td>779.6</td>
</tr>
<tr>
<td>May 2, 1997</td>
<td>778.5</td>
</tr>
<tr>
<td>May 3, 1997</td>
<td>776.7</td>
</tr>
</tbody>
</table>

This may be noticed in Figure A-13 (marked by a circle) that velocity vectors are moving towards Ste. Agathe area from the west. Analyzing the flow direction at different time steps reveals that water starts moving towards west when water level exceeds 776.0 ft on April 29, 1997 around 2:00 am. This is the strength of 2-dimensional model that path of flood wave can be traced accurately.
Fig. A-14 Wind Profile
Fig. A-15. One tile of LIDAR data showing the Winnipeg floodway on Red River

Fig. A-16. Topographic data after importing into MIKE 21
Fig. A-17 Discharge vectors drawn over bathymetry

Fig. A-18 Discharge vectors
Fig. A-19 Comparison of Observed and Simulated Water Levels at Ste. Agathe

Fig. A-20 Comparison of Observed and Simulated Water Levels at Floodway Inlet