3 Methods of Backwater Calibration

As outlined in Section 1, the objective of this study was to re-compute the "natural" water level relationship at the Floodway Inlet over a wide range of Red River flows and variable backwater affects from the Assiniboine River. The methodology used to calculate "natural" water levels at the inlet is also outlined in Section 1. "Natural" or "pre flood protection" conditions are defined in Section 1 as the level of development in the City of Winnipeg just prior to the 1950 flood and does not include: temporary dykes constructed to protect portions of the city (Figure 2-1); or the primary dyke system constructed in the fall of 1950 following that year's disastrous flood (Figure 2-12) to provide interim flood protection before more extensive flood proofing measures (i.e., Red River Floodway, Shellmouth Reservoir, Portage Diversion) could be built later in the 1960's.

Defining the Floodway's natural water level relationship or rating curve involves re-computing the backwater analysis that was initially done by the Province in the early 1960's (Section 1). For this re-computation analysis, the US Army Corps of Engineers River Analysis System, HEC-RAS model (Version 3.1.1) was used. The HEC-RAS model is a one-dimensional backwater flow model and is considered to be the universal standard for computing steady state (i.e., constant flow) water surface profiles. It was judged that one dimensional flow conditions predominate over this reach of the Red River. The Acres' HEC-RAS model extends from near Lake Winnipeg to just upstream of the Floodway Inlet. The major differences between the Province's modelling in the early 1960's and the Acres' HEC-RAS model were summarized in Section 1.

3.1 Overview to Model Development and Calibration

The following provides an overview to the model development and calibration. Specific details of these steps are discussed in the following sections with additional details found in the Appendices.

3.1.1 Study Area

Development of the physical cross section base of the model involves defining the river cross section up to bankfull stages as well as defining the overbank cross section area, which are inundated under large floods. Figure 2-1 shows the flooded extent of the 1950 flood (with flow estimated by the Province at

108,500 cfs Section 2.3.3. As noted in Section 2.3.3 this flow estimate is under review in this study. Notwithstanding some uncertainty in flow magnitude the extent of overbank flooding is largest in the upstream section of the Red River and decreases downstream of the confluence with the Assiniboine River indicating increased bank full capacity. The likely reason for this is that the Assiniboine River in its geological past has had an influence in increasing the bankfull capacity of the Red River downstream of the Forks (i.e., confluence of the Assiniboine and Red river). Figure 2-5 shows the estimated extent of even larger floods including the 1826 which is estimated at 225,000 cfs (Clark, 1950). The KGS Group (2001) report has re-examined the RRBI estimates for the 1826 flood and have determined that the 1826 flow could be as low as 180,000 cfs (if City of Winnipeg levels are used) or as high as 300,000 cfs (if non ice affected Selkirk levels are used). For this study floods up to 300,000 cfs are to be modelled, which will involve extending the overbank cross sections even further than shown in Figure 2-5. Discussion of the construction of the physical cross-section base of the model is outlined in Section 3.2 and discussed in more detail in Appendix B. A component of the river cross section portion of the model are the bridges that existed circa 1950 as shown in Figure 2-1 and described in Section 3.2.1 and in more detail in Appendix C.

3.1.2 Model Calibration Process

The model calibration process involves taking the cross-section model that was constructed and calibrating the backwater model to observed water levels for various flows. The key parameter that is adjusted in the calibration phase is the hydraulic roughness or Manning’s “n” of the cross section to enable a match between observed and computed water surface profiles for a given flow event. If the model does not calibrate with a reasonable roughness factor, the model is reviewed to determine whether there are errors in the construct of the model such as the physical cross sections or in the observed data that is being used to calibrate the model. The 1950 and 1966 datasets were critical to model calibration and were reviewed previously to determine whether there were any anomalies with the data. As noted in Section 2.3 there were errors in the 1950 flow estimates and unresolved differences in the water level data recorded at certain locations.

The hydraulic roughness of the cross section is not typically uniform across the cross section. For flows that are contained within the normal summer channel cross section, the channel roughness will be typically defined as one value. In the case of the Floodway channel the n value for bankfull conditions has been determined to be around 0.026 (Red River Floodway Operation Review Committee, 1999), it is expected that the normal river channel for the Red will likely be within this range.

For larger flows that flood the wooded bank areas up to bankfull stage, the channel roughness will increase in this section of the cross section due to the additional hydraulic roughness of trees, bush and debris. As flows increase even further, the water surface will exceed the top of riverbank and will flood the overbank/floodplain area. In the case of Winnipeg, ground cover in the floodplain can vary from developed residential/commercial areas and cultivated land in the outskirts particularly for circa 1950 conditions, each of which will have a different roughness value. A more complete discussion on n values is found in Section 3.3.

As there will be varying hydraulic roughness or “n” values depending on the conveyance area of the cross section, the model was initially calibrated for flows in the 20,000 to 40,000 cfs range. This range defines flows just above normal summer stage (i.e., 7 ft JAD) up to 15 ft JAD. The data for this calibration comes from a 1992 spring water level data collection program tabulated in Appendix D. While modelling up to 40,000 cfs will involve flooding of treed river banks, previous modelling (see Appendix B2.2) has shown that the backwater models are relatively insensitive to the selection of n values for overbank since the majority of flow is contained within the channel cross section.

The next step of model calibration was to increase the flow up to approximately 90,000 cfs to model water surface profiles up to the top of the existing primary dykes. This flow will involve some overbank flooding in the southern portion of Winnipeg (i.e., upstream end) and mostly bankfull stages in the north end of the city which should enable the calibration of “n” values within the overbank zone. The data for this phase of the calibration comes from recorded data from the 1966 flood as discussed in Section 2.4.

Once the model was calibrated to the 1966 data, the model was re-run to determine the effects on water levels of removing the primary and secondary dykes that were constructed after the 1950 flood. Based on the Hurst (1957) record, areas such as Elm Park flooded on May 3, 1950 (23.7 ft JAD) and the Riverview, Point Douglas and Wildwood temporary dykes failed on May 6, 1950 (26.2 ft JAD). Given that the temporary dykes provides some modest flood protection before they failed, it is assumed that these areas would have flooded at lower stages if no temporary dykes had been constructed.

Following model calibration to 1966 flows, flows were then increased up to 1950 levels. As previously discussed in Section 2.1 and illustrated in Figure 2-1 the 1950 flood involved extensive overbank flooding. As discussed in Section 2.3.2 and Section 2.3.3 there may be errors in the water level data collected in 1950 for some of the stations illustrated in Figure 2-1, that the backwater modelling should resolve. And more importantly there are errors in the meterings at Redwood Bridge in 1950 as illustrated in Figure 2-14. Because of the metering errors, sensitivity runs were made to determine a range of potential flows and corresponding n values to match the observed stage. Also as indicated in Section 2.4, to calibrate to observed water levels in the St. Boniface area the HEC-RAS model required incorporating a “wall” for the Lyndale Drive Dyke feature. Once the model was calibrated this “wall” or dyke was removed to duplicate “natural” conditions that would have occurred just prior to the 1950 flood.

One aspect of overbank flooding involves the portion of the cross section where water is actually flowing or being conveyed versus areas where water depths are shallow and floodwaters typically just pond. Modelling of this type of feature was initially discussed by the Province in their Flood Damage Reduction modelling of Winnipeg (Appendix B2.2). A full discussion on conveyance and ineffective flow area is found in Section 3.4. Defining portions of the cross section to be effective versus ineffective was an issue for flows greater than 90,000 to 100,000 cfs where water surface profiles exceed the top of bank elevations.

Once the model was calibrated up to the 1950 flood, it was extrapolated up to a flow of 300,000 cfs, using the n values that had been determined through model calibration.

The following section provides additional details on the model development and calibration phase.

3.2 Development of the Cross-Section Component of the Model

The process of developing the cross section base of the HEC-RAS model is outlined in Figure 3-1 and involves merging cross sections that define the river channels with cross-sections that define the overbank or floodplain portion of the cross section.

As outlined in Appendix B2.2, the channel cross sections were originally surveyed during the Red River Basin Investigation (RRBI) study. These surveys were carried out during the winter of 1950/51. An example of a typical RRBI channel cross section is shown in Figure 3-2. From these surveys, detailed bathymetric maps and cross section maps were developed. As outlined in Appendix B2.2, Manitoba Water Resources Branch (WRB) in the late 1970's took the channel cross sections and coded them into a format required for the HEC-2 model a precursor of the HEC-RAS model. As outlined in Appendix B2.2, WRB resurveyed some of the cross sections near the Floodway Inlet (i.e., in the RM of Ritchot) and noted that there was little change in the channel cross sections. In 2001, the KGS Group (2001) re-surveyed a channel cross-section near historic Lower Fort Garry and found no change in the channel cross-section from that of the RRBI 1951 surveys.

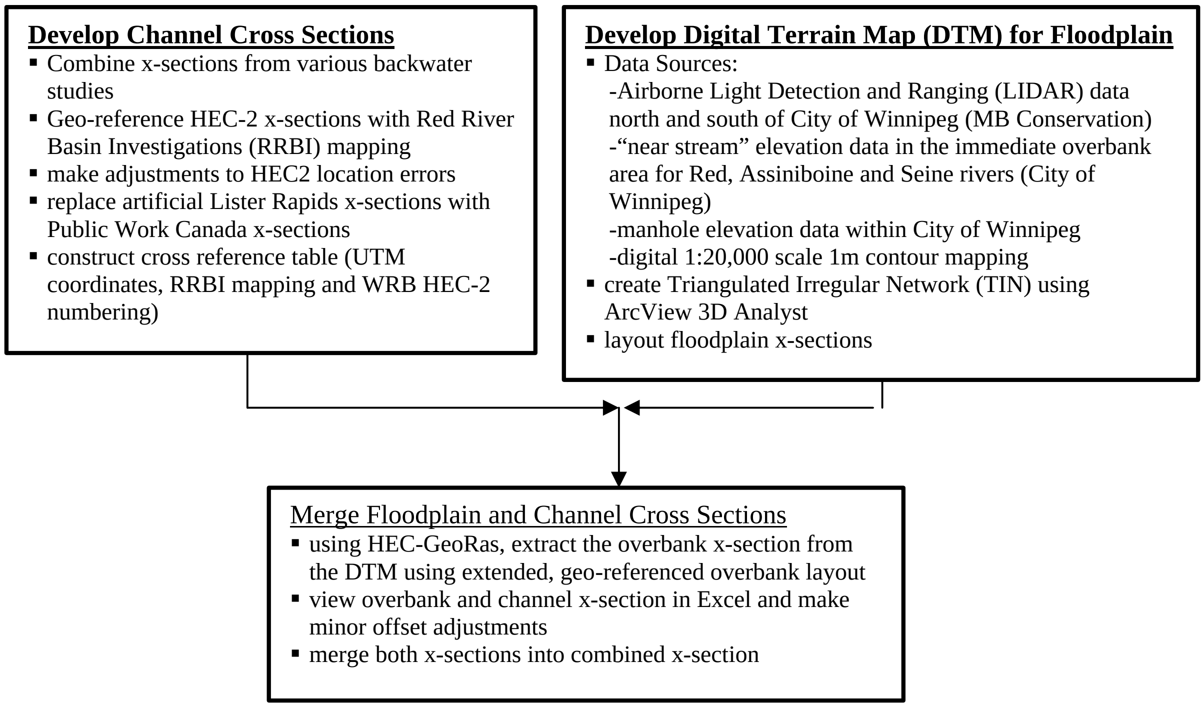
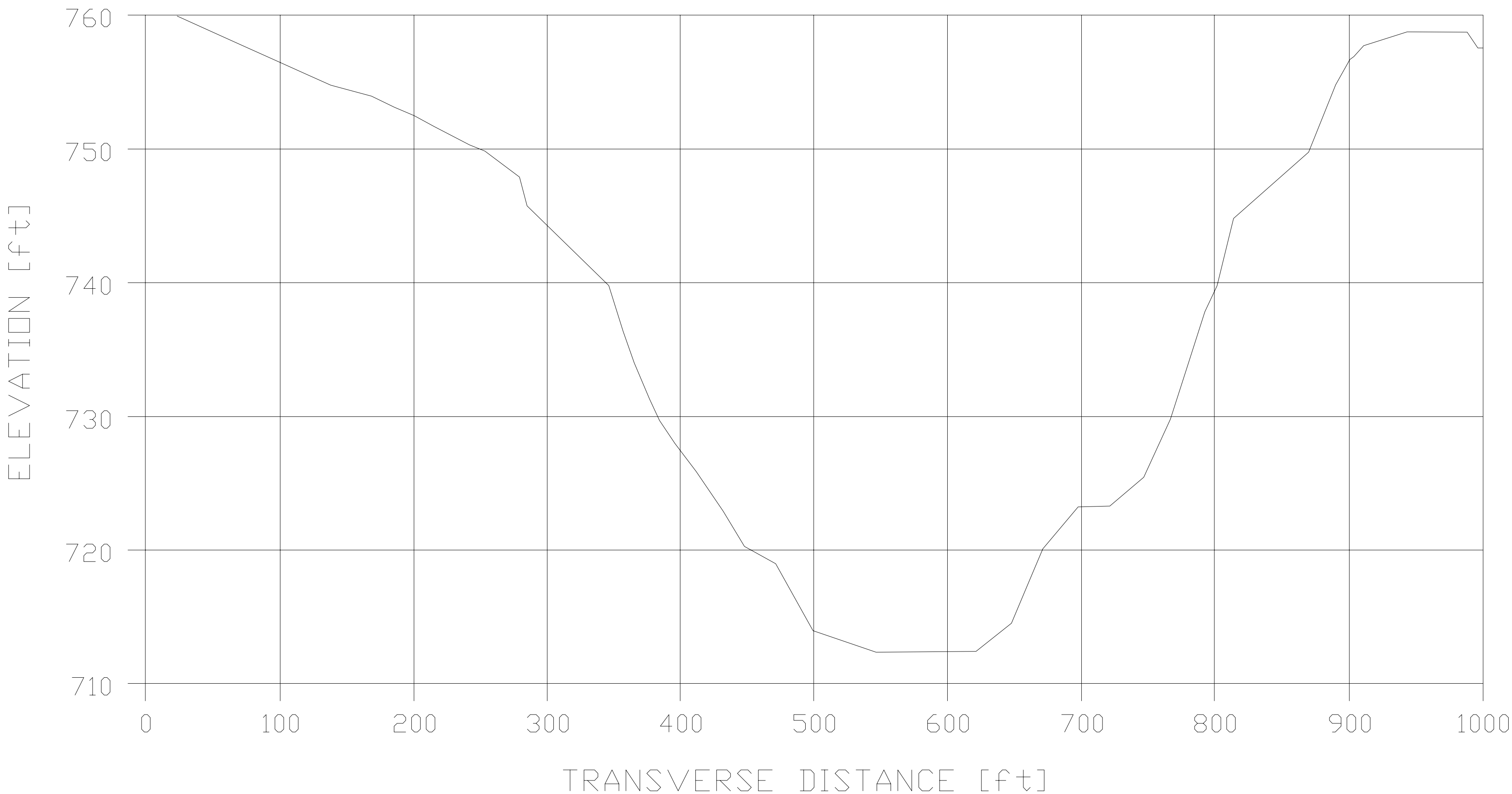


Figure 3-1
Schematic of Development of Acres’ HEC-RAS Model

As illustrated in Figure 3-1, part of the Acres development of the cross section information involved geo-spatially locating the WRB channel cross sections with their physical location shown on the RRBI plan maps. The procedures for doing this, including location adjustments are outlined in Appendix B3.2. As outlined in Appendix B3.2 the “pinning” down procedure corrected the error in chainage distances for the Bergen Cutoff to Redwood Bridge section of the river that was coded wrong in the original WRB deck (Appendix B2.3). Other corrections included replacing the artificial channel cross sections in the Lister Rapids section of the river that were estimated in the original WRB HEC-2 deck with cross sections surveyed by Public Work Canada in 1990 (Appendix B2.3). Appendix B also includes a cross-reference table (Table B1) that relates the Acres geo-spatially located channel cross section with the original RRBI cross section and the cross section numbering system from the original HEC-2 cross sections. Also included in Table B1 are the locations of water level gauge stations along the river.



NOTES: - CROSS SECTION NO. 19 GREATER WINNIPEG SURVEY, FROM THE 1951 RED RIVER BASIN INVESTIGATION, RED RIVER HYDROGRAPHIC SURVEY.
- LOCATED AT STA. 631+71

FIGURE 3-2
Typical 1951
River Cross Section

ACRES

As indicated previously in Section 3.1.1, part of the Acres' model development included the development of overbank/floodplain information to enable the computation of water surface profiles for floods up to the 300,000 cfs (approximately 3 times the size of a 1950 flood). To construct the topography of the floodplain from which cross sections would be later extracted, a three dimensional digital terrain map (DTM) was developed (Figures 3-3). As shown in Figure 3-1, data from a variety of sources was used in constructing the DTM. Descriptions of the various data sources, their relative accuracies and how the data is integrated (or prioritized) is discussed in detail in Appendix B3.1. The layout of the floodplain cross sections are shown in detail in Figure 3-4. The larger study areas are shown in Figure 3-5 for the south and Figure 3-6 for the north section of the study area. A larger scale map of the cross section layout can be found in Appendix F.

The final step in the model development process is usage of the HEC-GeoRas tool that "cuts" the overbank cross section from the DTM. The cross section that is developed by HEC-GeoRas defines the floodplain and overbank area and the riverbank down to normal summer water level, i.e., 6.5 ft JAD or elevation 734 ft ASL. Both overbank cross sections and channel cross sections were then viewed in Excel to make minor offset adjustments and to merge both cross sections to a final combined channel and overbank cross section (see also Appendix B3.5).

3.2.1 Bridges and Structures

Bridges and structures located along the Red River circa 1950 (see Figure 2-1) were also incorporated into the backwater model. The structures may act as constrictions, reducing the hydraulic conveyance of the river and creating a drop in water levels from upstream to downstream or a hydraulic loss. This reduction in conveyance area can be due to pier layout (size, shape or number), low bridge decks or loss of cross section area in the abutment region which will result in a measurable head loss under high flow situations. A review of historical bridge losses from the 1950 and 1966 floods was previously discussed in Section 2.3.1. A brief description of the bridges that were in place during the 1950 and 1966 floods follows. Additional details on the bridges and how they were modelled can be found in Appendix C.

- **Elm Park Bridge** - multi-span open truss type structure with minor approaches and minimal constriction on flow;