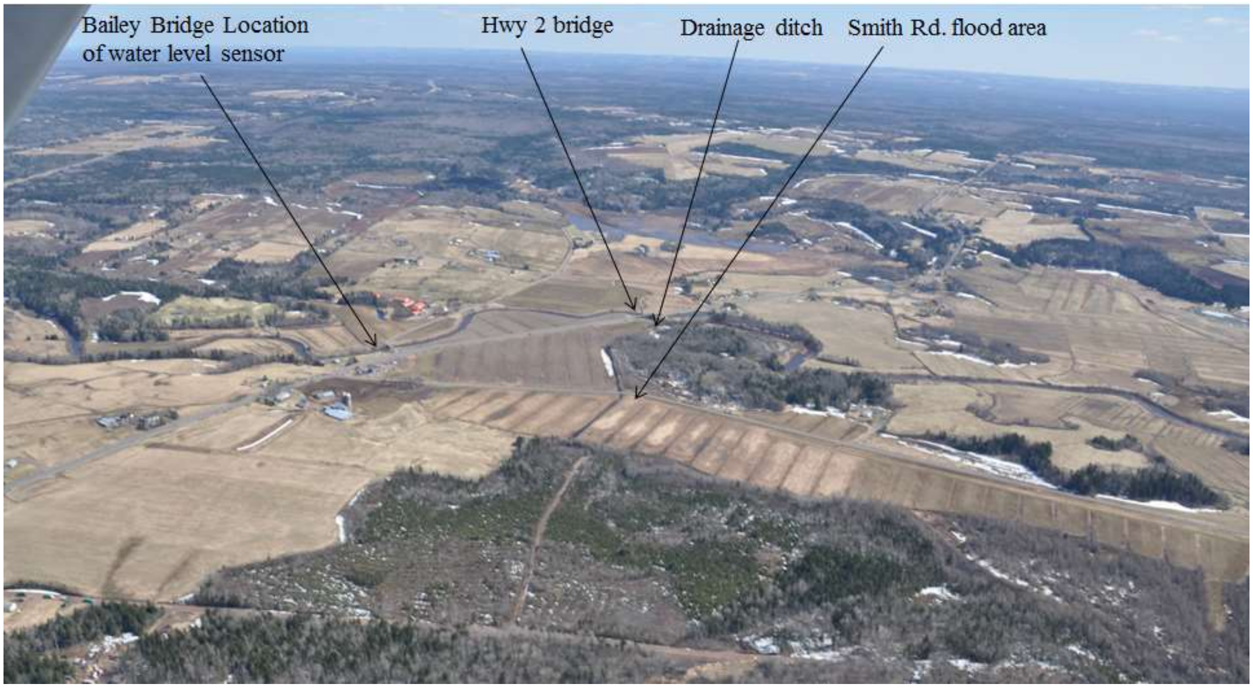
River Flood Risk Study of the Nappan River Incorporating Climate Change



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Executive Summary

This project has demonstrated the complexity of modeling a river whose discharge is controlled by an aboiteau. River flooding has been a problem in Nappan for several years with many people from the area contributing the problem to the presence of an aboiteau downstream. The aboiteau was constructed in the 1960's to control tidal influence upstream and reducing the problem of coastal flooding. The water in the Upper Bay of Fundy has a high degree of suspended sediment as a result of the clay rich soils that drain into the Cumberland Basin. This high level of suspended sediment has contributed to siltation of the river channel directly upstream of the aboiteau. This siltation has resulted in an increase of the elevation of the river bed and a widening of the river channel. Both of these conditions have probably contributed to the problem of flooding upstream. Smith Road is flooded on an annual basis as a result of the river stage increasing and the water traveling back up through a drainage ditch and inundating the road making it impassable. This drainage ditch was originally designed to drain the agriculture field and at one time had an aboiteau to prevent back flow from the river. We installed a pressure sensor to measure river stage from March 2011 to present. We visited the site on several occasions through the field season of 2011, measuring flow and stage, recording water levels with survey grade GPS, and executing a bathymetric survey of the river bed to supplement the laser altimetry (lidar) high-resolution elevation data we have of the floodplain. We have constructed a seamless elevation model (lidar + bathymetry) to facilitate the extraction of river cross-sections used in our 1-D hydraulic model. We have used the stage information along with a local Environment Canada weather station to develop a watershed rain-fall runoff model. In order to control the function of the aboiteau in our moadel, we developed a 2-D hydrodynamic tide model for the Upper Bay. We have simulated past flooding events, such as from July 2010 and increased the rainfall two and three fold to simulate possible increases in precipitation with climate change. We have also run simulations that block the drainage ditch to Smith Road, thus preventing the river from flooding the road. We have also simulated lowering the river bed in the area of silt build up which resulted in lowering the stage of the river and reducing the area flooded.

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1 Introduction

The floodplain of the Nappan River is prone to flooding during heavy rainfall events. This problem is further enhanced by the fact the lower section of the river is a tidal estuary in the Upper Bay of Fundy where the maximum tidal elevation is 14.4 m relative to chart datum. Previous anthropogenic interventions have included the installation of an aboiteau at the river outlet on South Hampton Road and a dam north of Highway #204 above the floodplain. The aboiteau was installed in the 1960s. It has been reported that part of the problem of flooding may be caused by siltation of the river bed upstream from the aboiteau. The watershed draining into the Nappan River extends landward for several kilometres where the land cover changes from agriculture to forest (Figure 1). The degree of clear cutting and regeneration of the forest was qualitatively evaluated by examining historical Landsat satellite images for the watershed (Figure 1). Above the floodplain there is a dam that has a spillway and fish ladder that drains the head pond developed upstream of the dam. The section of the river from the dam to the mouth of the outlet past the aboiteau has been covered with a lidar aerial survey in 2009. These data have been processed and high resolution digital surface and elevation models (DEM) produced. A hybrid 5 m DEM that covers the entire watershed has been constructed by combining the lidar DEM with the provincial DEM derived from the NSTDB 1:10,000 scale mapping.

A pressure sensor was installed by AGRG at the Lower Porter Road Bridge in March 2011 to gauge the river system in order to calibrate and validate a hydrologic model of the watershed. They also installed a barometric pressure sensor at the same location and have since recorded flow measurements of the river at that site with the intention of building a rating curve to relate river stage to flow. The weather information was downloaded from the Weather Underground site (http://www.wunderground.com/) which obtains data from sources such as Environment Canada. The hourly precipitation was used along with the daily temperature minimum and maximum to calculate evapotranspiration, which was used to drive the runoff model.

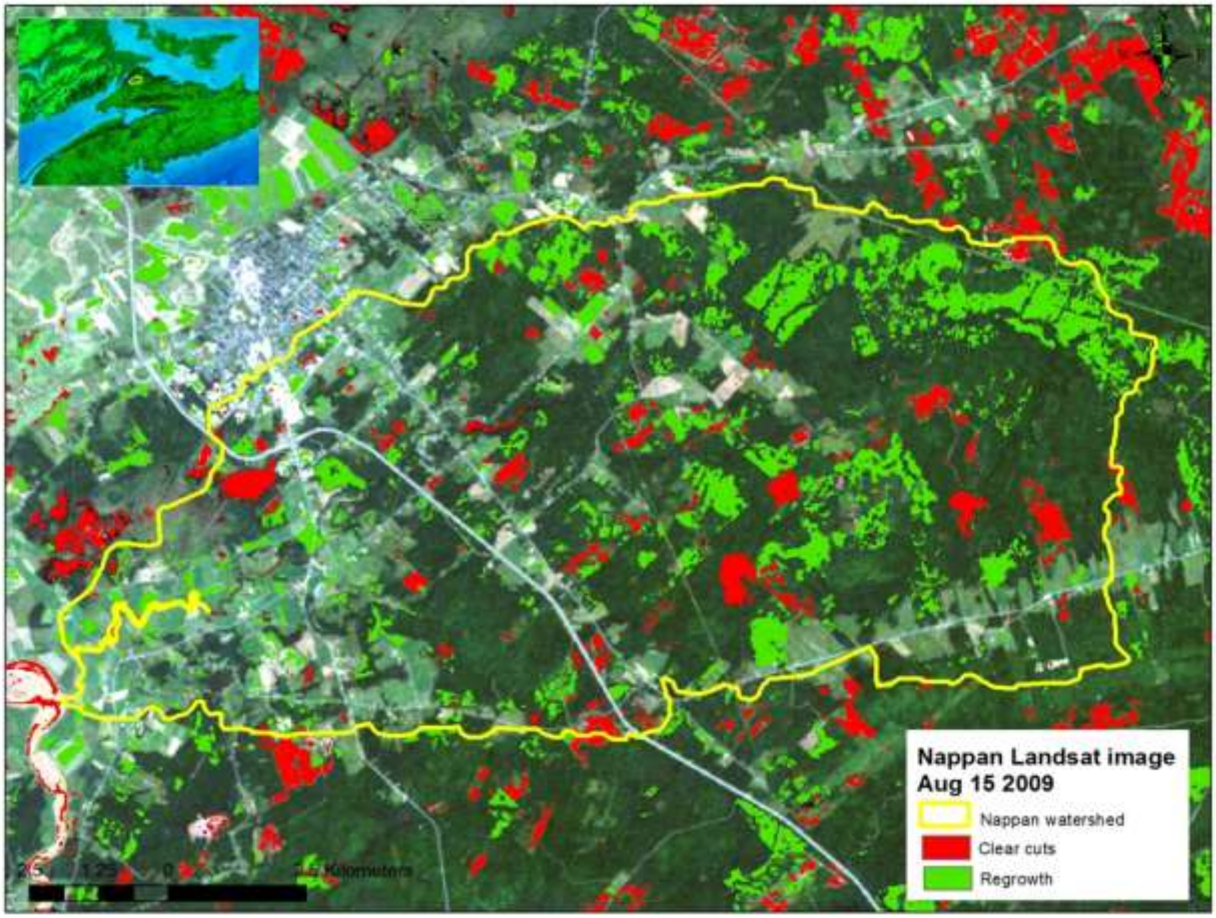

Figure 1 Nappan watershed (yellow outline) with Landsat image backdrop, top left corner show inset location map. The red patches represent clear cuts between 2009 and 1999 and the green patches represent regrowth between 2009 and 1999.

Since the discharge of the Nappan River is controlled by an aboiteau, which in turn is controlled by the state of the tide and thus stage of the water downstream from the aboiteau, we needed to establish an accurate model of the predicted tide so that we could model the action of the aboiteau. Unfortunately the closest station to where tidal prediction are available for Nova Scotia is at Joggins Wharf, which is a large distance away and would not accurately predict the local conditions up the Nappan River channel. To overcome this we developed a 2-D hydrodynamic model, Mike 21 from DHI, for the Upper Bay of Fundy and used a predicted tidal boundary condition from WebTide, a regional hydrodynamic model developed by the Department of Fisheries and Oceans (Dupont et al., 2005). This model was used to develop a time-series that controlled the river flow through the aboiteau. We used the weather and watershed areas with a lumped rainfall-runoff model from DHI, called a NAM model. This model keeps track of antecedent conditions which is very important if you have had a stretch of wet weather and the ground is saturated. Additional rainfall may cause flooding during such conditions. We used a 1-D river hydraulic model in combination with the NAM model to determine the stage and flow of the river channel and floodplain, Mike 11 from DHI. Once the model was established and validated using field measurements, extreme precipitation events were imposed to determine the extent of flooding along the floodplain. Past flooding events were used to qualitatively validate the model, since no accurate records were available of the spatial extent and depth of flooding. We did have photographs and some dates of flooding events, courtesy of Jim Hannon, regional Emergency Management Office coordinator for the area. We supplemented these data with searches of local newspaper for other flooding events (Figure 2).

Figure 2 Historic flooding events for the Nappan Floodplain. Smith Road is an area that is frequently flooded as seen in the lower two photos. Source: Amherst daily News and Jim Hannon, EMO.

Various scenarios were tested to simulate changes in the system. For example, the bathymetry of the river bed channel was altered to simulate siltation to determine the effects on bank overtopping and flooding.

Some of the research questions that were of concern for this river system included:

1. What is the relationship between rainfall and river flooding?
2. How much influence does the tidal stage have on river discharge?
3. What volume of water flows through the system during flood events?
4. What river elevation (stage) occurs at normal and peak flows?
5. What extent of the flood plain is flooded during peak discharge events?
6. How does siltation affect the river discharge and flooding?

In addition to these fundamental questions that are required in order to understand how the system works, including the anthropogenic features including the upstream dam and downstream aboiteau, there are additional questions related to mitigation and adaptation. These questions are also framed in the context of climate change which may affect the area in two significant ways: higher sea-levels in the Bay of Fundy, and more intense rainfall events.

2 Methods

2.1 Topographic Data Collection and Integration

2.1.1 Nappan River Floodplain Lidar Elevation Model (2m)

Airborne lidar (Light Detection and Ranging) information for the Nappan River and surrounding area was collected on October 29th, 2009 using an Optech ALTM (Airborne Laser Terrain Mapper) 3100. The flight included a total of 24 flight lines flown at an altitude of 1600 m AGL (Above Ground Level) with a pulse frequency of 70 kHz and a 20^{o} scan angle at a frequency of 30 Hz. The resultant point cloud data was interpreted into ground and non-ground accordingly and geometrically corrected as outlined in Webster et al (2012). The ground points from the corrected data were then rasterized by means of a linear TIN (triangulated irregular network) type interpolation into a 2 m spatial resolution DEM (Digital Elevation Model). This grid included the entirety of the flood plain area for the Nappan river system.

2.1.2 Nappan Watershed Elevation Model (5m)

Supplementary land elevation information for the Nappan watershed was extracted from the NSTDB (Nova Scotia Topographic Database) 20 m spatially resolved Nova Scotia DEM (Digital Elevation Model). Elevation values in the database have 1 m vertical resolution and derived through stereo pair aerial photography. This data was integrated with the above described lidar derived DEM at a resolution of 5 m. This separate grid was computed in order to produce a comprehensive overland elevation model for the watershed which was as accurate as possible. Unfortunately, the lidar derived DEM alone encompassed only a small portion of the entire Nappan River watershed.

2.1.3 River Bottom Bathymetry Survey Procedure

Elevation data collected by the lidar system does not include areas which were underwater at the time of the lidar flight. Do to the nature of the infrared laser pule emitted from the lidar system, the incident beam is either absorbed in the water column, the outgoing beam is specularly reflected away from the sensor by the water surface, or a reading is returned representing turbid water surface elevation. Thus, to supplement the lidar elevation model within the channel Nappan River , low cost bathymetry elevation data was collected using a simple canoe affixed with a sonar transducer (precise to 10 cm vertically) and a RTK (real time kinematic) survey grade GPS (global positioning system) receiver (accurate to 2.5 cm, three dimensionally). Such river channel depth data, combined with lidar data, is crucial for the accuracy of the resultant hydrodynamic model. This is specifically applicable in terms of generating accurate cross sectional area, a proper estimation for the conveyance of flowing water through the channel and the overall storage capacity of the river. Unfortunately, a comprehensive bathymetric survey of the Nappan River was not possible with this method as the river became too shallow nearer the aboiteau.

The collected bathymetry points were processed using the Leica Geo Office software package to an orthometric elevation. Points with a GPS height standard deviation greater than 10 cm were omitted. A manual digitization of the lidar DEM was performed to remove the erroneous water surface elevation values of the Nappan River. Where bathymetry points existed, they were interpolated along with lidar 'bank' elevation points directly adjacent to digitized water surface using a spline interpolation method in ArcGIS to create a smooth and comprehensive elevation model of the river channel. This raster was then mosaicked into the existing 2 m lidar DEM; where the digitized water surface polygon existed.

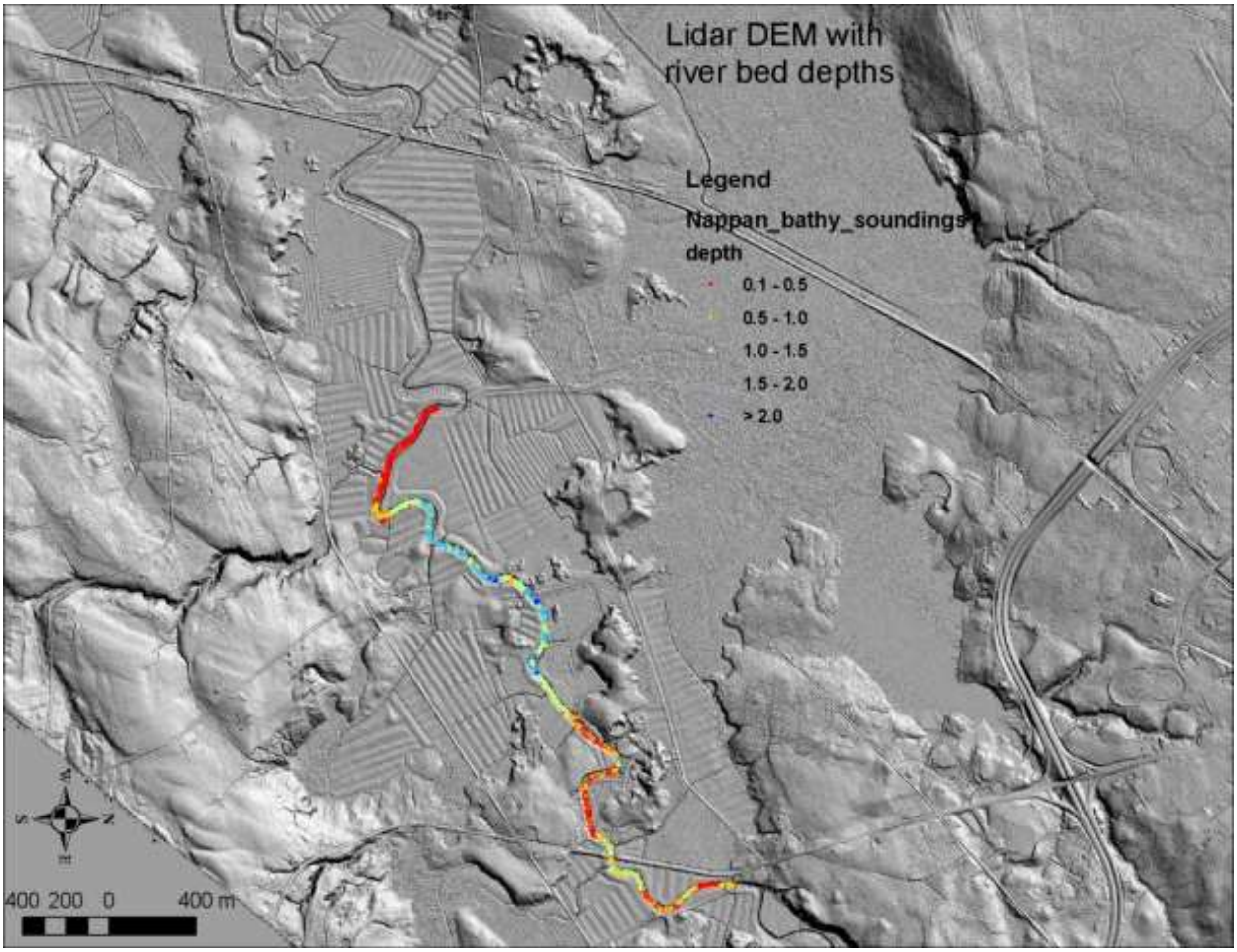

Figure 3 Shown is the collected bathymetry overlain on the lidar DEM hill shade. The decrease of depth, ultimately resulting in a cease to the bathymetry survey, in the downstream portion of the river is indicative of increased sedimentation.

2.1.4 Approximation of Unsurveyed River Bottom Procedure

Where bathymetry was not collected, or was inaccessible, depth information was estimated. This was accomplished by means of a Euclidian distance calculation from the water polygon generated from the lidar grid inward, whereby the middle of the estimated channel was given a high value and values closer to the banks a low value. These values were then factored by an estimation of slope which was experimentally determined to be 10%. Additionally, a grid was generated to represent the trend of bank elevations from the lidar data directly adjacent to the estimated water polygon for which the above mentioned depth estimation calculation grid could be subtracted from. The resultant grid was mosaicked into the lidar DEM at a spatial resolution of 2 m.

The dimensions and characteristics of each structure pertaining to the flow of surface water in the Nappan watershed, and specifically in the direct path of the Nappan River, were taken into account where possible. Culvert locations and orientations for the entirety of the watershed were collected from the provincial roads database, provided by Nova Scotia Topographic Database and other sources. Dimensions for the Southampton Rd aboiteau (NS109-23), located at the mouth of the Nappan River, and were provided in part by the Department of Agriculture. In places where design schematics were not complete for the Nappan aboiteau as supplied, dimensions for the sister aboiteau at Great Village (NS114) were used with the assumption of having a similar depth and width measurements. Additionally, survey grade GPS orthographic elevation information was collected on the top of structure, both on the upstream and downstream portions, with the aim of positioning the design information relative to CGVD28 (the Canadian Geodetic Vertical Datum of 1928). Dimensions for the spillway dam, located upstream of the Nappan River flood plain, were taken from the lidar point cloud and supplemented where possible with survey grade GPS in conjunction with laser positioning with a Leica Total Station survey system.

2.1.5 Mike 21 Grid Preparation

Model bathymetry roughly covered the extent of the Cumberland Basin. Bathymetric data were composed of coarse resolution Canadian Hydrographic Services (CHS) digitized nautical chart soundings (circa 1969) around the basin and deep river systems. The bathymetric data were supplemented with high resolution lidar elevations flown in October, 2009. Standard lidar does not penetrate water, so it was ensured that surveys were conducted at low tide. As a result, several intertidal mud flats and coastal features within the study area were clearly defined within the dataset. CHS soundings were converted from chart datum to CGVD28 in order to establish a common vertical datum between all datasets. An interpolation algorithm was used to fill the gaps between coarse CHS soundings in order to create a continuous raster of surface elevations suitable for hydrodynamic modeling. A spline with boundary interpolation was decided to be the most suitable based on the available data.

The boundary of the spline corresponded to the extent of non-water lidar returns. The spline technique ensured a smooth bathymetric surface which extended to the intertidal lidar extent despite the coarse and irregular point spacing of CHS soundings while the boundary ensured that lidar data were left un-interpolated. The final product was a seamless transition between interpolated bathymetry and terrestrial lidar. The surface was gridded at a 25 m cell resolution and was suitable for hydrodynamic modeling.

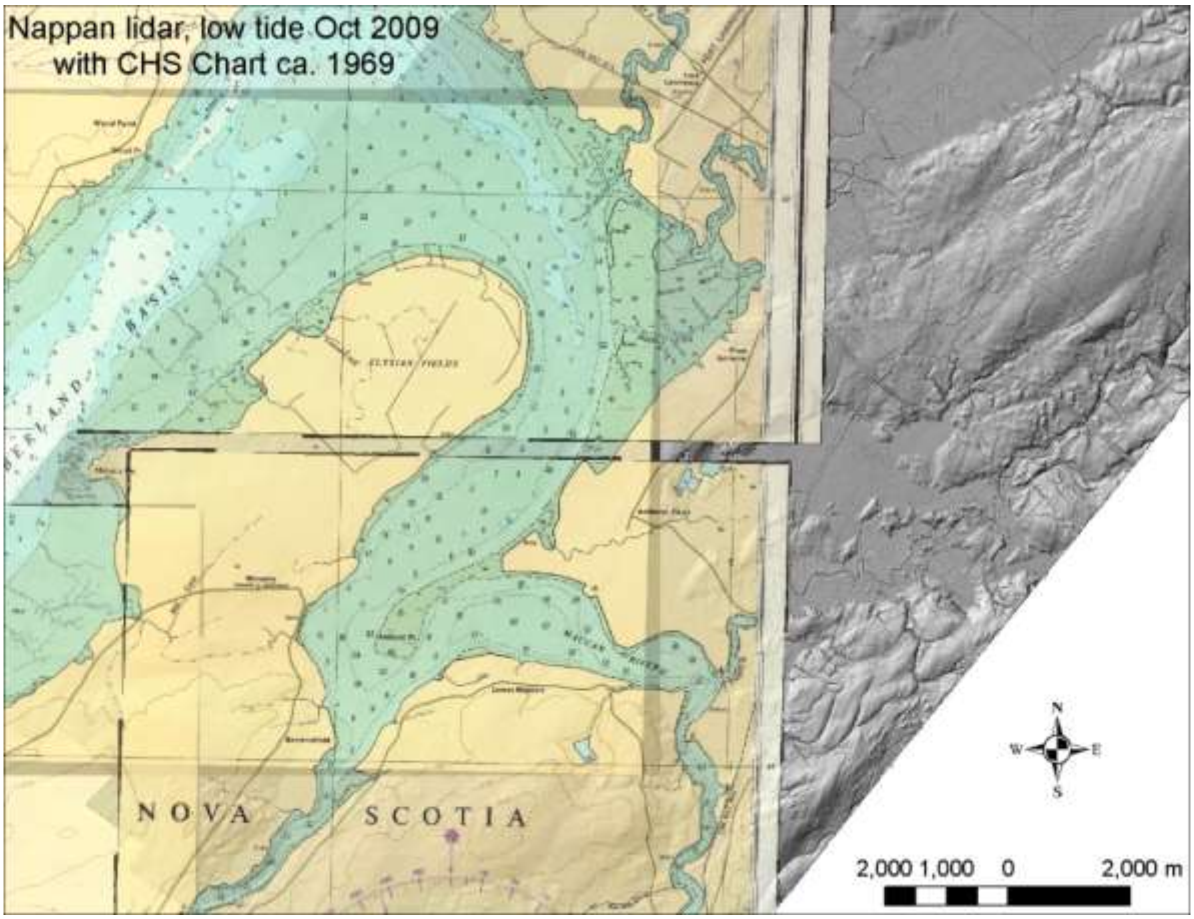

Figure 4 This map shows the Amherst area lidar DEM overlain by georeferenced Canadian Hydrographic charts. CHS charts were used to digitize bathymetry points in the upper bay of Fundy for use in the tidal model.

2.2 Environmental Data Collection and Processing

2.2.1 Weather Data

Historical weather data for Nappan were downloaded from Weather Underground, Inc. as a comma separated value (CSV) file containing the following parameters: date and time, temperature, dew point, humidity, sea level pressure, visibility, wind direction, wind speed, gust speed, precipitation, events, conditions, wind direction. Data at Nappan were available in hourly time intervals from 2004 to present. The original source of the data was cited as the Environment Canada Nappan Weather Station (Figure 5).

The Weather Underground data were compared to AGRG observations from weather stations located at Oxford and Collingwood (Figure 5) which had been established for a sister study of the Oxford watershed.

Daily minimum and maximum temperatures as well as the latitude, longitude and elevation of the Nappan weather station were used in a calculation of Evapotranspiration; an approximation of the rate of water evaporated from the system daily. More information on this equation can be found in the appendix of this report.

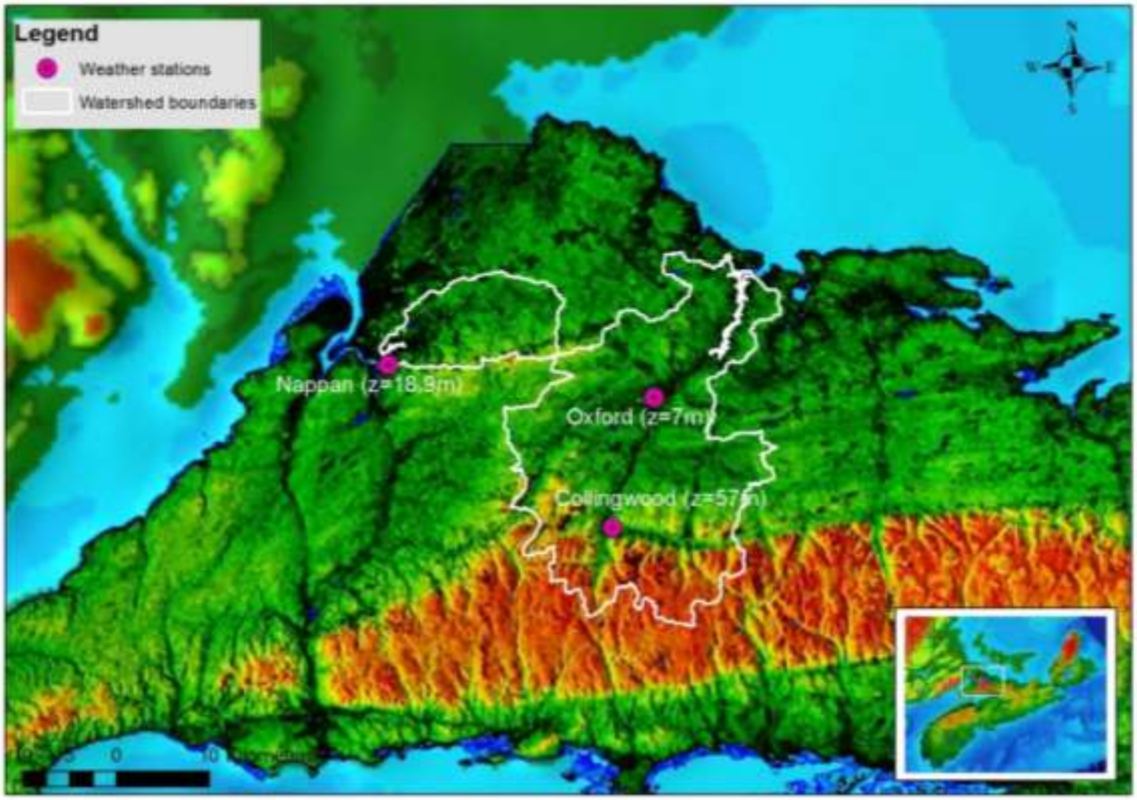

Figure 5 AGRG weather station at Oxford and Collingwood, and Environment Canada meteorological station at Nappan. The elevation of each weather station is denoted on the figure e.g. Z=18.9m.

2.2.2 Tidal Data

Figure 6 shows the locations of tidal elevation data. The CHS provided predicted tidal elevations for three stations in the Upper Bay of Fundy: Pecks Point, Grindstone Island, and Joggins Wharf for October 1998 to October 1999, and June to December 2010. Tidal elevations were measured between October 2009 and December 2010 at the Tantramar Dam, and between May 2006 and July 2010 at the Shepody Dam; data were provided by NB Agriculture and Aquaculture. Mount Allison University observed water levels at Fort Beausejour from June 15 to December 11, 2010.

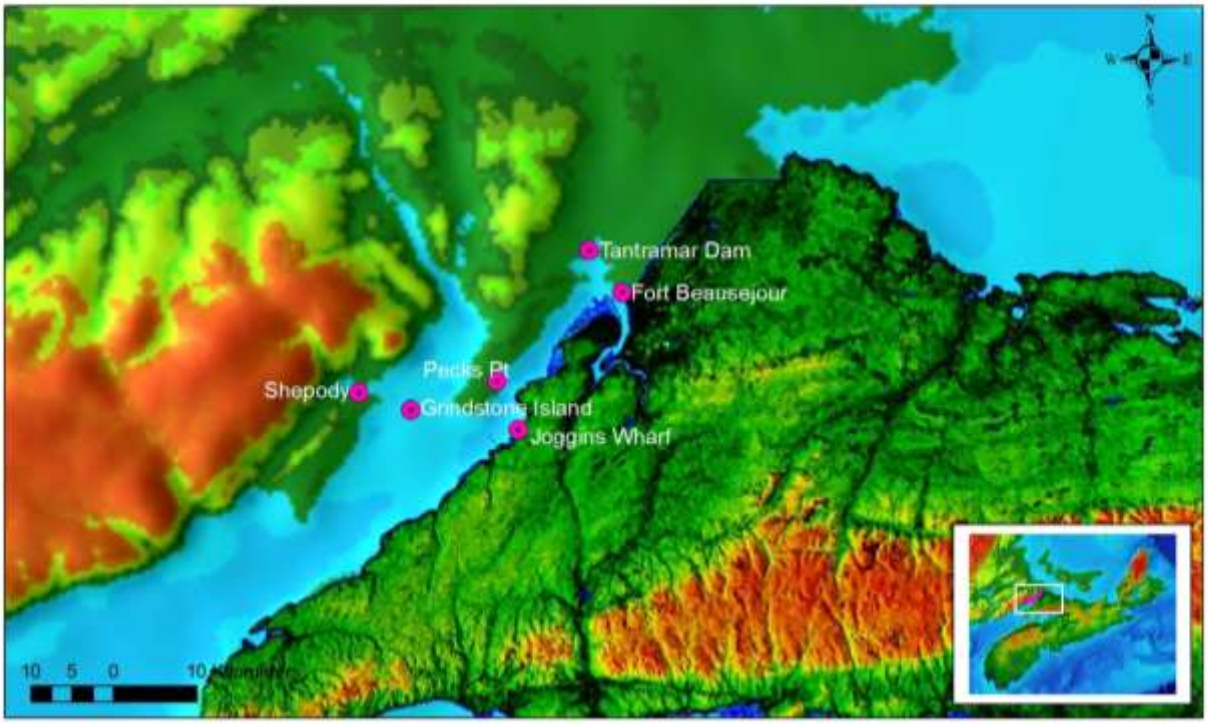

Figure 6 Locations of CHS tidal elevation stations (Pecks Point, Grindstone Island, Joggins Wharf); NB Dept. of Agriculture and Aquaculture tide gauges (Tantramar Dam, Shepody); and Mount Allison University tide gauge (Fort Beausejour).

2.2.3 In situ river Stage Measurement Procedure

A pressure transducer was installed in the Nappan River under the Lower Porter Road Bridge to capture the full range of river stage continually as of April 14, 2011. Data collected from the river stage sensor was calibrated to account for pressure changes in the atmosphere using data a barometric pressure sensor installed nearby to the river stage sensor. Periodic survey grade GPS measurements of the water elevation near the location of the pressure transducer were collected to verify the accuracy as well as establish the relationship of the river stage information to the CGVD28 vertical datum.

2.2.4 Periodic River Flow Measurement Procedure

Flow measurements were recorded systematically along the stream profile at the location of the in situ river stage measuring device at the Lower Porter Road Bridge. Measurements were taken during several different river stage conditions over the course of several months.

River flow measurements were collected every 1 m to 2 m perpendicular to flow along with channel depth and a single water level. Where the water level was less than 0.5 m, a flow measurement was recorded at 40% depth and at both 20% and 80% depth where the water level was greater than 0.5 m. Measurements where collected using a handheld Electromagnetic Flow measuring device when possible and a rope suspended impeller flow measuring system at times when the flow and/or stage were elevated. A discharge calculation was performed for each of the flow transects completed using the total cross sectional area of the river profile.

2.2.5 RTK GPS High water measurement Procedure

High precision Real time Kinematic GPS points were collected to record flood water extents as depicted by local residents for the recent flooding event on July 10, 2010 which had notably inundated the Smith Road and much of the surrounding area (Figure 12). Such real world overland water level data is always useful when determining the efficacy of flood simulation results. The survey was performed on July 7, 2011 using a Leica 1200 rover and a Leica 503 base station located over a the local HPN near Amherst. All points recorded achieved a vertical accuracy less than +/-5cm.

Figure 7 Smith Road was specifically targeted in this study as it was known to frequently experience a particularly high frequency of inundation. Shown here is a portion of Smith Road during such an inundation event.

2.3 Hydrodynamic Modeling

2.3.1 DHI Mike Hydrodynamic Modeling Tools

Hydrodynamic modeling was performed using the Danish Institute of Hydrology (DHI) suite of tools; MIKE21, MIKE 11 and the MIKE GIS extension for ESRI ArcMap.

2.3.2 Upper Bay of Fundy Hydrodynamic Tidal Model Calibration

The boundary condition was developed using deep ocean tidal predictions obtained from the Department of Fisheries and Oceans WebTide Tidal Prediction Model v0.7.1 (Dupont et al., 2005). The WebTide application was used to predict ocean elevation within the Upper Bay of Fundy using ten tidal constituents. Predictions were made at three points which were perfectly coincident with the top, middle and bottom of the Mike21 hydrodynamic model boundary. Predictions were made at 5 minute intervals for 25 years between the dates of January 1st 1990 December 31st 2015. The gaps between points along the boundary were filled using a linear interpolation for each of the 5 minute predictions. The end result was a predicted ocean elevation boundary which was suitable for forcing the simulation of high resolution hydrodynamics within the study area over the 25 year prediction period.

2.3.3 Nappan River Watershed Calculation

A new watershed extent was calculated using the suite of MikeGIS hydrological tools. The 5 m DEM, a hybrid of NTS and lidar data, was first prepared by artificially lowering the elevation values where culverts existed. This step is crucial for ensuring proper hydro-connectivity exists throughout the watershed. The resultant grid was then used in a sink filling procedure, followed by a flow accumulation algorithm and then a final watershed delineation calculation. For this project, the outlet for the watershed was determined to be the location of the aboiteau control structure. Delineating a watershed in-house was done as a cautionary measure to ensure the most accurate watershed area metric available was employed by the river-runoff component of the hydrodynamic model. The resultant watershed polygon represents the precise geographic area which drains into the Nappan River (Figure 5).

2.3.4 Nappan River Hydrodynamic Model Setup

2.3.4.1 Model Topography

Using the Euclidian distance grid generated from the lidar river water polygon as a guide (Section 2.1.4), the Nappan River center was digitized manually. This was performed using the reach digitization tool included in the MIKE GIS extension for ArcMap. Cross section locations were digitized manually using the built-in cross section digitization tool. Cross section elevation data was extracted from the 2 m lidar bathymetry hybrid DEM every 2 m laterally along each cross section line. Cross sections were placed with the following considerations: (1) cross sections were relatively evenly spaced along the chainage (the downstream distance) of the reach, (2) inflection in the cross sectional profile of the river was well represented -both in terms of changes in the width of the river as well as depth, (3) cross sections did not cross a channel reach more than once, and (4) cross sections are perpendicular to the direction of flow (Figure 8).

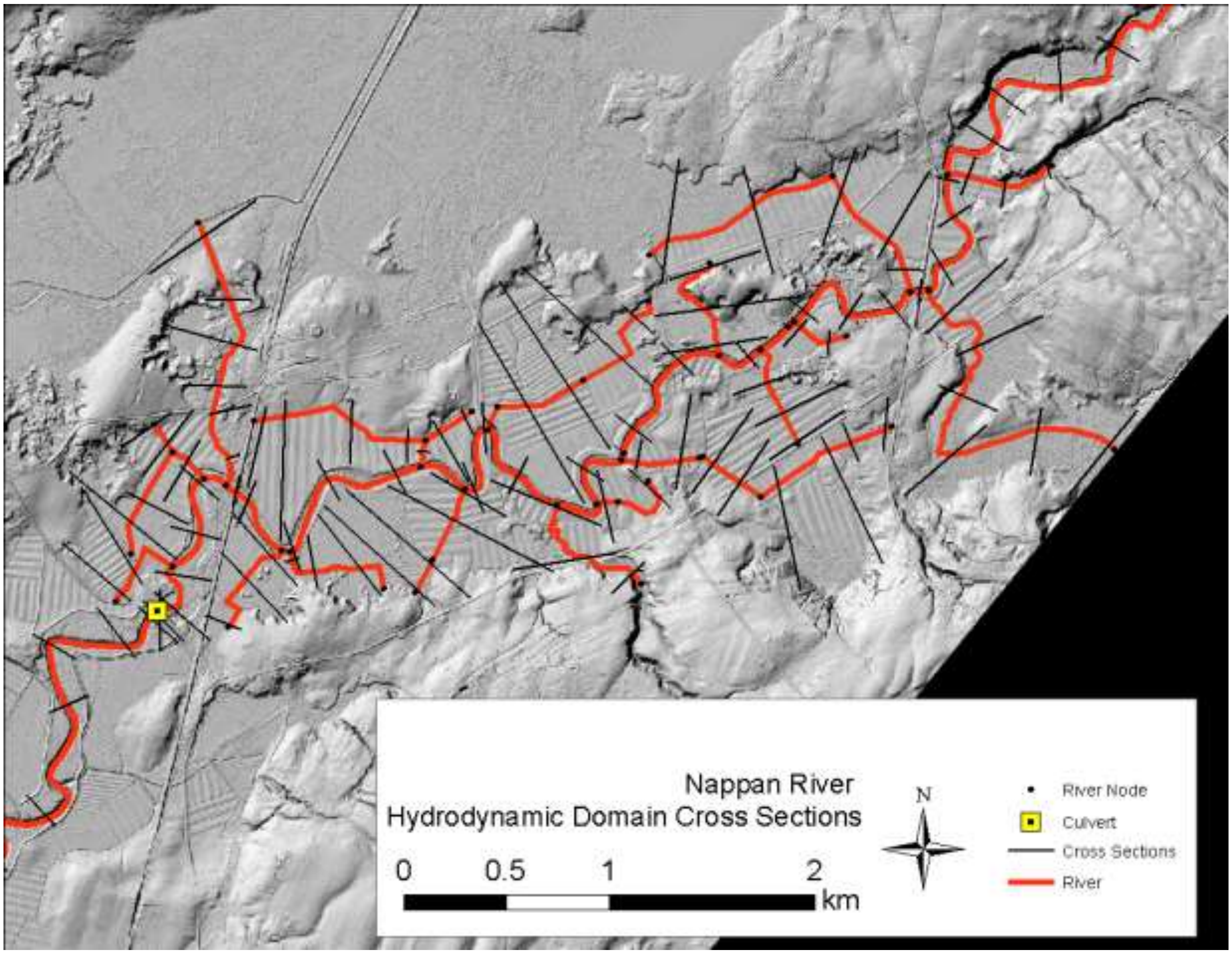

Figure 8 Map of the 1-D model design typically used by this study includes the main Nappan River channel, secondary drainage channels, all related topographic cross sections as well as aboiteau control structure.

Due principally to time constraints, the flood plain for the river system was incorporated using a 1-D hydrodynamic system as a replacement for linking the 1-D system of the river channel with a 2-D model of the flood plain. This was accomplished using a methodology similar to what was described for the digitization of reaches and cross sections outlined above for each of the secondary drainage channels within the flood plain. In addition to the considerations for cross section digitization listed above, cross sections made for secondary drainage channels were digitized in such a way as to encompass the entirety of the Nappan River flood plain to as best of a degree as reasonably possible.

All cross sections were assigned a uniform transversal distribution relative resistance value which linked to a resistance value in the larger HD model as to allow for more flexibility in the calibration step. The cross sectional area and storage width was calculated in increments of cross sectional height to be related to discharge. Conveyance curves were calculated in a similar fashion to ensure a monotonously increasing relationship between conveyance and water level which suggests good model stability for open water hydraulics.

The lowest elevation values for each of the secondary drainage channel cross sections nearest to the main river channel were lowered to match the lowest elevation value of the main river channel cross section directly before the point of intersection between the two channels. This was done as a best practice to improve the model stability.

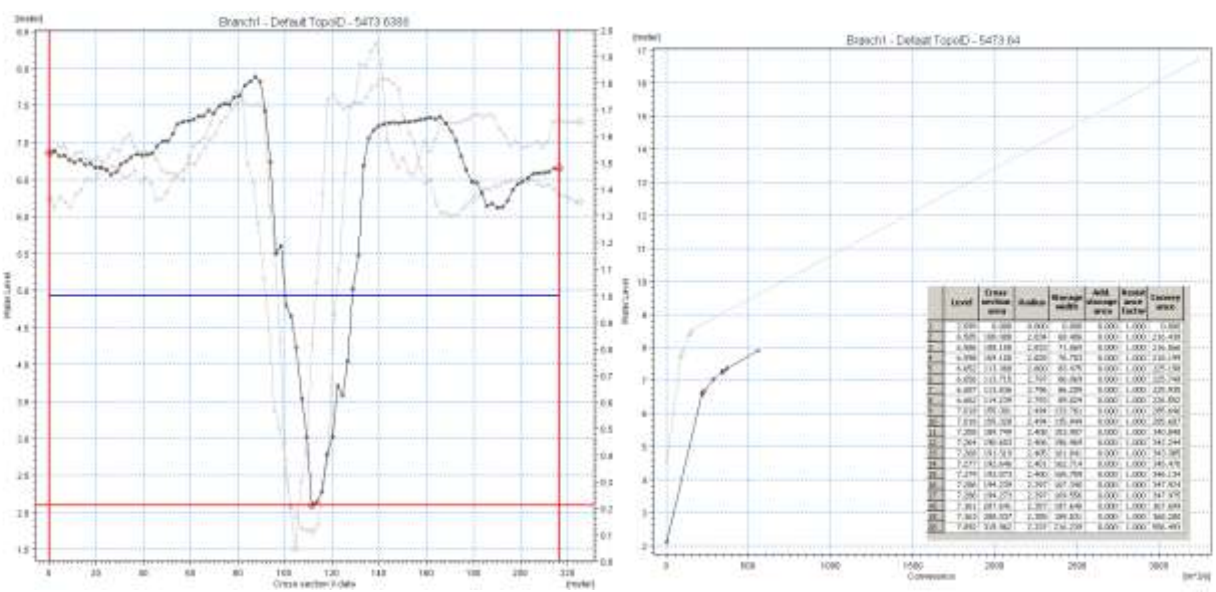

Figure 9 An example of three sequential cross sections extracted from the lidar and Bathymetry DEM (CGVD28) along with the calculated convergence curves. Sequential cross sections should have similar conveyance slopes. Inset is an example of the set of calculations made incrementally with height for each cross section.

It was determined that a channel must exist though the entirety of the main river at a depth no greater than approximately 3.0 m CGVD28. It was observed in the collected stage data that the system was capable of draining to this level. To accommodate this, a slight channel was imposed in the cross sections generated from estimated bathymetry depths of 3.0 m. This was supported by the conclusion that a small channel could be observed in the collected bathymetry data leading up to the estimated section of the river.

To represent the aboiteau structure, a two cell rectangular culvert was included at chainage 9687 (-64.2430, 45.7725 Decimal Degrees). Each cell contains a width of 3.85 m, a height of 3.048 m, and a length of 34 m. The upstream and downstream inverts of the culvert (the elevation of the base of each opening) were both determined to be 0 m CGVD28 based upon an approximation of the available data. River cross sections were extracted from the DEM for the upstream and downstream opening of the culvert. The structure simulation allows for flow only in a positive direction stopping any upstream tidal discharge entirely as well as blocking drainage in a positive downstream flow direction entirely when the downstream (tidal) water level exceeds the upstream (river) water level. An assumed Manning's *n* bed resistance value of 0.013 was used in the calculation of friction loss and ten steps of maximum discharge to cross sectional height relationships. For the purpose of model stability, each of the discharge-height values calculated assumes a fully wet culvert. Critical flow was not considered for the upstream or downstream opening of the culvert in this simulation.

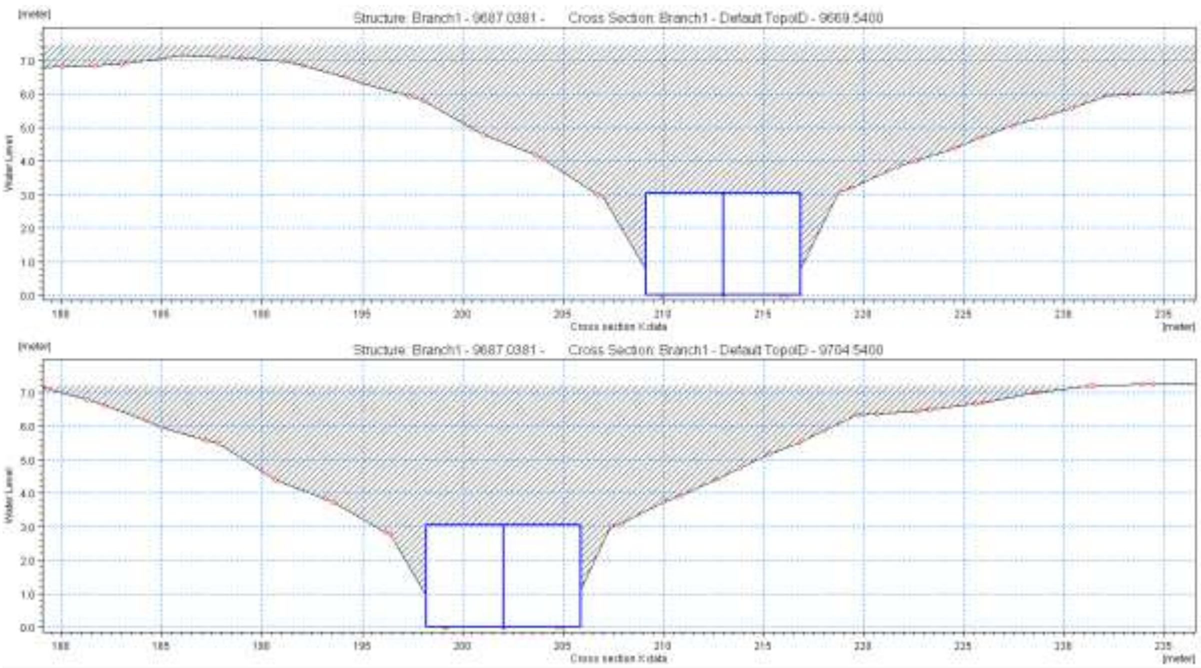

Figure 10 The upstream (above) and downstream (below) cross sections of the one way culvert feature built to simulate the Nappan River aboiteau.

2.3.4.2 Environmental Data Model Integration

A river runoff (RR) module was built for the system to distribute rainwater though the main river channel of the module per unit area of the watershed. The system chosen for calculating river runoff was a Rainfall Dependent Inflow and Infiltration model (RDII, or Nedbør-Afstrømnings-Model AKA NAM). The module incorporates parameters for storage capacity of the watershed represented in millimetres (a maximum water content in surface storage and a maximum water content in root zone storage), various runoff coefficients (an overland flow ratio and a time constant for routing interflow) as well as an input for approximate base flow discharge and several groundwater flow parameters. The watershed area metric used for the NAM model was the in-house calculated Nappan River watershed area. A step accumulated time series for both hourly precipitation and daily evapotranspiration were compiled using the Environment Canada Nappan weather station data from 2004 to 2011 and fed into the river runoff module. Such a large time series allowed for flexibility in the specification of model simulation periods. The river runoff output was linked to the river reach network from chainages 0 to 1223 of the main Nappan river channel such that the output hydrograph of the RR model would feed directly into the HD model.

A tidal water level boundary was set at the end chainage (12231) of the main river channel, 2.54 km downstream from the aboiteau in a tidal environment. Tidal elevations were taken from the MIKE 21 simulations.

2.3.5 Nappan River Model Calibration

Unfortunately, upon inspection of the water level logger data it was determined that a rating curve could not be developed. A rating curve relates river stage to discharge as a one to one relationship between discharge and river stage, which did not exist for the system. Due to the presence of the aboiteau and a resultant tidal influence on the flow rate at the location of the level logger, a consistent signature of spiking water levels was coincident with a drop in discharge (Figure 11). As such, the NAM model could not be calibrated simply using the output hydrograph (discharge over time) in relation to the recorded water level. Instead the NAM calibration was done iteratively using a full hydrodynamic simulation where the NAM values considered a full watershed extent of the river extending to the aboiteau. This calibration scheme was perhaps more time consuming than the typical rating curve method, though it functioned to ensure the accuracy of the entire hydrodynamic model equally as well.

HD calibration runs were performed over the period in which river stage data existed from April 4, 2011 at 12:00 PM until September 20, 2011 at 12:00 PM with a simulation time step of 10 seconds. Each run produced a simulated river stage output for the location of the in situ level logger of which it was compared qualitatively. After each run, a suspected erroneous NAM parameter was modified before re-running the simulation until a final model parameter suite was established. The NAM model setup functioned as a single lumped model for the entirety of the watershed.

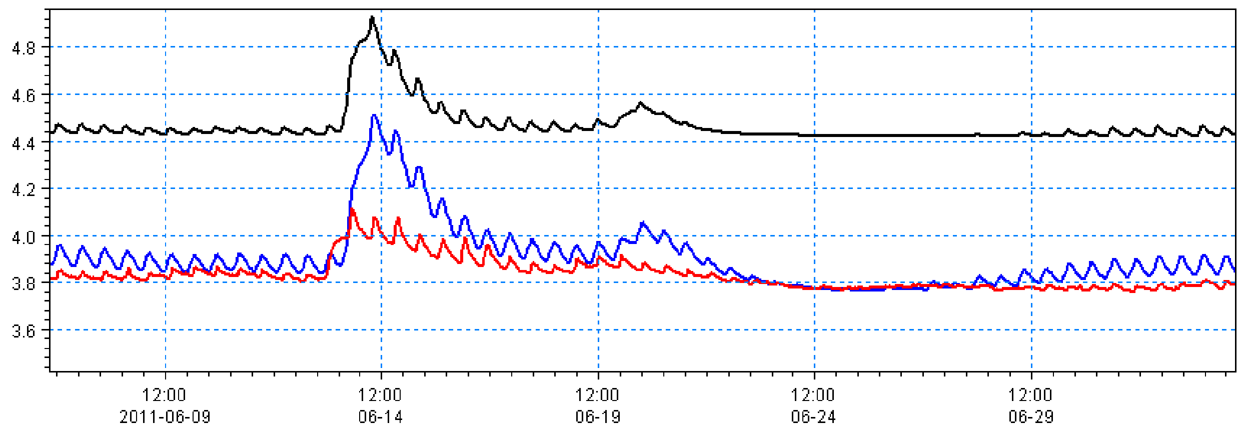

Figure 11 Shown is an example model output of water level as observed during the model calibration. The data was extracted at the location of the real world level logger data which is represented by the red line. The black line represents the previous model output while the blue a refined version of the model. The rapid undulation of water level in attributed to high tide backing up the aboiteau.

2.3.6 Nappan River Flood Simulation Procedure

A flood simulation was run for the period of June 15, 2010 at 1:00 AM to July 30, 2010 at 11:50 PM to recreate the flooding event which was documented to have effected to Smith Road. The simulation was run using the final model parameters from the calibration simulations at a similar 10 sec time step and no modifications were made to model topography. A unique tidal boundary was generated for the flood simulation using MIKE 21 for the appropriate time period. The tidal boundary water level was increased by 55 cm to remain consistent with the calibration simulation. Map outputs were generated for maximum water levels by interpolating simulated maximum water levels between cross sections and intersecting the 2 m lidar DEM. Flood simulations were run for scenarios including observed rainfall, times two observed rainfall, and times three observed rainfall in order to observe the effect of different levels of flooding in the area.

2.3.7 Nappan River Flood Mitigation Simulation Procedure

This study included several simulations of flood mitigation activities. To illustrate the flood vulnerability of Smith Road, a one way culvert was simulated by placing a structure at the mouth of the channel in the model river network which drains into the main river near the

Highway #2 Bridge as it crosses the Nappan River. Simulations were ran for the June 15, 2010 at 1:00 AM to July 30, 2010 at 11:50 PM time period to simulate two times observed rainfall and three times observed rainfall.

To simulate the effect of dredging the heavily sedimented portion of the river as it approaches the aboiteau, model cross sections downstream were modified to reflect cross sectional area of the noticeably unsedimented portion of the upper river. Simulations were run for the June 15, 2010 at 1:00 AM to July 30, 2010 at 11:50 PM time period to simulate two times observed rainfall and three times observed rainfall. Sediment removal was performed by modifying cross sections (Figure 13).

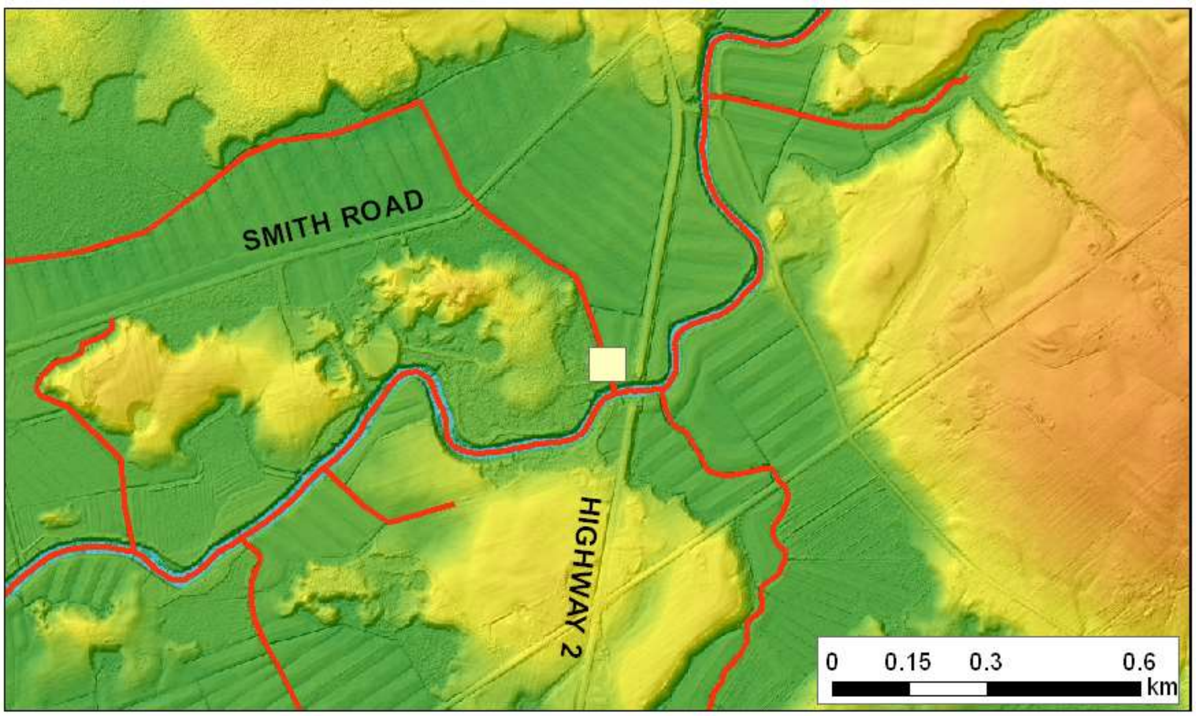

Figure 12 The location of the simulated one way culvert placed as a mitigation strategy to protect Smith Road. The graphic is north oriented and model river channels are indicated in red.

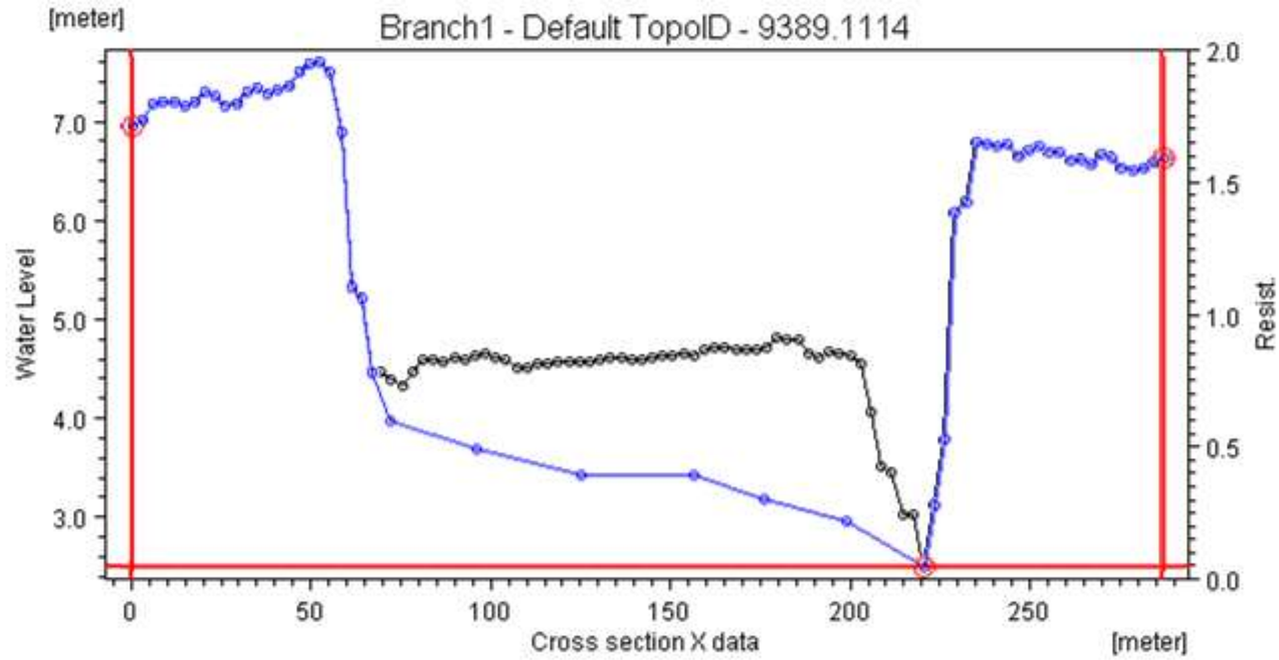

Figure 13 Each cross section demonstrating sediment buildup was lowered to convey larger discharge rates and also house a larger total storage capacity.

3 Results

3.1 Topographic Data Validation

Lidar validation has been provided under the ACAS lidar processing and flood risk report (see Webster et al. 2012). The derived watershed boundary presumes the accuracy of the NSTDB 20 m elevation model as provided. River depths used in the model, either derived from collected bathymetry or estimated by channel width, were compared to cross sections which were physically from the three bridges which cross the Nappan River. Though not thorough enough to infer a statistically significant accuracy of the bathymetry data, the physically measured profiles suggested good bathymetry estimation at each of the bridges – where the data typically showed a deviation of around 0.10 m, which is within the precision of the sonar equipment used in bathymetric collection.

3.2 Weather Data Validation

Temperature, pressure and wind direction of the Environment Canada station at Nappan (collected from The Weather Underground digital warehouse) and the deployed AGRG weather stations in the Oxford area agreed well (Figure 14, panels 1, 2, 5). Differences in elevation and proximity to the coast could account for the difference between precipitation and wind speed measurements between Nappan and Oxford (Figure 14, panels 3 and 4).

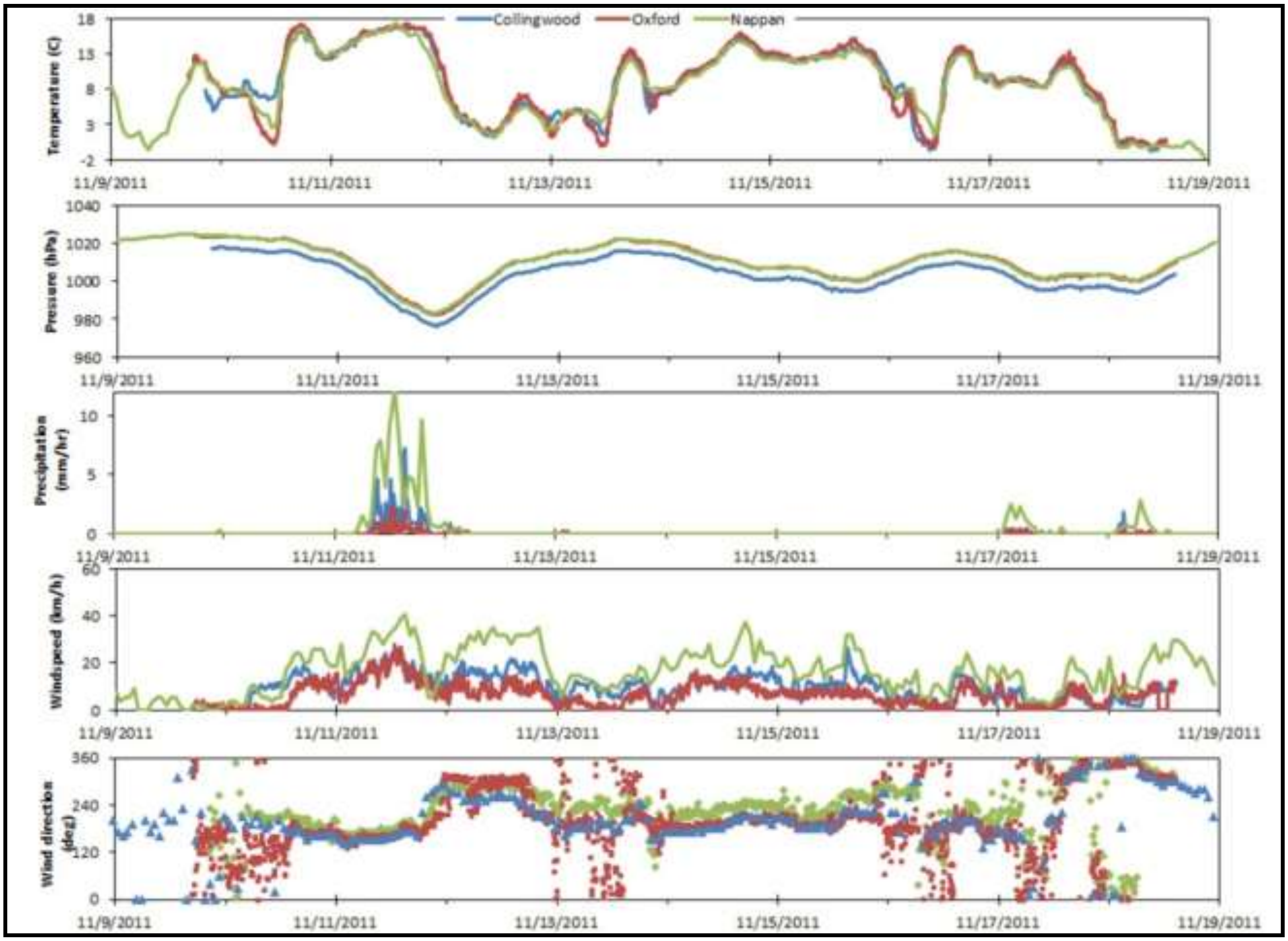

Figure 14; AGRG weather station observations from Collingwood and Oxford compared to EC observations from Nappan

3.3 Tidal Model Calibration Results

The 2-D hydrodynamic tidal model for the Upper Bay of Fundy was validated against the provided tide gauge records from the Fort Beausejour and the Tantramar. This was performed to support the assumption that reasonable tide level data, exhibiting a proper tidal period, could be extracted from the model at near shore locations such as near to the Nappan River Aboiteau. Results were favorable and deemed sufficiently accurate in terms of the assumed limitations of this methodology (Figure 15).

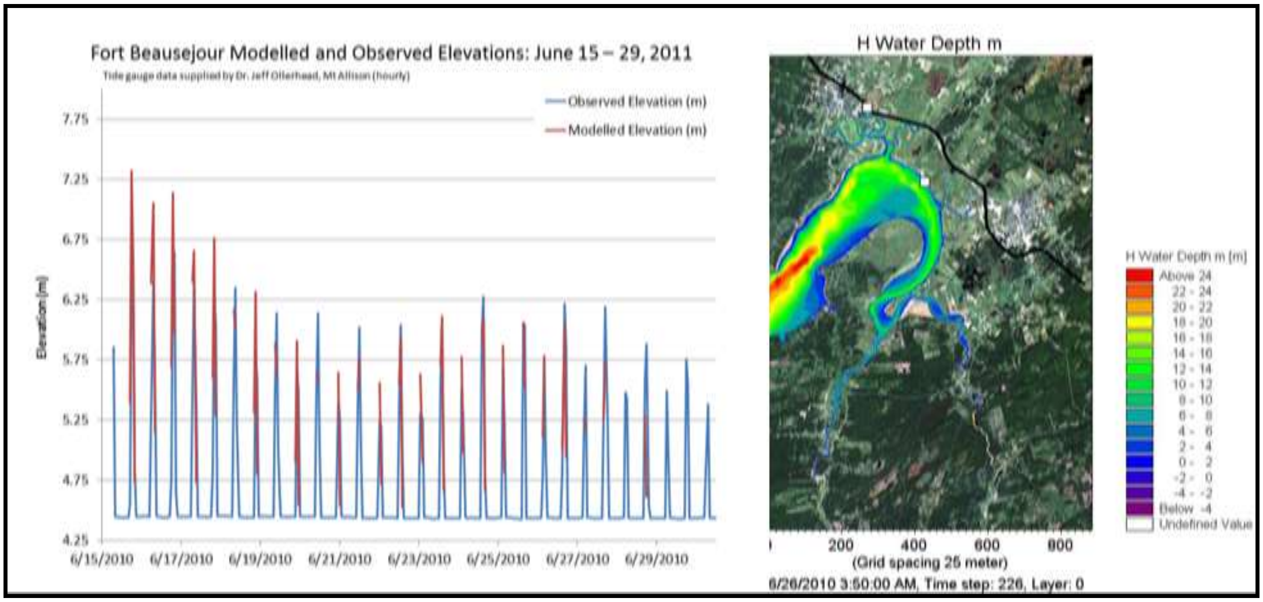

Figure 15 Comparison of water levels at with data from Fort Beausejour. Right image shows the bathymetry of the Upper Bay with a true colour composite Landsat image, white box denotes the location of the tide gauge. A tidal model of the Upper Bay of Fundy was needed to control gates on the aboiteau at the Nappan. Model boundary was derived from Website (DFO) and does not currently take into account storm surge or wind.

3.4 River Model Calibration Results

The watershed was very complicated with the lower discharge controlled by an aboiteau and the upper level influenced by a large dam. We visited the site several times in 2011 and measure river flow and stage as well as GPS elevations of the water level (Figure 16). We measured the river at various levels of stage and tide levels (Figure 16).

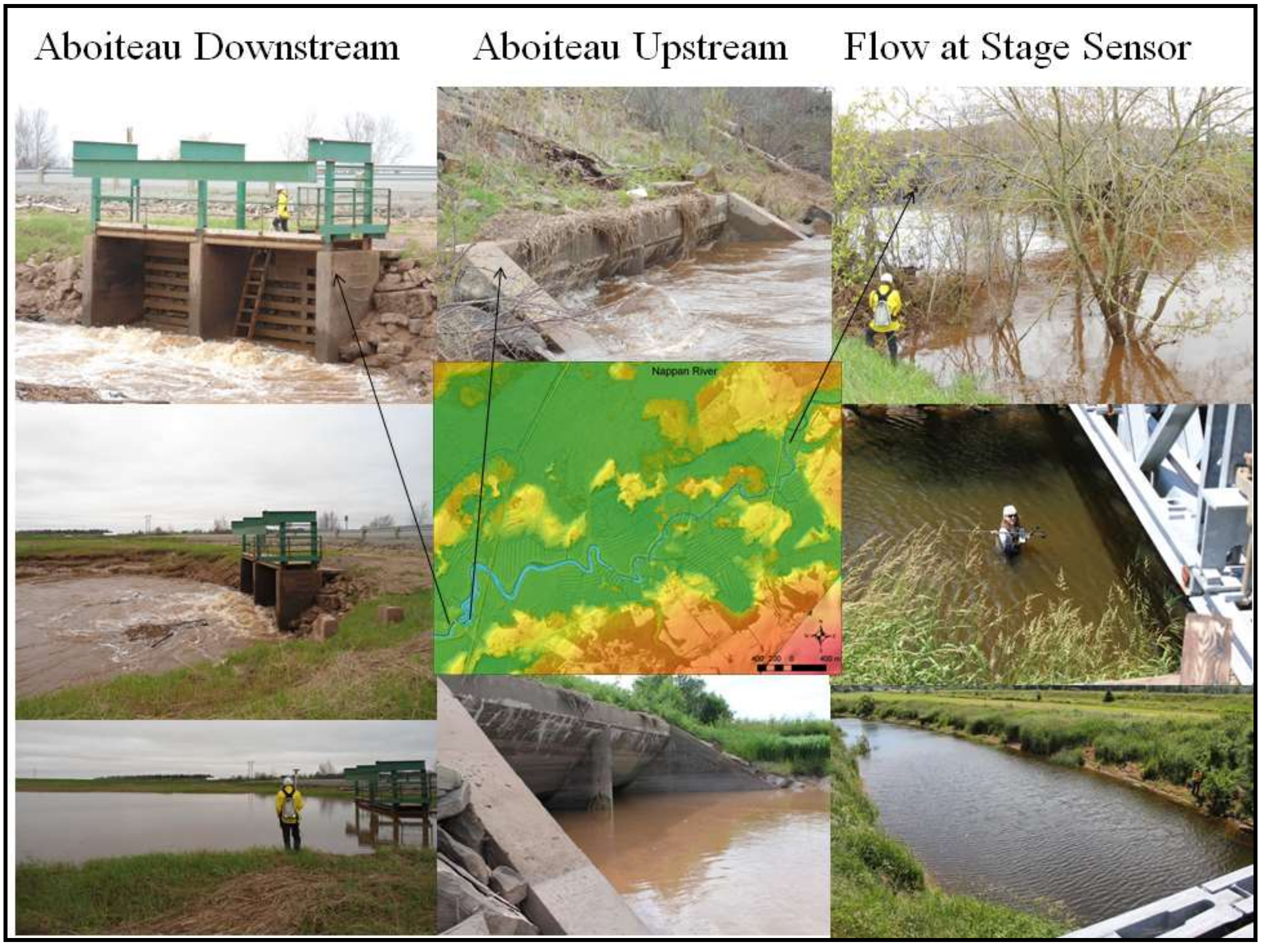

Figure 16 Example of field activity 2011. Left panel show the downstream view of the aboiteau. Center panel shows the upstream view of the aboiteau and lidar map. Right panel is Lower Porter bridge at different river stage (location of the pressure sensor).

For several events in the summer of 2011, we simulated the predicted tide to control the aboiteau gate that permits the discharge of the river during high tide. A time series of the tide was used as a boundary condition in the river Mike 11 model. The stage information measured at Lower Porter Road Bridge displayed the signal of the aboiteau operation with increases in stage of approximately 1 m during normal flow conditions. The stage begins to rise at the pressure sensor when the tide is almost at a maximum ca. 0. 5 m CGVD28 and the stage peaks when at the midpoint of the tide ca. 0 m CGVD28 (Figure 17).

A detailed list of the finalized river model parameters deduced during the calibration phase can be found in the appendix of this report.

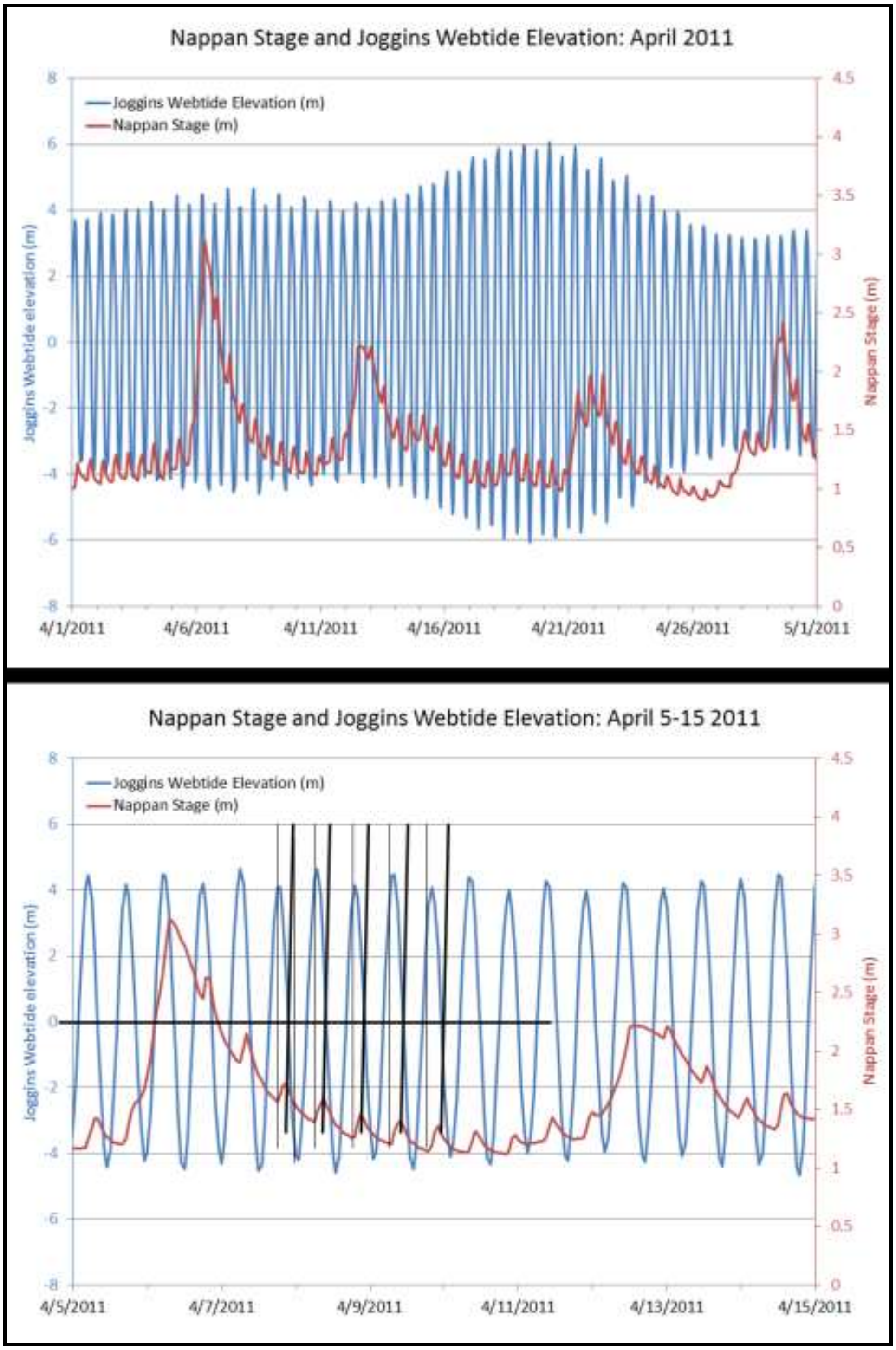

Figure 17 Predicted tide from Mike 21 model with stage measurement from Lower porter Road Bridge. The top graph is from April 1 to May 1, 2011. The bottom graph, April 5-15, 2011, thin black line denotes the beginning of stage increase near high tide. Thick black line denotes stage maximum near mean tide level.

The tidal signature observed in the collected river stage data was useful in assuring a reasonably modeled rate of discharge for the simulated aboiteau. Furthermore, the presence of this signature in the modeled results suggested a good bathymetry approximation was being applied throughout each of the cross sections between the location of the level sensor and the aboiteau, as well as the height implemented for the aboiteau drainage channel itself. In effect the channel topography was partially validated as the entirety of the channel system together was filling to a proper level and draining at a proper rate with individual tidal cycles. The model, however, tended to over represent the dampening effect from neap tides of the lunar cycle on the tidal signature as observed upstream.   This effect was alleviated by increasing the overall water level of the tidal boundary (as generated from Mike 21) by 55 cm.

The final modeled river stage output at the location of the gauge sensor for the calibration time period between April 10, 2011 and September 15, 2011, had an average difference from the collected stage of 26.2 cm; when sampled at hourly intervals. Much of that difference is attributed to initially low model run up water levels contemporaneous with seasonably high waters associated with spring melt. By removing this bias, the average difference in water levels between June and September is instead 2 mm, with a standard deviation of 10 cm; when sampled at hourly intervals.

Some examples of specific deviance between the modelled and observed water levels at the sensor location could be attributed to missing precipitation data records, as is evidently the case in late-mid July (Figure 19).

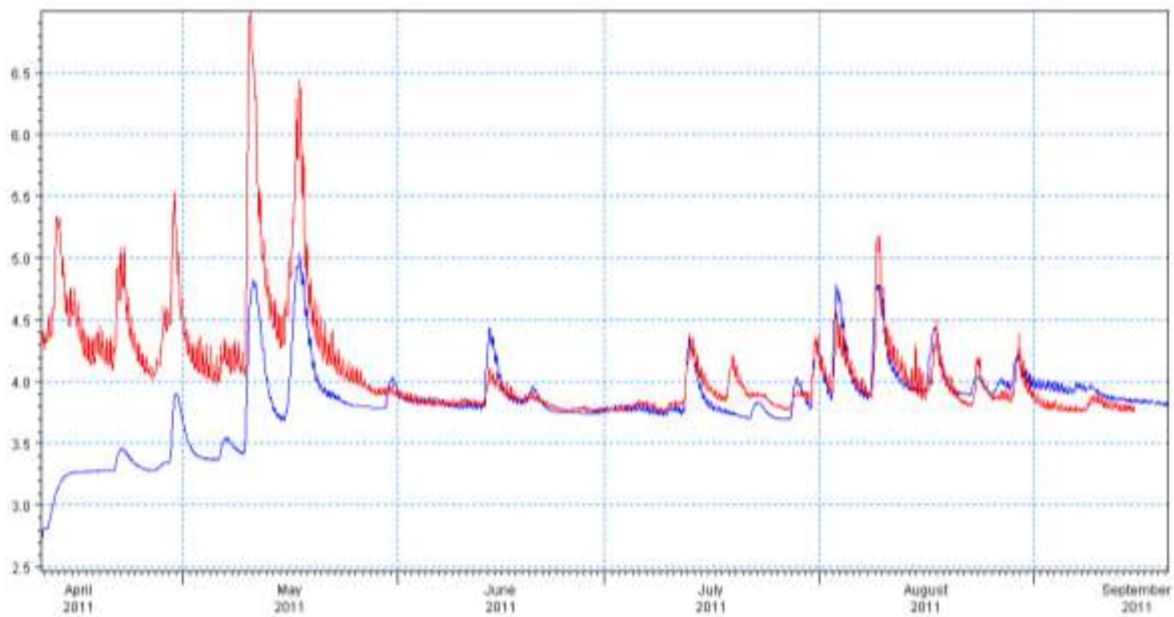

Figure 18 The calibrated model output of river stage elevation (m, CGVD28) at the location of the level logging device, shown as a blue line. The observed river stage (m, CGVD28) data is shown in red. This output simulates the entire time period of data collected from the level logging device.

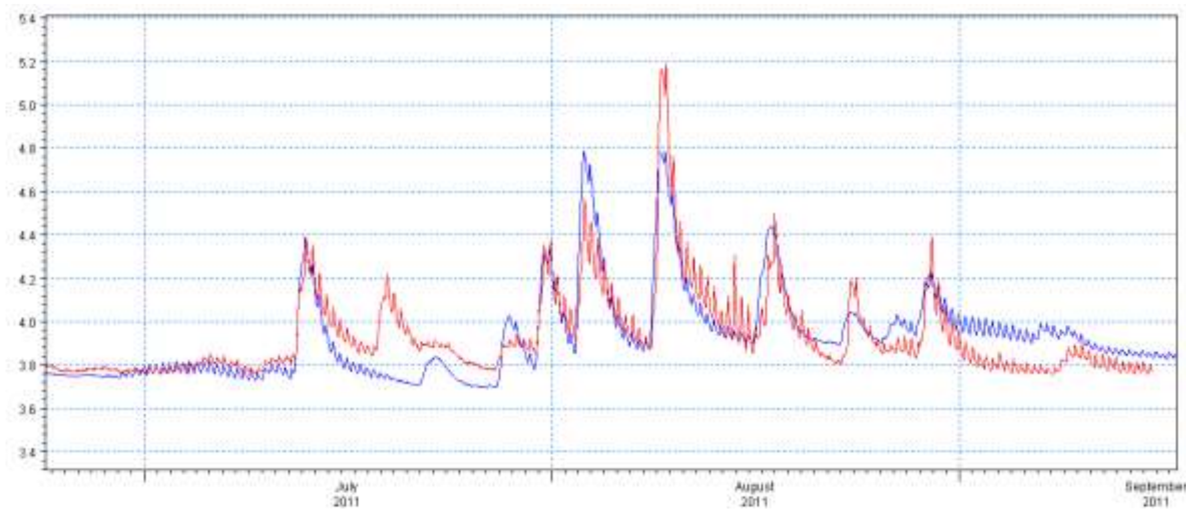

Figure 19 The calibrated model output again, of river stage elevation (m, CGVD28) at the location of the level logging device, shown as a blue line. The observed river stage (m, CGVD28) data is shown in red. Here the simulation period of July 2011 to September 2011 is observed to illustrate the accuracy of the model in excluding the effect of spring thaw and model run up. Note the high frequency signature of tide interactions with the aboiteau backing up the river, along with the lower frequency water level maxima as results of high precipitation events.

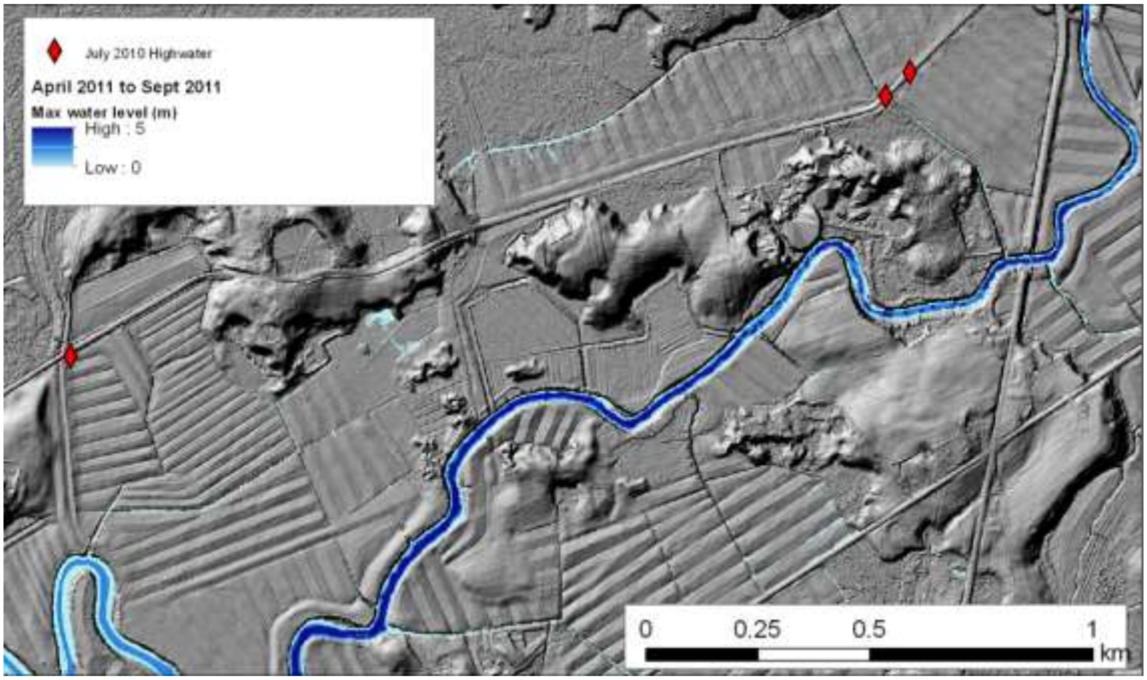

Figure 20 Shown is the maximum water level modelled during the calibration period as it would intersect with the 2m lidar flood plain elevation model. No flooding was reported during the summer of 2011.

The total output high water area of this model, 0.387 km^2 was used as a baseline for calculating inundation statistics. This value represents the area of the river channel.

3.5 Flood Simulation Results

With the calibration of the river model assumed to be adequate as outlined, several flood simulations were calculated – specifically for the July 2010 documented inundation event.

Initial results for the simulation were unfavorable, showing only a slightly higher maximum water level than the calibration simulation and no major overtopping of the banks or inundation of surrounding fields, a total of 0.022 km^2 affected (Figure 21). There are several possible contributors to the misrepresentation of the July 2010 flood, including a lack in the precipitation data – either some precipitation events missing or existing data being inaccurate - or some effect on the storage capacity for the watershed was not adequately observed and accounted for during the calibration period.

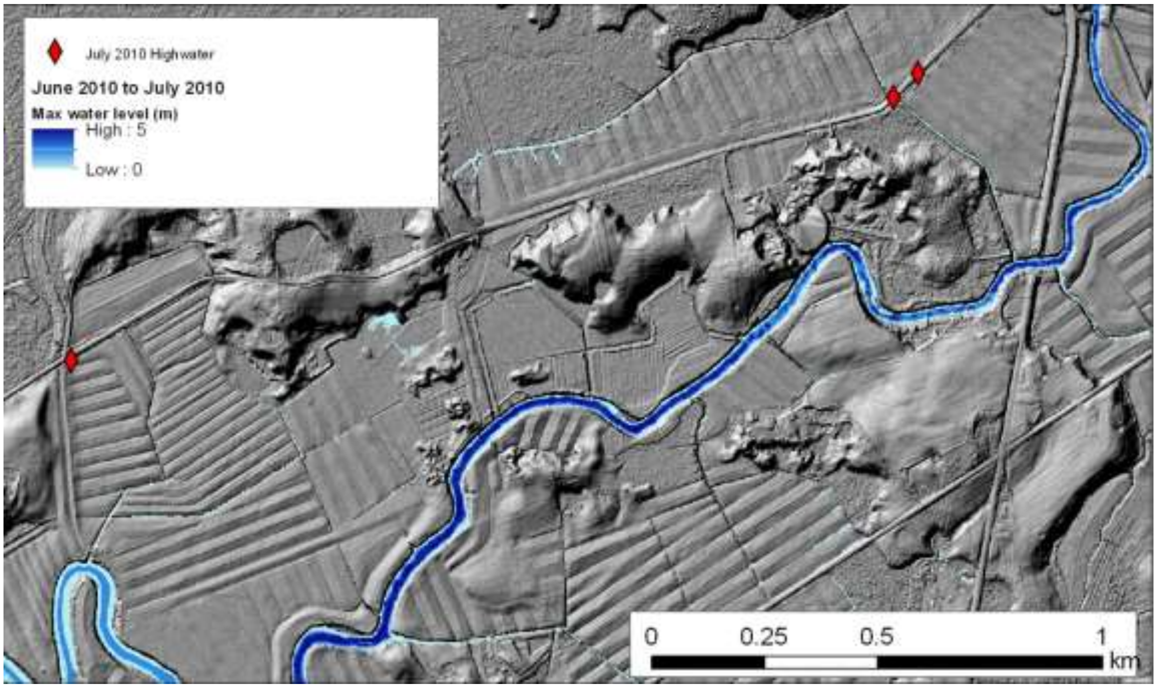

Figure 21 Shown is the maximum water level modelled during July 2010 as it would intersect with the 2m lidar flood plain elevation model. The red markers indicate the flooding extent of the July 10th event as indicated by local residents.

As such, and to ensure the recreation of flood scenarios, several additional July 2010 simulations were performed with elevated precipitation rates.

A simulation of the July 2010 precipitation, tide and evapotranspiration, whereby precipitation values were multiplied by three, yielded a total area affected by inundation of 1.36 km^2 with a maximum water level reached of 6.79 m CGVD28 (Figure 23).

A simulation of the July 2010 period precipitation, tide and evapotranspiration, whereby precipitation values were multiplied by two, yielded a total area affected by inundation of 0.163 km^2 with a maximum water level reached of 5.90 m CGVD28 (Figure 22).

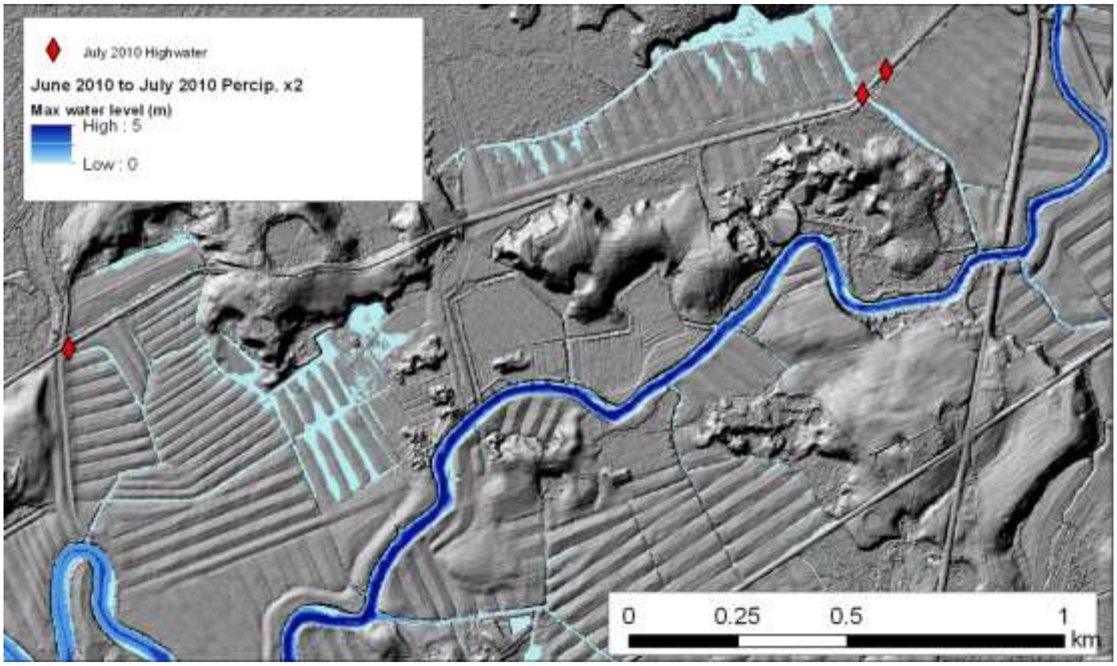

Figure 22 Map output of the July 2010 flood simulation showing maximum flood extent where precipitation values doubled. Red markers indicate observed flood inundation extent of July 10, 2010.

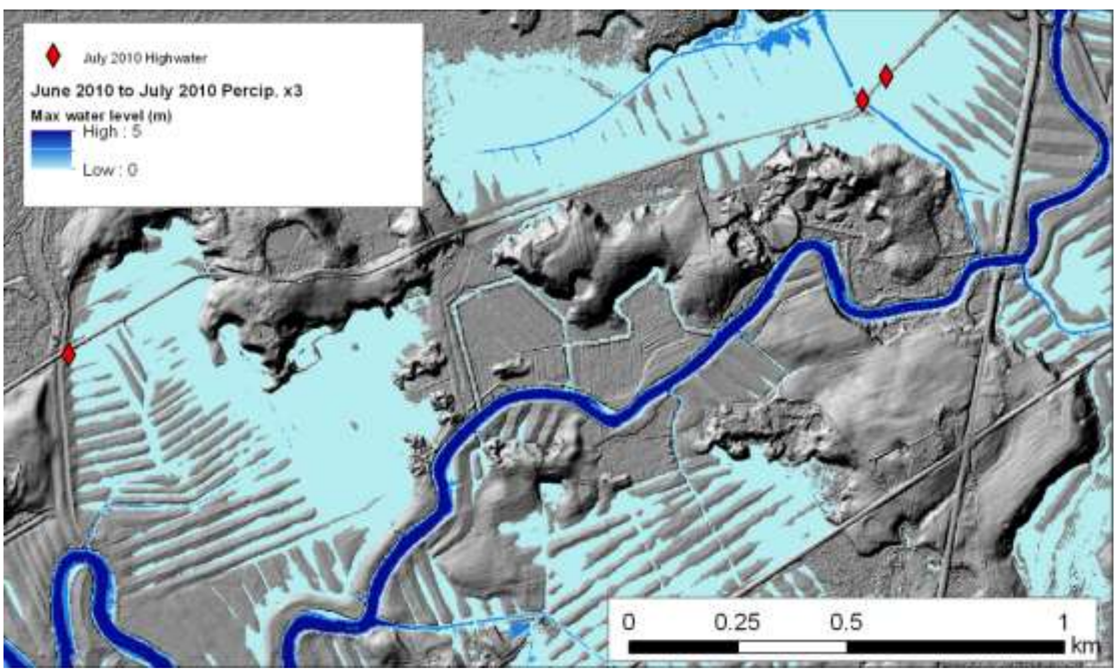

Figure 23 Map output of the July 2010 flood simulation showing maximum flood extent where precipitation values doubled. Red markers indicate observed flood inundation extent of July 10, 2010.

3.6 Mitigation Strategy Results

3.6.1 One way Culvert Simulation

Specific flood scenario simulations were run in which a simple one way culvert was modeled to protect Smith Road from flood hazard.

A simulation of the July 2010 period precipitation, tide and evapotranspiration, whereby precipitation values were multiplied by two, with a one way culvert installed yielded a total area affected by inundation of 0.132 km^2 with a maximum water level reached of 5.91 m CGVD28 (Figure 24). In this scenario, the one way culvert protected 0.031 km^2 of land from inundation while raising the maximum water level reached in the simulation by 1mm, or nil.

A similar simulation whereby precipitation values were multiplied by thee yielded a total area affected by inundation of 1.071 km^2 with a maximum water level reached of 6.89 m CGVD28 (Figure 25). In this scenario, the one way culvert protected 0.287 km^2 of land from inundation while raising the maximum water level reached in the simulation by 10.1 cm.

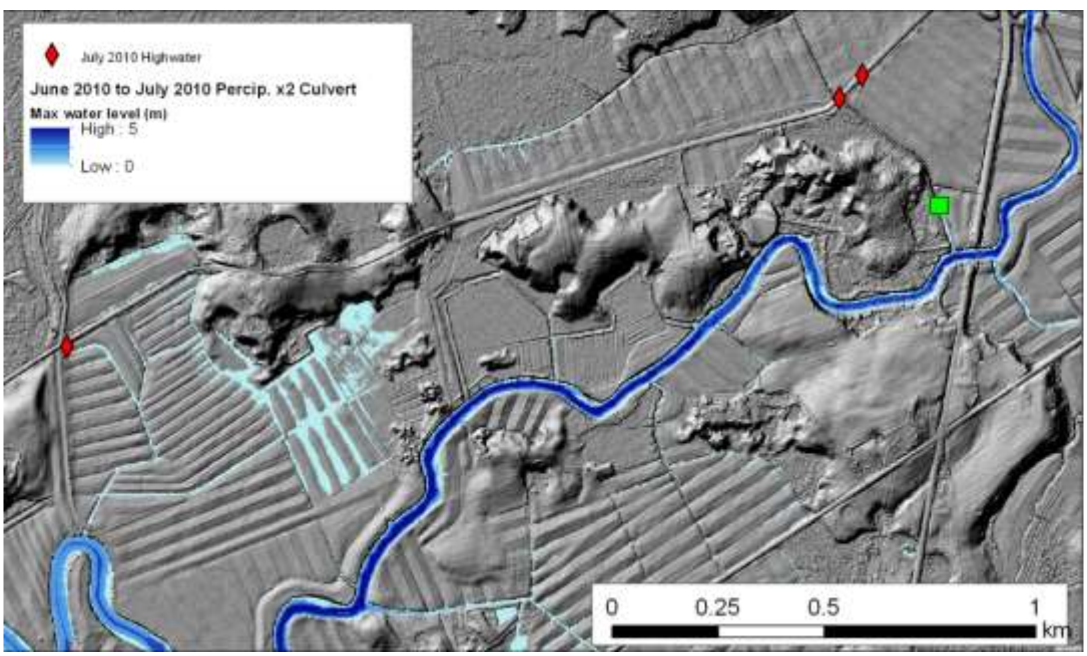

Figure 24 Map output of a flood simulation whereby a one-way culvert was installed to protect Smith Road (green square). The output shows maximum flood extent for double the precipitation July 2010. Red markers indicate observed flood inundation extent of July 10, 2010.

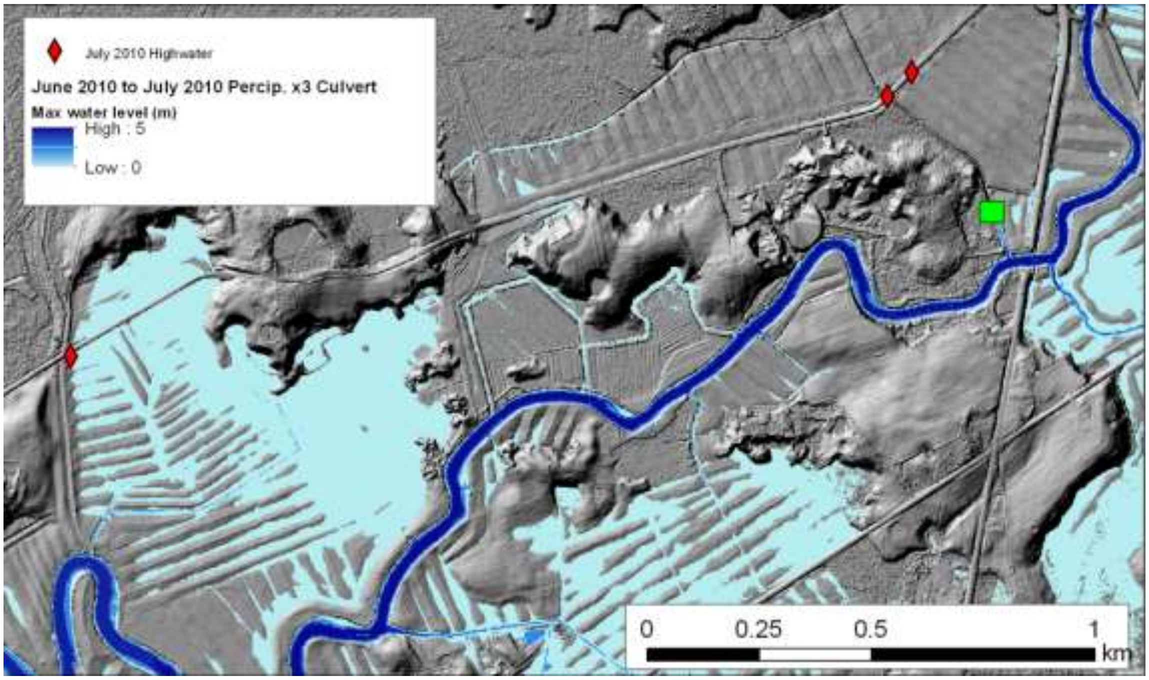

Figure 25 Map output of a flood simulation whereby a one-way culvert was installed to protect Smith Road (green square). The output shows maximum flood extent for triple the precipitation July 2010. Red markers indicate observed flood inundation extent of July 10, 2010.

3.6.2 Sediment Removal Simulation

A simulation of the July 2010 period precipitation, tide and evapotranspiration, whereby precipitation values were multiplied by two, with sediment dredging as described yielded a total area affected by inundation of 0.051 km^2 with a maximum water level reached of 5.56 m CGVD28 (Figure 26). In this scenario, dredging protected 0.111 km^2 of land from inundation while lowering the maximum water level reached in the simulation by 0.338 m.

A similar simulation of the July 2010 period precipitation, tide and evapotranspiration, whereby precipitation values were multiplied by three, with sediment dredging yielded a total area affected by inundation of 0.788 km^2 with a maximum water level reached of 6.66 m CGVD28 (Figure 27). In this scenario, dredging protected 0.570 km^2 of land from inundation while lowering the maximum water level reached in the simulation by 0.134 m.

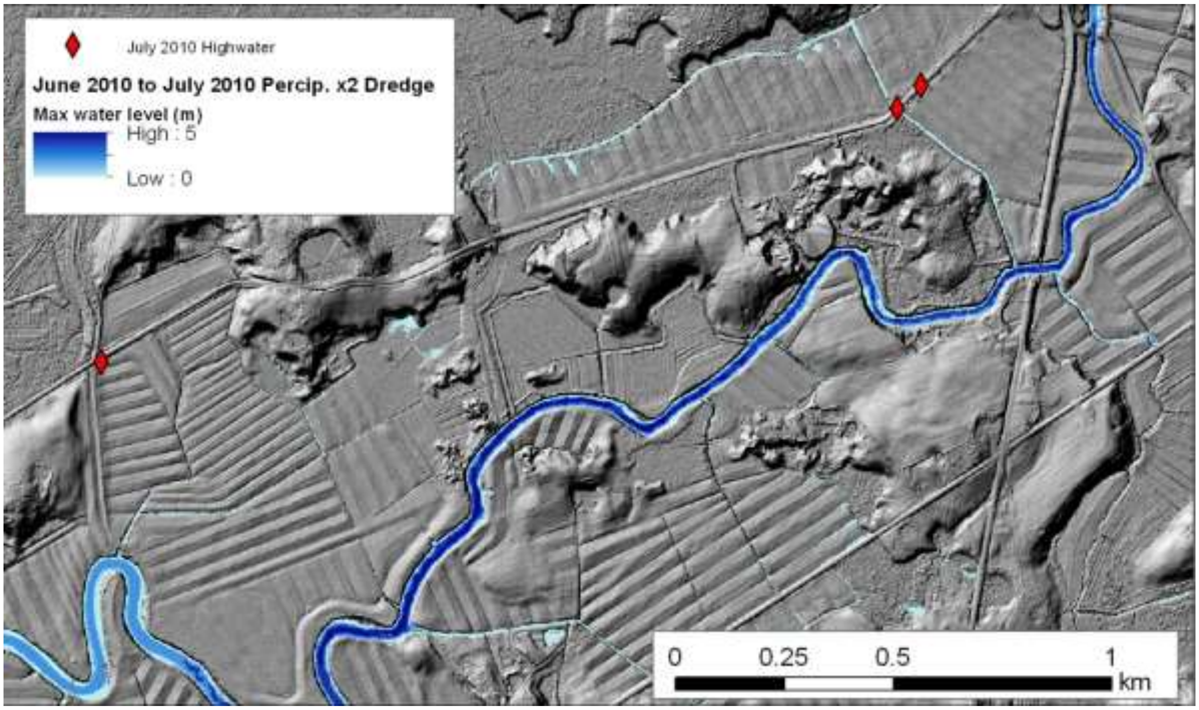

Figure 26 Map output of the dredging simulation showing maximum flood extent for double the precipitation July 2010.

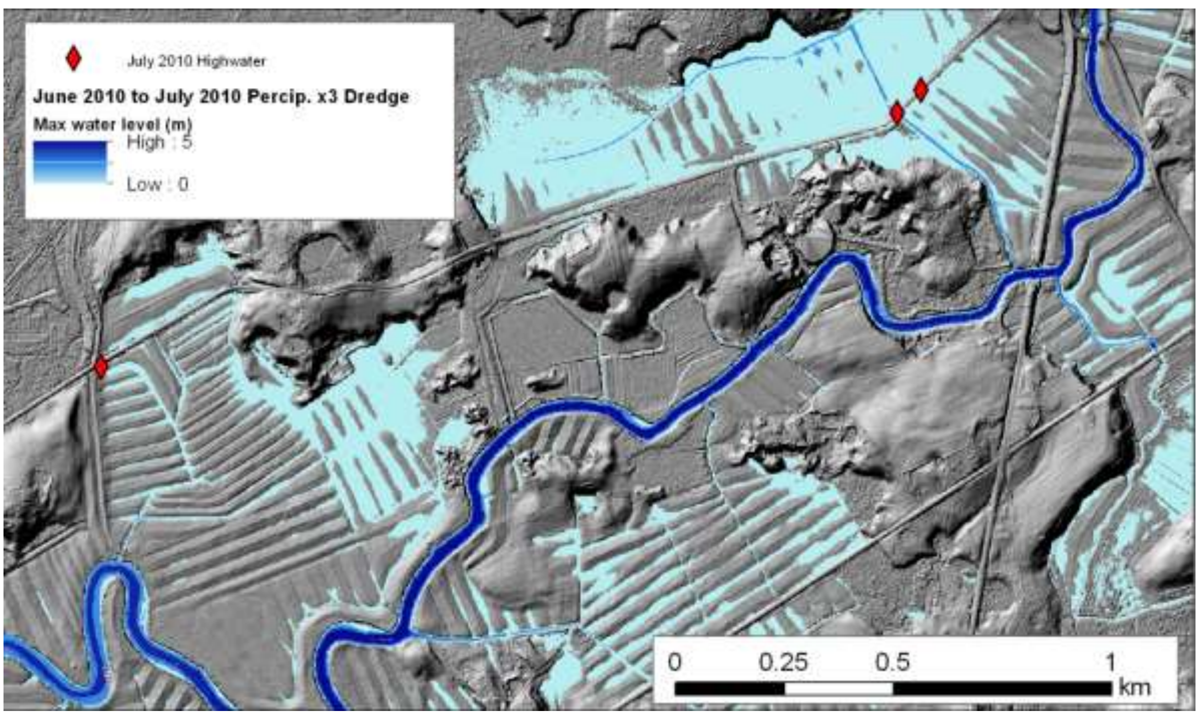

Figure 27 Map output of the dredging simulation showing maximum flood extent for triple the precipitation July 2010.

4 Discussion and Conclusions

Considering the complex nature of the interaction of precipitation, temperature, watershed characteristics and tidal influence in this hydraulic system, the calibration results could be considered satisfactory for the scope of this project. Though the complex tidal signature observed in the river stage data was initially presumed to be a burden on the project, based upon pervious river modeling work, it may have in the end been of great benefit to project as a whole. By calibrating the system using full hydrodynamic processing as opposed to a single mass balance Rainfall Dependent Inflow and Infiltration calculation, we can assure that our simulated aboiteau structure is functioning reasonably well within the context of this project.

Our method of simulating tidal water elevations was required in lieu of having a tide gauge located seaward of the aboiteau within a reasonable distance. While this method does allow for the hind casting of tidal scenarios based on tidal constituents, and thus flexibility in the time periods of events we are capable of simulating, it does not take into account the effect of storm surges or any residual high and low tidal water levels deviant from predicted tide. This factor may contribute to an explanation for the necessity of increasing the simulated tidal boundary water elevation by 55 cm to best simulate the river backup at the location of the in situ logger.

The apparent underestimation of the July 2010 flooding event in simulation may, based upon the generally good calibration 2011 data, be the result of poor precipitation data for the time period in terms of its representation of the watershed as a whole. In essence, precipitation quantification by means of a device with a small foot print, such as a rain bucket, would likely not necessarily capture the reprehensive rainfall throughout a watershed reasonably, despite the accuracy of the device itself. A contributing factor in the simulations underestimation of water level may as well be the lack of model run-up time as compared to the calibration model simulation. This is decidedly less likely the case as a longer time period simulation was quickly ran with an artificial tidal bound with similar results.

Currently, a general oversight - and perhaps a contributing factor to the July 2010 flood underestimation – is the exclusion of any specific parameters relating to the spillway located upstream from the in situ gauge. The spillway damn was not included in the final project design principally due to time constraints and as the calibration seemed to function well regardless. Any specific effect the spill way has on the system could, though, presumably be amplified during a particularly high water event which was perhaps not sufficiently captured during the calibration period.

Given the one-dimensional design of the model, the simulated installation of the one way culvert to protect Smith Road as described would of course function as required because the only channel by which water may reach Smith Road is, by effect of the culvert, blocked entirely from upstream flow. In a two-dimensional model setup, water would be capable of widespread overland flow in any directions as permitted by the 2-D elevation model whereas in a one-dimensional system, water may only flow in preordained channels and cross sections as digitized by the user. This is not to say that the presented depiction is inaccurate as there appears to be no clear path other than the selected channel by which water may infiltrate smith road sooner. This limitation should be taken into consideration, though, and currently applies thought the rest of the model as well.

This project, and the hydrodynamic model created for it, represents a steep learning curve and the compilation of many exhaustive and complex data sets. Many of the developed tools and practices stemming from this project may be now more easily employed in other water ways or as to improve the now existing model. Despite the oversights of this model as described, the general topography and hydrodynamic properties of the river system are considered to be well replicated and physical flooding is likely well represented - despite the particular environmental forcings applied.

5 Recommendations

This project has involved the development of a calibrated watershed rainfall-runoff model that drives a 1-D river hydraulic. We had to employ a 2-D hydrodynamic model to simulate the action of the aboiteau. Another suite of model linkages available from DHI that was beyond the scope of this project that involves Mike FLOOD, a two-dimensional flooding model that can integrate the 1-D river hydraulic model and the ocean 2-D model. The benefits of this model are that instead of working with 1-D cross-sections of the floodplain, this model uses a 2-D elevation grid of the floodplain to determine the flooding. Now that the watershed and tide model have been calibrated and established, Mike FLOOD could provide more robust predictions of inundation.

Most watersheds that drain into the ocean are large, ca. greater than 50 square kilometers. In many cases we have to rely on a single weather station to distribute the precipitation throughout the watershed. This can result in missing a precipitation event, for example if the weather station is located near the outlet of the watershed, an intense rain event could occur in the upper watershed that is not captures, although the stream stage and discharge will show the signal. As a result of this we have examined the use of estimates of precipitation on an aerial basis rather than a point weather station basis. We could further develop and validate information available from the Environment Canada Radar Precipitation maps. An alternative of this would be to distribute more weather stations throughout the watershed to ensure significant rain events are captured.

In this study there was uncertainty regarding the exact elevations of the aboiteau floor and water levels on either side of it. We could install pressure sensors to record stage on either side to better understand and model the function of the aboiteau. Also we could survey the floor of the aboiteau at low water.

6 References

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Weather Underground, Inc. http://www.wunderground.com/history/airport/CXNP

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District of Yarmouth, Chignecto Isthmus and Minas Basin**.** Atlantic Climate Adaptations Solutions Association unpublished report.

7 Acknowledgements

We would like to thank Dave Browning of NS Department of Agriculture for the initial discussions related to this project. Will Green of NS Environment for funding the project. Gary Gilbert NS Department of Agriculture for showing us around in the field and providing us background information. Jim Hannan, Cumberland County Emergency Management Office. Claude Robichaud and Rod Stultz NB Department of Agriculture and Aquaculture provided tide gauge data for Tantramar Dam and Shepody Dam, Brian Sanderson private consultant for supplying the actual data and Jeff Ollerhead at Mount Allison University provided the Fort Beausejour water levels. Chris Coolen at the Canadian Hydrographic Serivice provided the predicted tides at Joggins, Pecks Point, and Grindstone Island.

8 Appendix

8.1 Evapotranspiration Calculation

Described below is the procedure for calculating evapotranspiration (ET_o) using FAO Penman-Monteith with only minimum and maximum temperature

Required data:
Elevation, metres [m]
Latitude, degrees [°]
Minimum Temperature, degree Celsius [°C]
Maximum Temperature, degree Celsius [°C]
Classification as Coastal or Interior
Classification as Arid or Humid
Julian day

Assumed data or constants:

Wind speed	2 m/s
Albedo or canopy reflection coefficient, α	0.23
Solar constant, G_{sc}	0.082 $MJ^{-2}min^{-1}$
Interior and Coastal coefficients, K_{Rs}	0.16 for interior locations
0.19 for coastal locations	
Humid and arid region coefficients, K_o	0 °C for humid / sub-humid climates
2 °C for arid / semi-arid climates	

Procedure:
Calculate mean air temperature, T [°C]

$$T = \left(\frac{T_{min} + T_{max}}{2} \right)$$

Calculate actual vapour pressure, e_a [kPa]
Use minimum temperature and adjustment factor depending on climate classification humid or semi-arid.

$$e_a = 0.6108 \ \exp\left[\frac{17.27\,(T_{min} - K_o)}{(T_{min} - K_o) + 237.3} \right]$$

where:
K_o = 0 °C for humid and sub-humid climates
K_o = 2 °C for arid and semi-arid climates

Stations are classified as coastal and interior, interior stations are considered semi-arid, while coastal stations are considered to be humid.

Calculate saturated vapour pressure for T_{max}, $e_{(T_{max})}$ [kPa]

$$e_{(T_{max})} = 0.6108 \exp\left[\frac{17.27\ T_{max}}{T_{max} + 237.3}\right]$$

Calculate saturated vapour pressure for T_{min}, $e_{(T_{min})}$ [kPa]

$$e_{(T_{min})} = 0.6108 \exp\left[\frac{17.27\ T_{min}}{T_{min} + 237.3}\right]$$

Calculate saturated vapour pressure, e_s [kPa]

$$e_s = \left(\frac{e_{(T_{min})} + e_{(T_{max})}}{2}\right)$$

where:
$e_{(T_{max})}$ = Step 3
$e_{(T_{min})}$ = Step 4
Calculate inverse relative distance Earth-Sun, d_r [rad]

$$d_r = 1 + 0.033 \cos\left(\frac{2\pi}{365} J\right)$$

where:
J = Julian day
Convert latitude to radians, φ [rad]

$$\varphi(rad) = \frac{\pi}{180} lat(°)$$

where:
lat = latitude of station in degrees
Calculate solar declination, δ [rad]

$$\delta = 0.409 \sin\left(\frac{2\pi}{365} J - 1.39\right)$$

where:
J = Julian day
Calculate sunset hour angle, ω_s [rad]

$$\omega_s = \arccos\left[-\tan(\varphi)\tan(\delta)\right]$$

where:
δ = Step 7
φ = Step 8
Calculate extraterrestrial radiation, R_a [$MJm^{-2}\ day^{-1}$]

$$R_a = \frac{24 \times 60}{\pi} G_{sc}\ d_r \left[\omega_s \sin(\varphi)\sin(\delta) + \cos(\varphi)\cos(\delta)\sin(\omega_s)\right]$$

where:
G_{sc} = solar constant = 0.082 $MJm^{-2}\ min^{-1}$
d_r = Step 6
δ = Step 7
φ = Step 8
ω_s = Step 9
Calculate clear sky solar radiation, R_{so} [$MJm^{-2}\ day^{-1}$]

$$R_{so} = \left(0.75 + 2 \times 10^{-5} z\right) R_a$$

where:
Z = elevation of climate station above sea level [m]
R_a = Step 10
Calculate solar radiation, R_s [MJm^{-2} day^{-1}]
Use adjustment factor K_{Rs} depending on station location, coastal or interior

$$R_s = K_{Rs} \sqrt{(T_{max} - T_{min})}\ R_a$$

where:
K_{Rs} = 0.16 for interior locations
K_{Rs} = 0.19 for coastal locations
Calculate net longwave radiation, R_{nl} [MJm^{-2} day^{-1}]

$$R_{nl} = \sigma \ \frac{(T_{max} + 237.15)^4 + (T_{min} + 237.16)^4}{2} \left(0.34 - 0.14\sqrt{e_a}\right) \left(1.35 \frac{R_s}{R_{so}} - 0.35\right)$$

where:
e_a = Step 2
R_s = Step 12
R_{so}= Step 11
σ = 4.903 X 10^{-9} $MJK^{-4}m^{-2}day^{-1}$

Calculate net solar radiation, R_{ns} [MJm^{-2} day^{-1}]

$$R_{ns} = (1 - \alpha)\ R_s$$

where:
R_s = Step 12
α = 0.23
Calculate net radiation, R_n [MJm^{-2} day^{-1}]

$$R_n = R_{ns} - R_{nl}$$

where:
R_{ns} = Step 14
R_{nl} = Step 13
Calculate slope vapour pressure, Δ [kPa °C^{-1}]

$$\Delta = \frac{2504\ \exp\left(\frac{17.27\,T}{T + 237.3}\right)}{\left(T + 237.3\right)^2}$$

Calculate atmospheric pressure, P [kPa]

$$P = 101.3\ \left(\frac{293 - 0.0065\,z}{293}\right)^{5.26}$$

where:
z = elevation above sea level [m]
Calculate psychometric constant, γ [kPa °C^{-1}]

$$\gamma = 0.665 \times 10^{-3}\, P$$

where:
P = Step 17
Calculate evapotranspiration, ET_o

$$ET_o = \left[\frac{0.408\,\Delta\,R_n + \gamma\,\left(\frac{900}{T+273}\right)\,u_2\,(e_s - e_a)}{\Delta + \gamma\,\,(1 + 0.34\,u_2)} \right]$$

ET_o reference evapotranspiration [mm day^{-1}],
R_n net radiation at the crop surface [MJ m^{-2} day^{-1}],
G soil heat flux density [MJ m^{-2} day^{-1}],
T mean daily air temperature at 2 m height [°C],
u_2 wind speed at 2 m height [m s^{-1}],
e_s saturation vapour pressure [kPa],
e_a actual vapour pressure [kPa],
e_s - e_a saturation vapour pressure deficit [kPa],
Δ slope vapour pressure curve [kPa $°C^{-1}$],
γ psychrometric constant [kPa $°C^{-1}$].

8.2 River Model Parameters

Rainfall-Runoff Parameters
Model: NAM
Catchment size: 145.45 sq. km.

Maximum water content storages
Surface storage (Umax): 5 mm
Root zone storage (Lmax): 100 mm

Overland flow runoff coefficient (CQOF): 0.23
Time constant for routing overland flow (CKIF): 1000

Root zone threshold for groundwater recharge (TG): 0
Time constant for routing baseflow (CKBF): 2000
Initial Base flow (BF): 0.12

Hydrodynamic Parameters
Initial water depth: 0.01 m
Bed resistance: Uniform 30 Manning (M)
Wave Approximation: Fully Dynamic

Default Values:
Computation Scheme:

Delta:	0.5
Delhs:	0.01
Delh:	0.1
Alpha:	1
Theta:	1
Eps:	0.0001
Dh Node:	0.01
Zeta Min:	0.1
Struc Fac:	0
Inter1Max:	10
Nolter:	1
MaxIterSteady:	100
FroudeMax:	-1
FroudeExp:	-1

Flood Plain Resistance:
Global Value: -99

Encroachment:
Iteration: Max no. of iterations 20

Stratification:
No. of layers: 10

Turbulence model:

Viscosity:	0.003
Turbulence model in fluid:	k-eps model
Turbulence model at bed:	drag coefficient
Richards numbers correction:	true

Corrections:

Baroclinic pressure

Factor:	1
Local bed slope:	0

Convection / Advection

Factor horizontal momentum:	1
Factor vertical momentum:	1
Factor Advection:	1

Dispersion:

Factor horizontal viscosity:	1
Factor vertical viscosity:	1

8.3 Tidal Model Parameters

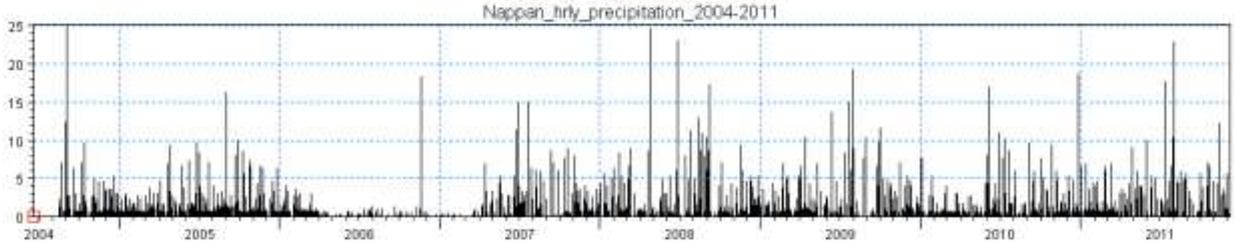
Basic Parameters:

Module Selection:	Hydrodynamic model only
Bathymetry:	
Type:	Cold Start
Apply Coriolis forcing:	True
Number of areas:	1
Simpulation time-step:	15 sec
Flood and Dry:	
Drying Depth:	0.1 m
Flooding Depth:	0.5 m

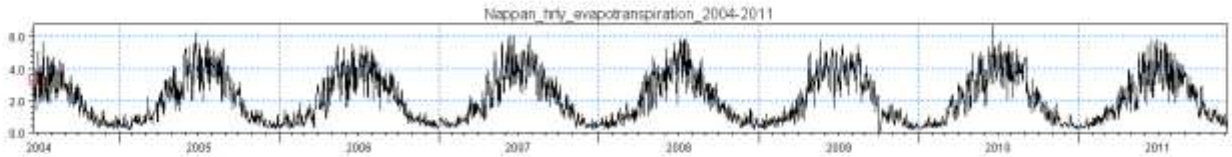
Hydrodynamic Parameters:

Initial surface elevation:	equal to WebTide prediction
Boundary:	
FAB Type:	12
No tilting:	True
User defined flow direction:	False
Eddy Viscosity:	
Velocity Based:	9
Resistance:	
Constant:	25 (manning)

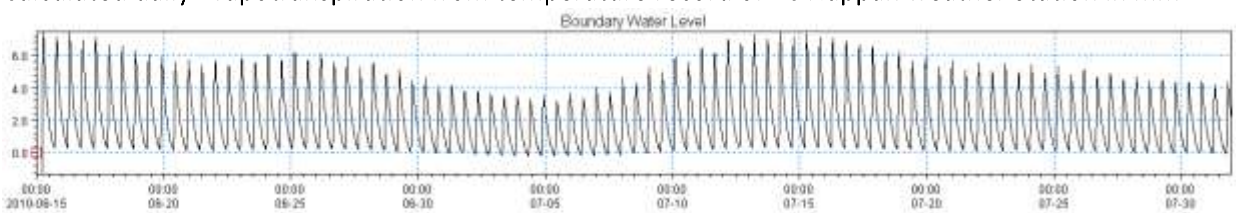
8.4 Environmental Data Time Series



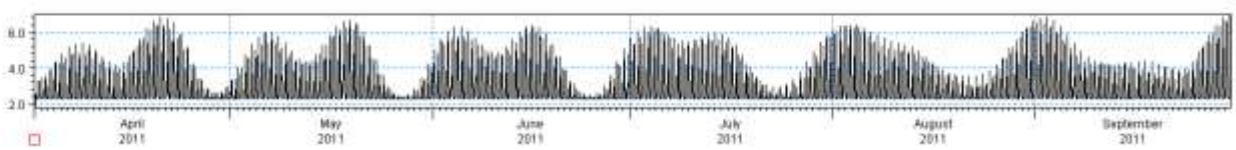
2004-20011 hourly precipitation record of EC Nappan weather Station in mm



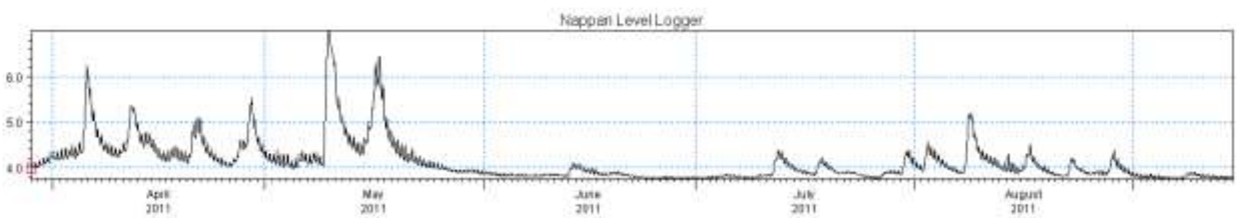
Calculated daily Evapotranspiration from temperature record of EC Nappan weather Station in mm



Simulated June to July 2010 tidal boundary for the near shore of the Nappan Aboiteau at 10 minute intervals in m (CGV28) – Used in flood scenario simulations



Simulated April to September 2011 tidal boundary for the near shore of the Nappan Aboiteau at 10 minute intervals in m (CGV28) – Used in calibration simulations



Level logger recorded from April to September 2011 in 15 minute in m (CGV28) – Used in calibration simulations