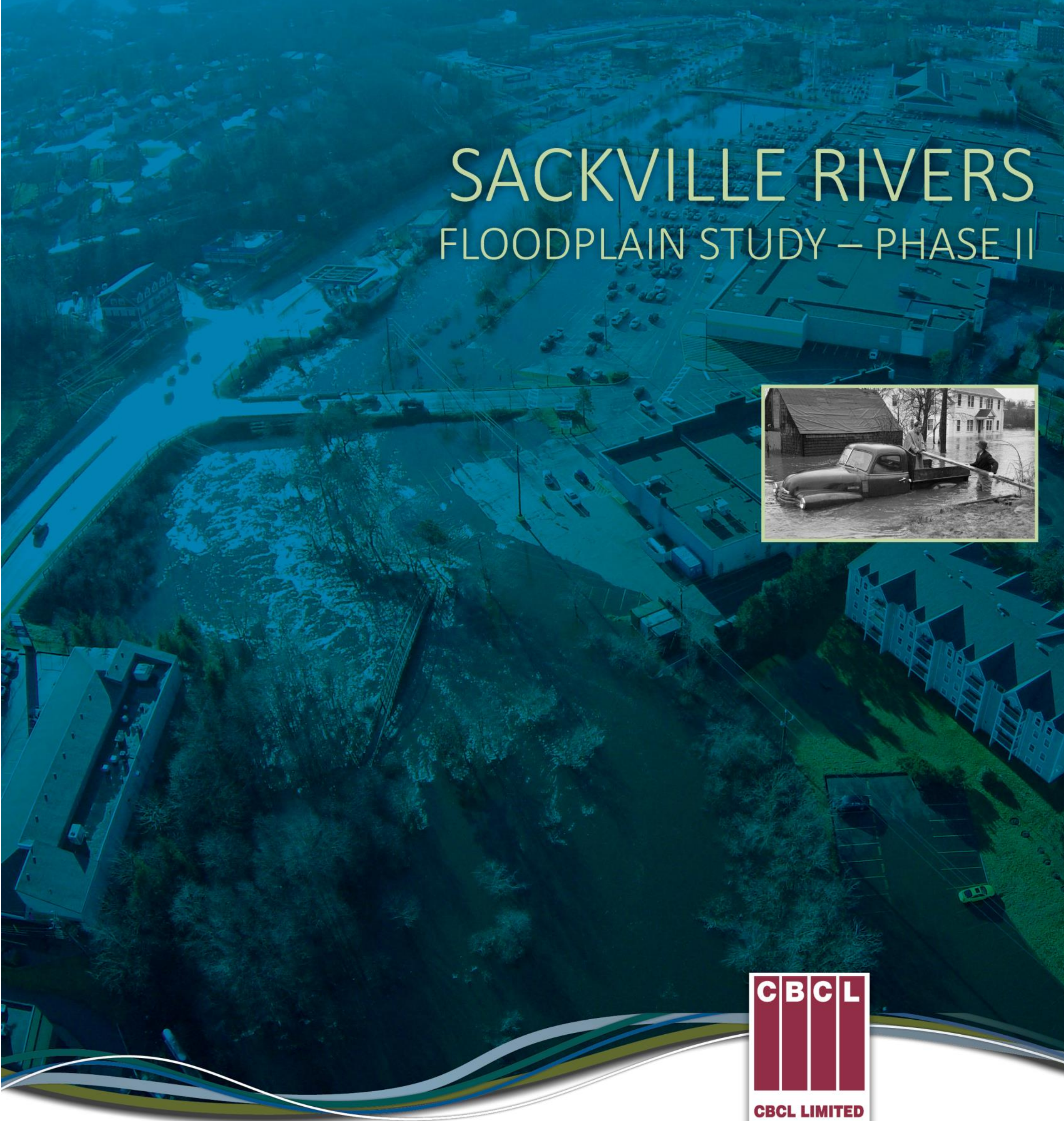
SACKVILLE RIVERS
FLOODPLAIN STUDY – PHASE II



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Prepared for:

HALIFAX

161016.00 • March 2017

Final Report	Alexander Wilson	04/12/17	Jeff Marvin
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REGISTERED PROFESSIONAL ENGINEER

DATE 12 Apr 2017

A. WILSON

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CBCL LIMITED
Consulting Engineers

1489 Hollis Street
PO Box 606
Halifax, Nova Scotia
Canada B3J 2R7

Telephone: 902 421 7241
Fax: 902 423 3938
E-mail: info@cbcl.ca
www.cbcl.ca

Solving today's problems with tomorrow in mind

Cameron Deacoff
Environmental Performance Officer
Halifax Regional Municipality
40 Alderney Drive
2nd Floor
Dartmouth NS, B2Y 2N5

Dear Mr. Deacoff:

RE: Sackville Rivers Flood plain Study – Phase II Final Report

We are extremely pleased to submit this final report for the Sackville Rivers Floodplain Study – Phase II. We believe we have conducted a thorough hydrologic and hydraulic analysis, complemented with an analysis of climate change impacts, that supports the discussion of the causes of flooding and the resulting flood maps.

It is hoped that this report provides the Halifax Regional Municipality with the suitable tools to undertake the next steps in the planning process and increasing public safety in the floodplain area.

If you have any questions or comments, please do not hesitate to contact us.

Yours very truly,

CBCL Limited

Alexander Wilson, M.Eng., P.Eng.
Senior Water Resources Engineer
Phone: (902) 421-7241 Ext. 2502
E-Mail: alexanderw@cbcl.ca

Project No: 161016.00

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Executive Summary

A need to update the previous flood line delineation analyses was identified by the HRM. This need arose from the emergence of updated information and tools of much better quality (topography, flow and water level, rainfall, hydrologic and hydraulic computer models), as well as research on climate change, and pressure from the business community.

This study has assessed the hydrology and hydraulic regime of the Sackville River and the Little Sackville River, as well as their respective watersheds, in order to produce floodplain maps for various flood scenarios. Flood risks were evaluated based on a calibrated hydrologic and hydraulic model using PCSWMM, and an ice jam hydraulic model using HEC-RAS. Model calibration and validation for the PCSWMM model was carried out for flood events corresponding to each of the four seasons, and for each of the two rivers. Design flood scenarios included variations in seasonal conditions, rainfall conditions under climate change, sea level conditions under climate change, development conditions and ice conditions for various rainfall events and sea level events. The resulting flood lines delineated for this study include seasonal changes, historical design storm, existing climate, existing and future development, various scenarios of climate change for existing and future development, ice jam analysis and previous flood line comparison. Mapping of the Phase I river flow frequency analysis results is presented as well.

The thorough analysis presented in this report was carried out to support the flood extents produced by the hydrologic and hydraulic models. The flood extents may be incorporated into future planning documents, which warrants this thorough analysis. Included in this assessment was also an in-depth analysis of climate change impacts on rainfall and sea levels. Since climate change is to be considered in planning documents, it was essential to use the best science and tools available to evaluate those effects. Other significant inputs to this assessment included a radar-rainfall analysis to improve the model calibration, an ice jam analysis and model calibration and validation for each season in the year for both rivers.

The Request for Proposal (RFP) required a recommendation for the selection of a Base Flood. This was defined by HRM as a pair of flood lines, for the floodway (1 in 20 year) and floodway fringe (1 in 100 year), for planning and regulatory purposes. Since the scope of this study does not include any stakeholder consultation, assessment of vulnerability of floodplain infrastructure, land uses and services, nor any review of existing and future planning challenges and opportunities, the current recommendation is strictly related to river hydrodynamics and the current state of climate change science.

In this respect, CBCL agrees with following HRM's proposition, which is to select the most conservative model result to ensure that known risks to public safety are not being ignored.

This means that the future 1 in 20 year and 1 in 100 year flood lines in worst case climate conditions is recommended, which, in this instance, includes the following characteristics:

- Fall seasonal watershed characteristics for the Little Sackville River;
- Winter seasonal watershed characteristics for the Sackville River;
- 24-hour duration design storm event for the Little Sackville River;
- 48-hour duration design storm event for the Sackville River;
- Future development conditions for both watersheds (as known at the time of this study by HRM);

1489 Hollis Street
PO Box 606
Halifax, Nova Scotia
Canada B3J 2R7

Telephone: 902 421 7241
Fax: 902 423 3938
E-mail: info@cbcl.ca
www.cbcl.ca

CBCL LIMITED
Consulting Engineers

1489 Hollis Street
PO Box 606
Halifax, Nova Scotia
Canada B3J 2R7

Telephone: 902 421 7241
Fax: 902 423 3938
E-mail: info@cbcl.ca
www.cbcl.ca

- Climate change conditions for the Western University IDF-CC Tool upper bound result for the 2070-2099 period; and,
- 1 in 20 year and 1 in 100 Year return periods.

The first result of interest is the comparison with the previously generated flood lines. A comparison is made between the 1 in 100 year flood lines from the 1980's regulated floodplain width, the Porter Dillon study from 1999 and the current modelling results. It is clear that the use of the lidar data and computer mapping techniques improved the resolution and consistency of the model results (previous results were drawn by hand). Beyond this, hydrotechnical modelling also shows the flow regime in steep sections allowing the river width to narrow (for example, around the Downsview Mall), which was not identified in previous assessments. The other prominent difference is in the downstream areas of the Highway 101 and the Bedford Place Mall. The updated model results show significantly larger flood extents, where both locations are under extensive flooding during the 1 in 100 year event. Those changes are estimated to result more from the improved quality of calibration, hydrodynamic modelling and surface topographical data, rather than the increased extent of the flow monitoring record.

Other findings from this analysis include the identification of factors that lead to the flooding extents generated by the models. The analysis of structure constrictions only identified four structures that create notable impediments to the passage of water. Those structures are the Beaver Bank Cross Road, Beaver Bank Road and Sackville Drive structures along the Little Sackville River and the Lucasville Road structure along the Sackville River. Other than those structures, there are few anthropogenic impacts to the natural shape of the river channel, other than river diversions to circumvent development. This is a notable finding, because it demonstrates that flooding outside of the river channel (i.e. in the floodplain) is a natural phenomenon. Natural rivers create over time a natural channel whose size is reflective of average river flows. Flows above average values carve a natural floodplain in the landscape. The majority of floodplain extents in Nova Scotia rivers were created during the melting of the last ice age glaciers, approximately 10,000 years ago. These are natural floodplains, which rivers occupy in higher than average flows. The model results show that the current 1 in 100 year peak flood extents occupy a large portion of this natural "ice melt" floodplain. Notably, the model results also indicate that events of a greater magnitude, including the 1 in 500 year event, the Probable Maximum Precipitation (PMP) or future events influenced by development and climate change lead to increased floodplain width (as expected), but only by a small relative amount. This means that high flows will regularly fill the floodplain, but that extremely high flows will still stay within this main floodplain. It is important to note this because it means that the floodplain is necessary for the conveyance of high flows. Development within the floodplain will unavoidably be at risk of flooding, and any restriction of this floodplain will lead to higher upstream water levels. Notable development in the floodplain includes the road crossings noted above, the Downsview Mall, the development around Sackville Cross Road, the Contessa Ct. and Sami Dr. residential developments, the Bedford Place Mall and adjacent residential development. The most notable infrastructure that alters the floodplain is the Highway 101 crossing and its interchange with Highway 102. All the above areas are at risk of flooding because they lie within the natural floodplain. Their impacts on flood levels seem to be limited, but this has not been confirmed by modelling a scenario where this development does not exist.

The assessment of seasonal effects on flood risks also yielded interesting results. The Little Sackville River, being more urbanized, did not show notable seasonal variations in flood elevations. However, the Sackville River showed high sensitivity to seasonal changes, with

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close to a metre of difference in water levels, downstream of its confluence with the Little Sackville River. Development projections showed little influence, with an increase in the order of 100 mm in the downstream end of the Sackville River. Tidal effects, with and without climate change, were shown by the model to be limited to just upstream of the Bedford Highway.

The hydrologic model was calibrated on historic flow records. The results of the model are therefore consistent with the historical peak flows (e.g. the 1 in 100 year peak flow is calculated in the model to be a seasonal average of 38.5 m^3/s in the Little Sackville River, which compares to 26.3 m^3/s from the flow gauging data and 109.75 m^3/s in the Sackville River, which compares with 115 m^3/s estimated directly from the flow gauging data). Compared to the historical storm of March 2003, the water level results are slightly higher throughout the river system, which is consistent with the finding that the March 2003 event was less significant than a 1 in 100 year event.

Results of modelling rainfall impacted by climate change were also generated. It was found that the large number of existing climate change models, combined with the various methods of transformation of the results into rainfall amounts, produced a wide range of results, with the highest rainfall amount calculated at 283.9 mm, a 70% increase compared to the existing 1 in 100 year rainfall amount (166.7 mm). Interestingly, while the water levels increased accordingly, the floodplain width did not significantly widen. This is mostly a result of the existing floodplain topography in which the floodplain edges have higher slopes, resulting in a small change of width when water levels increase. The 1 in 500 year event results showed larger flooding extents than the 1 in 100 year event, but again, to a limited extent. Since the total rainfall amount in 24 hours is 199 mm for the 1 in 500 year event, it is only marginally higher than the 1 in 100 year total rainfall amount (166.7 mm), and notably lower than the climate change amount (283.9 mm). Results are therefore much closer to the 1 in 100 year event than the worst case climate change scenario.

A discussion of potential flood mitigation options considers the benefits and challenges associated with each potential measure. Although this assessment did not investigate in detail, nor model, any flood mitigation option, certain high level aspects can be drawn from the results. The flood line delineation showed that climate change impacts clearly have the potential to increase flooding risks and should be considered in any future planning decision. The planning regulations will be central to managing future development and it is recommended that they include language on setback limits, runoff control, flood proofing or limited uses in floodplain areas. Designating environmentally sensitive areas (e.g. Watercourse Greenbelt zoning in East Hants) is also recommended to prevent future development in water storage and undeveloped floodplain areas.

While the upper reaches of the Sackville River are mainly undeveloped, its lower reaches, and most of the Little Sackville River, are quite highly urbanized, which is both increasing river flows as well as creating vulnerabilities. The following list of factors have contributed to the prioritized recommendations noted below.

- Risks associated with climate change;
- Increased interest in sustainability;
- Increased awareness of liability;
- Increasing costs of maintenance, and
- General reduction in funding for infrastructure projects

Recommendations have been generally oriented towards more sustainable, low maintenance, more nature-oriented approaches, which provide not only solutions to

1489 Hollis Street
PO Box 606
Halifax, Nova Scotia
Canada B3J 2R7

Telephone: 902 421 7241
Fax: 902 423 3938
E-mail: info@cbcl.ca
www.cbcl.ca

CBCL LIMITED
Consulting Engineers

1489 Hollis Street
PO Box 606
Halifax, Nova Scotia
Canada B3J 2R7

Telephone: 902 421 7241
Fax: 902 423 3938
E-mail: info@cbcl.ca
www.cbcl.ca

flooding risks, but also additional advantages in terms of erosion protection, water quality improvements and overall aesthetics and protection/restoration of the natural character of the rivers. This is consistent with the Sackville Greenway Plan, the Halifax Regional Plan and the Halifax Green Network Plan (Greenbelting and Open Space Plan).

Recommendations for flood mitigation, beyond adopting the floodlines in this report into planning regulations, are the following:

1. **Stormwater Infiltration - Best Management Practices (BMPs) and Low Impact Development (LID):**
   The least intrusive and most cost-effective flood mitigation option is to implement stormwater infiltration measures (LID and BMPs). It is recommended that such measures be enforced for all future development (more effective than detention ponds) through planning regulations and during resurfacing or repair works. BMPs and LID can have a very low direct cost but make a clear impact in flood reduction, in a manner that mimics natural processes;
2. **Increasing channel capacity through river restoration**:
   Other recommended approaches include conducting river restoration to increase capacity and storage in river sections that have been channelized. Significant ecosystem benefits are also achieved;
3. **Purchasing properties at risk**:
   The impacted individuals are now permanently safe, properties at risk can be restored to the natural floodplain, upstream flooding risks can be reduced, there is no further maintenance cost or residual risk, and the riverfront area can now be enhanced for public enjoyment. The challenges are its cost and resistance from property owners. Where not yet developed, purchasing floodplain lands can ensure their protection in the future;
4. **Flood Protection Infrastructure**:
   Options such as upgrading bridge structures, building berms, or raising the level of the land or homes, should only be used after the above options have been exhausted. They will be expensive, require maintenance, will move the problem downstream and will place public safety at increased risk for events greater than the design event.

In all cases, stakeholder consultations and modelling should be carried out to identify the best compromise between protecting vulnerabilities, overall stakeholder needs, ecosystem protection and costs. The creation of a dedicated floodplain committee (possibly cross-municipal to include the Municipality of East Hants) with regular meetings can streamline this process.

Overall, this study has updated the current state of knowledge on rainfall, hydrologic (including seasonal) characteristics, river flow responses, impacts of structures and ice jams, mechanisms leading to flooding, potential climate change impacts and potential flood mitigation options. This study has brought very detailed data sets of high resolution and quality, combined with state-of-the-art modelling and analysis to inform the results and recommendations presented.

Recommendations to improve this analysis in the future would include conducting further flow gauging in various areas of the watershed, evaluating in more detail ground infiltration and exfiltration characteristics, being cognizant of the latest climate change research as it progresses, and trying to collect as much calibration data (water levels) as possible in the rivers during flood events.

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In terms of recommended next steps for the HRM, the first goal of this study is to provide information to support an update to the planning regulations. An essential step, as noted by the HRM, is to make every effort to communicate the results and implications of this study and planning regulation to the public and all affected stakeholders, which is best achieved by using a wide range of approaches. Communication of flooding risks and emergency procedures, as well as flood proofing techniques, is also very valuable to help residents understand and deal with flooding risks. Warning systems, including flood forecasting and warning, can be very valuable tools to increase public safety. In terms of flood mitigation options, next steps will need to include conducting more detailed analyses and modelling of potential options. This can be done in parallel with an assessment of vulnerabilities along the river system, conducted through consultation with each of the relevant stakeholders. Vulnerabilities for land use, infrastructure and services can be obtained from stakeholders. Together with vulnerabilities in the management of emergency procedures (e.g. ensuring reliable communications or access to emergency services), these can be ranked by priority to define flood protection goals. How well each flood mitigation measures addresses each vulnerability can then be used to evaluate the efficiency of each flood protection measure.

1489 Hollis Street

PO Box 606

Halifax, Nova Scotia

Canada B3J 2R7

Telephone: 902 421 7241

Fax: 902 423 3938

E-mail: info@cbcl.ca

www.cbcl.ca

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CHAPTER 1 INTRODUCTION

1.1 Background

The first major floodplain studies completed for the Sackville River and the Little Sackville River were carried out in the 1980's under the Canada-Nova Scotia Flood Damage Reduction Program (FDRP), which included the "Hydrotechnical Study of the Sackville River" by Interprovincial Engineering Limited (1981) and the "Hydrotechnical Study of the Little Sackville River Floodplain" by Nolan Davis & Associates Limited (1987). Both of these studies produced 1 in 20 year and 1 in 100 year flood lines in the respective rivers based on HEC-2 hydraulic models, which were then adopted as land use regulation boundaries in 1994 to regulate development in the floodplain. An update of the Little Sackville River flood study was then carried out for the Halifax Regional Municipality (HRM) by Porter Dillon Limited in 1999 to update the flood lines with updated land use and future development information, as well as to identify potential remedial measures for flood damage reduction. Although these updated floodplain extents were not adopted by Halifax Regional Council, they were used as reference material by HRM staff in consideration of development applications.

Since the time of the previous flood studies, land use in the watershed has changed and future development potential has been updated. In particular, the Little Sackville River has undergone significant urbanization in the past two decades, including several commercial and single unit residential developments in the Bedford area, as well as the Sackville-Bedford Greenway Connector. Furthermore, the impacts of future climate change and sea level rise have become better understood, hydrologic and hydraulic computer modelling tools have become more sophisticated, measured flow and water level data has increased and topographic information has become significantly more detailed. Responding to requests by the Sackville Business Association, HRM decided to complete an updated floodplain study of the Sackville River and the Little Sackville River to update the previous flood line delineations based on the new information. The first phase of the floodplain study was carried out by GHD in 2016, and was intended to collect and analyse background information for the second phase of the study, including survey data and information on historical floods.

1.2 Purpose of Study

This report presents Phase II of the Sackville Rivers Floodplain Study for the Sackville River and the Little Sackville River, which includes a comprehensive hydrologic and hydraulic modelling analysis and floodplain delineations along the two rivers for various flood scenarios. This updated floodplain study therefore presents detailed flood maps along the river that are associated with existing and future conditions for land use, development, climate change (in precipitation and sea level rise) and ice jamming. The resulting flood lines delineated as part of this study provide HRM with updated information on flood risks within the Sackville River and Little Sackville River watersheds that will support the planning of future development and flood mitigation efforts.

1.3 Study Approach

This study was carried out in the following three phases:

1. Collection and analysis of available data and field data (Chapter 2);
2. Development and calibration of a hydrologic and hydraulic model (Chapter 3);
3. Flood scenario modelling and delineation of flood lines (Chapter 4).

A more detailed list of the tasks carried out for this study is as follows:

- Collection of existing data including lidar topographic mapping, GIS data, lake bathymetry, rainfall data, flow data, water level data, radar data, tide data, topographic survey data, hydraulic structure information, previous floodplain study reports, historical flood information and watershed operational practices;
- Collection of field data on water levels in McCabe Lake and hydraulic structure measurements;
- Delineation of sub-watersheds, land cover mapping, soil mapping, compilation of the flow, water level rain gauging data and development of a database of the hydraulic structures;
- Development of a hydrologic and hydraulic model using PCSWMM;
- Calibration and validation of the PCSWMM model to various historical rainfall events for each of the four seasons, including an analysis and processing of radar rainfall data;
- Selection of design model parameters for PCSWMM to carry out flood scenario modelling;
- Development of a hydraulic model using HEC-RAS for the purposes of ice jam analysis and encroachment analysis;
- Development of design rainfall events, design sea level events and design ice accumulation events based on existing, future climate change and future sea level rise conditions;
- Flood scenario modelling for various design scenarios using the PCSWMM and HEC-RAS models;
- Delineation of flood lines for the various design flood scenarios;
- Encroachment analysis to determine areas that are at higher risk of impact from infilling and development;
- Overview of potential flood mitigation efforts, and
- Reporting.

CHAPTER 2 DATA COLLECTION AND ANALYSIS

2.1 Data Collection

This section presents an overview of the existing data that was obtained and the field data that was collected for this study.

2.1.1 Review of Sackville Rivers Floodplain Study Phase I Report

The Sackville Rivers Floodplain Study Phase I report (GHD 2016) was reviewed to identify any information that could potentially support the Phase II analysis and reduce data collection needs.

The Phase I report conducted a thorough peak event flow analysis of the hydrometric data for the two rivers. Since the Phase II analysis was based on flows developed using a hydrologic analysis with a calibrated dynamic model, the Phase II analysis allowed the estimation of flows throughout the system that took into account the effect of lakes, storage, river constrictions, structure restrictions, as well as tidal influence. The results of the Phase I analysis, which only included information at one single point for each river, were therefore not needed for the Phase II assessment.

The Phase I study conducted an analysis of sea levels in the Halifax Harbour. In this analysis, the tidal effects were removed from the data. Since the Phase II study needed to use the maximum water levels in the Halifax Harbour, this analysis was not needed. The Phase II analysis instead used the actual tide gauge measurements in the Halifax Harbour, complete with tidal effects, storm surges and seiching effects.

The topographic survey data collected in Phase I was helpful where the lidar data did not include underwater points, and was included in the Phase II hydraulic model.

The hydraulic model developed in Phase I provided little information, aside from hydraulic structure geometry, that supported the Phase II study.

A historical review of flooding factors was conducted in Phase I, and did not offer sufficient detail to be used in the Phase II study. However, the list of main precipitation events was helpful as a starting point, but since the largest precipitation events were found to differ from the largest flooding events, the value of the list was limited.

2.1.2 Existing Data Collection

The following existing data summarised in **Table 2.1** was obtained and reviewed for the study.

Table 2.1: Summary of Existing Data Collection

Data Description	Details	Source
Lidar Data	• HRM Digital Elevation Model (DEM); • HRM Digital Surface Model (DSM); and • HRM Hydrologically Corrected DEM.	HRM
GIS Shape Files	• Contours (5 m resolution); • Watercourses; • Waterbodies; • Watersheds; and • Railways.	Government of Nova Scotia
	• Halifax County (ns13b) Soils.	Agriculture and Agri-Foods Canada
	• Forestry Inventory.	Nova Scotia Department of Natural Resources
	• Future Development Areas; • Buildings; and • Streets.	HRM
Lake Bathymetry Drawings	• McCabe Lake Bathymetry; • Webber Lake Bathymetry; and • Feely Lake Bathymetry.	Nova Scotia Fisheries and Aquaculture
Intensity-Duration-Frequency (IDF) Curves	• Shearwater Airport IDF Curves; and • Halifax Airport IDF Curves.	Environment Canada
Historical Precipitation & Rainfall Data	• Shearwater Airport Precipitation Data; • Halifax Airport Precipitation Data; and • Bedford Rifle Range Precipitation Data.	Environment Canada
	• Beaver Bank (INOVASCO65) Rainfall Data; and • Lewis Lake (INOVASCO52) Rainfall Data.	Private Rain Gauges
	• Sandy Lake Rainfall Data.	Dalhousie University
Historical Radar Data	• 10-Minute Radar Rainfall Data.	Environment Canada
Historical River Flow & Water Level Data	• Sackville River at Bedford Hydrometric Station (01EJ001) Flow and Water Level Data; and • Little Sackville River at Middle Sackville Hydrometric Station (01EJ004) Flow and Water Level Data.	Environment Canada
	• Golf Course (Site 1) Stream Flow Data; • Mount Uniacke (Site 2) Stream Flow Data; • Little Sackville River at HWY-101 (Site 3) Water Level Data; • Sackville River Upstream of Confluence (Site 4) Water Level Data;	Dalhousie University

Data Description	Details	Source
	• Sandy Lake (Site 5) Stream Flow Data; and • Sackville River at HWY-102 (Site 6) Water Level Data.	
Historical Tide Data	• Halifax Harbour Tide Gauging Data.	Fisheries and Oceans Canada
Topographic Survey Data	• 2015 survey of river cross sections and bridge structures along the Sackville River from Halifax Harbour to approximately 500 m upstream of the Little Sackville River confluence (along both tributaries) – carried out by GHD.	HRM
Hydraulic Structure Drawings	• Record & Design Drawings for Various Bridges, Culverts and Dams.	HRM
Previous Flood Study Reports & Data	• Hydrotechnical Study of the Sackville River (Interprovincial Engineering Limited 1981); • Hydrotechnical Study of the Little Sackville River Floodplain (Nolan Davis & Associates Limited 1987); • Hydrotechnical Study of the Little Sackville River Floodplain (Porter Dillon Limited 1999); and • Sackville Rivers Floodplain Study – Phase I (GHD 2016).	HRM
Historical Flood Information	• Drone images from December 11, 2014; and • Drone footage & images from April 24, 2015.	FliteLab Imaging
	• HRM Service Request Reports.	HRM
	• Photos from various historical flood events.	Sackville Rivers Association
Watershed Operational Practices	• Dam operational control procedures for Feely Lake and Lumber Mill Pond.	Barrett Lumber Company

2.1.3 Field Data Collection

2.1.3.1 McCabe Lake Water Level Measurements

Water level data was collected at the outlet of McCabe Lake from the 7th of June 2016 to the 3rd of August 2016 to be used as calibration data for the hydraulic model. Data was collected using a Solinst LTC Levelogger Junior data logger. The water level gauge uses piezoresistive silicon with Hastelloy sensors to measure pressure, which can then be converted into water level by subtracting the barometric pressure. Hourly barometric pressure from the Environment Canada Bedford Rifle Range climate station was therefore used to convert the water level data. One water level gauge was placed just upstream of the outlet to measure lake levels (44.775103°, 63.740060°), and a second water level gauge was placed just downstream of the outlet to measure water levels in the downstream channel (44.775102°, -63.739838°). The water level measurements are presented as part of the hydraulic model calibration results (see **Appendix C**).

2.1.3.2 Hydraulic Structure Measurements

Measurements and photos were collected for bridges, culverts and dams along the Sackville River and the Little Sackville River for structures where record drawings and survey data was not available. Further information on data collected is provided in **Section 2.2.6** and **Appendix A**.

2.1.3.3 Topographic Survey of Benchmark for Little Sackville River Hydrometric Station

A topographic survey was carried out by CBCL Limited and Environment Canada staff for the "Little Sackville River at Middle Sackville" hydrometric station (01EJ004) to determine the geodetic elevation of the benchmark used to define the datum for the station's water level measurements. While a previous topographic survey was completed for this station, the geodetic elevation previously measured for the benchmark was identified as being erroneous due to poor correlation with the lidar data. Following the survey completed as part of this study, the zero water level reading for the station was found to be at 26.037 m geodetic.

2.2 Data Analysis

The collected data was then analysed and used to develop the following datasets and maps:

- Watershed Delineation;
- Land Cover Mapping;
- Soil Mapping;
- Flow and Water Level Gauging Data Compilation;
- Rain Gauging Data Compilation; and
- Hydraulic Structure Data Sheets.

2.2.1 Watershed Delineation

Watersheds were delineated for the Sackville River and for the Little Sackville River using the hydrologically corrected lidar DEM and are presented in **Figure 2.1**. As shown in the figure, the Sackville River watershed has a drainage area of 150 km^2 and the Little Sackville River watershed has a drainage area of 15 km^2. The highlighted river and lake areas in **Figure 2.1** depict the hydraulic modelling and floodplain extents selected for this study.

Sub-watersheds were then delineated to each bridge or culvert structure along the Sackville River and the Little Sackville River such that flows could be estimated immediately upstream of each structure. The sub-watersheds were then further divided into smaller sub-areas as needed for the hydrologic model such that watershed flows would be accurately distributed throughout the river channels. This included dividing sub-watersheds in locations where there were long distances between hydraulic structures or where tributary watercourses intersected with the main river channel. Additional refinement was also introduced in areas with high potential future growth, in order to be able to represent the potential impact of development on the local watersheds and downstream. Sub-watershed delineations are presented in **Figure 2.2** for the upper reach of the Sackville River, **Figure 2.3** for the lower reach of the Sackville River and **Figure 2.4** for the Little Sackville River. For this report, the upper reach of the Sackville River is defined by the Sackville River upstream of its confluence with the Little Sackville River, and the lower reach of the Sackville River is defined by the Sackville River downstream of its confluence with the Little Sackville River. The term “reach” is a geographic term that describes a particular river segment.

2.2.2 Land Cover Mapping

Land cover areas for existing development conditions were delineated within the watersheds based on aerial photography and the Nova Scotia Department of Natural Resources Forestry Inventory GIS database for the following seven land cover types: Lake, Wetland, Forest, Clear Cut, Low Density Development, Medium Density Development and High Density Development. The level of density (low, medium or high) was assigned following the levels of impervious area[1] found in the study area watersheds. Low development therefore corresponds to rural residential development with large lots, whereas high density development corresponds to an urban business park for example. The resulting land cover mapping of the watersheds for existing development conditions is presented in **Figure 2.5**. The future development areas provided by HRM were then used to develop land cover mapping for future development conditions, which is presented in **Figure 2.6**. The level of density assigned to future development was selected based on the average of the existing development densities in the watersheds.

As shown in the land cover maps, the Sackville River watershed is mostly rural or forested with several lakes, whereas the Little Sackville watershed is highly urbanized. Towards the lower end of the Sackville River after its confluence with the Little Sackville River, the watershed becomes significantly more developed. Finally, while future development is planned within both watersheds, the Sackville River watershed will see significantly more of its undeveloped areas become urbanized.

2.2.3 Soil Mapping

The soil survey data obtained from Agriculture and Agri-Foods Canada for Halifax County (ns13b) was used to develop a soil map of the watersheds by grouping the soil layers into the following six soil layers: Water, Rock, Sandy Loam, Loam, Sandy Clay Loam and Clay Loam. The resulting soil mapping of the watersheds is presented in **Figure 2.7**.

2.2.4 Flow and Water Level Gauging Data Compilation

All historical flow and water level data obtained from Environment Canada and Dalhousie University was compiled into a database. A summary of the available flow and water level gauging data that was compiled for this study is presented in **Figure 2.8**. As shown in the figure, flow and water level was obtained from two Environment Canada hydrometric stations within the study area, “Sackville River at Bedford” (Station 01EJ001) and “Little Sackville River at Middle Sackville” (Station 01EJ004). This data was of very good quality and it was very fortunate that such data existed for both rivers studied. Flow and water level data was also obtained from Dalhousie at 6 sites throughout the watershed. However, only three of the Dalhousie sites (Site #3, Site #4 and Site #6) were within the hydraulic model study area and could therefore be used for model calibration.

2.2.5 Rain Gauging Data Compilation

All historical rain gauging data obtained from Environment Canada, Dalhousie University and private sources was compiled into a database. A summary of the available hourly and sub-hourly rain gauging data that was compiled for this study is presented in **Figure 2.9**. As shown in the figure, hourly rainfall

1: 0% to 30% Impervious corresponds to Low Density – 31% to 70% corresponds to Medium Density – 71% to 100% corresponds to High Density data was obtained from three nearby Environment Canada hydrometric stations (Shearwater Airport, Halifax Airport and Bedford Rifle Range) and sub-hourly data was obtained from three private rain gauges (Sandy Lake Academy, Beaver Bank and Lewis Lake). All other nearby private rain gauging stations that were investigated were found to not have available data for the model calibration events described in **Section 3.4**.

2.2.6 Hydraulic Structure Data Sheets

A total of 30 hydraulic structures were identified within the study area along the Sackville River and the Little Sackville River, and their locations are presented in **Figure 2.10** and **Figure 2.11**. Information obtained from record drawings, previous reports, field measurements, survey data and lidar data on all hydraulic structures within the study area was compiled into a database consisting of data sheets for each structure. The hydraulic structure data sheets are presented in **Appendix A** and a summary of the data is presented in **Table 2.2**.

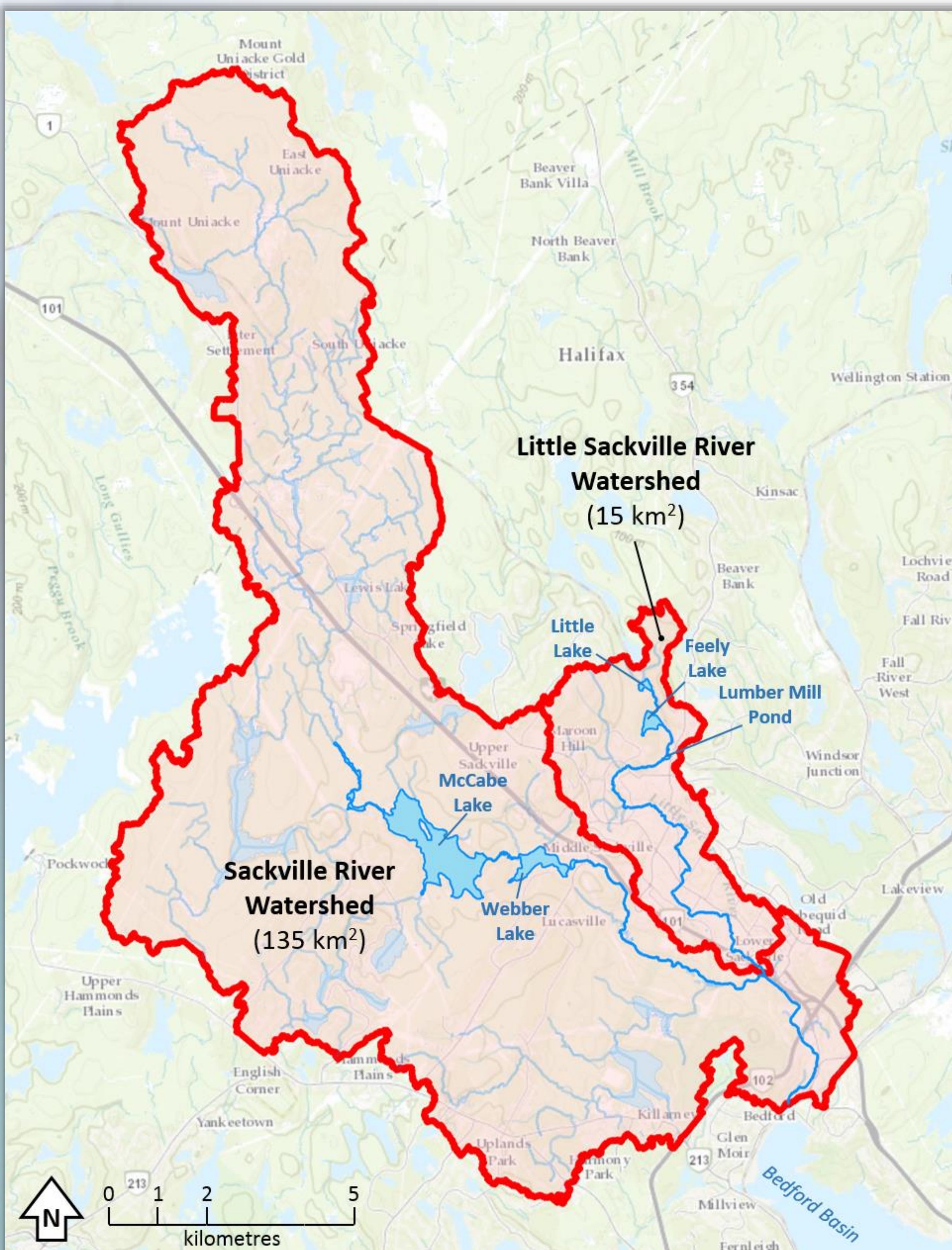


Figure 2.1: Watershed Delineation of Sackville River and Little Sackville River (Background Map Source: Esri World Topographic Map)

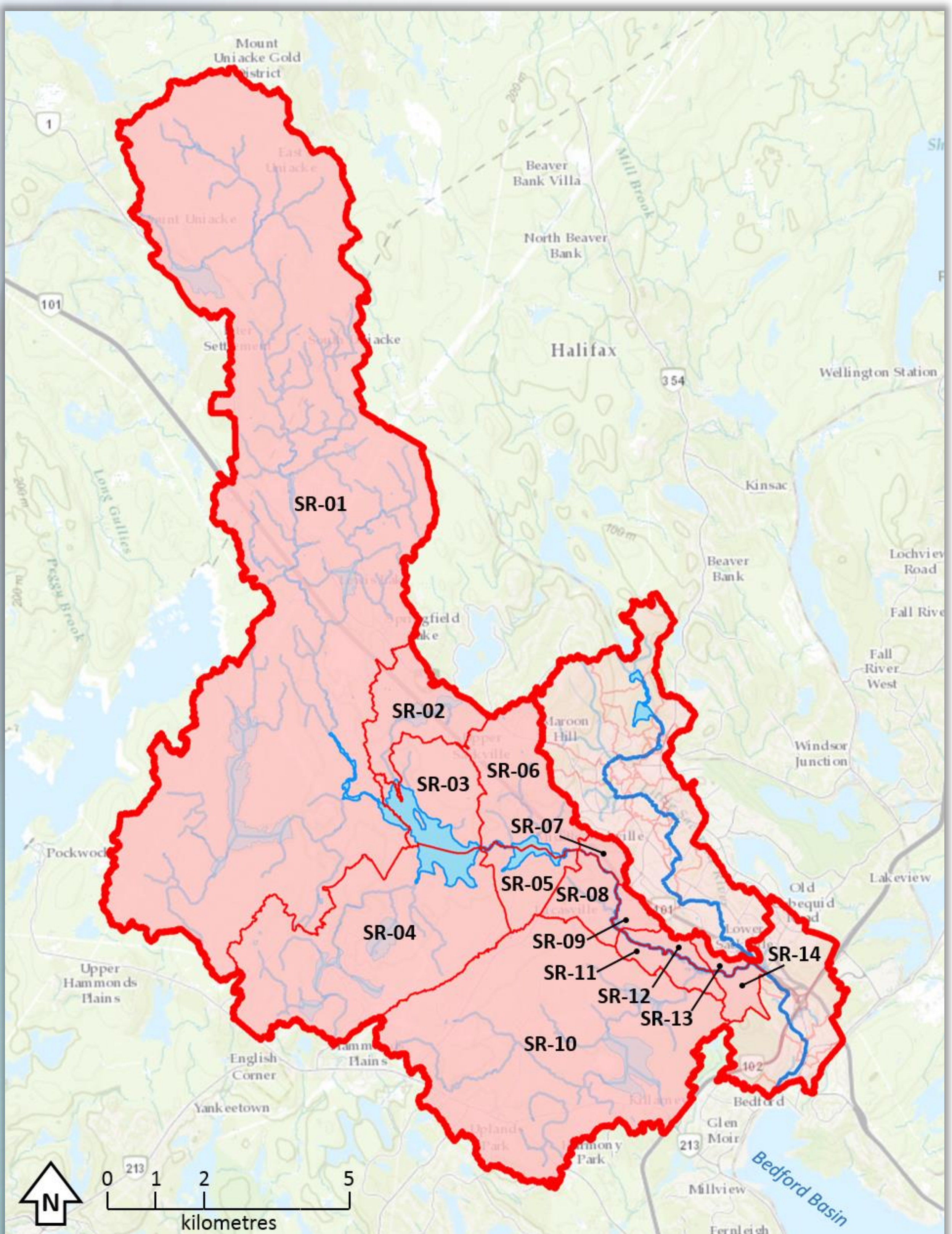


Figure 2.2: Sub-Watershed Delineation of the Upper Reach of the Sackville River **(Background Map Source: Esri World Topographic Map)**

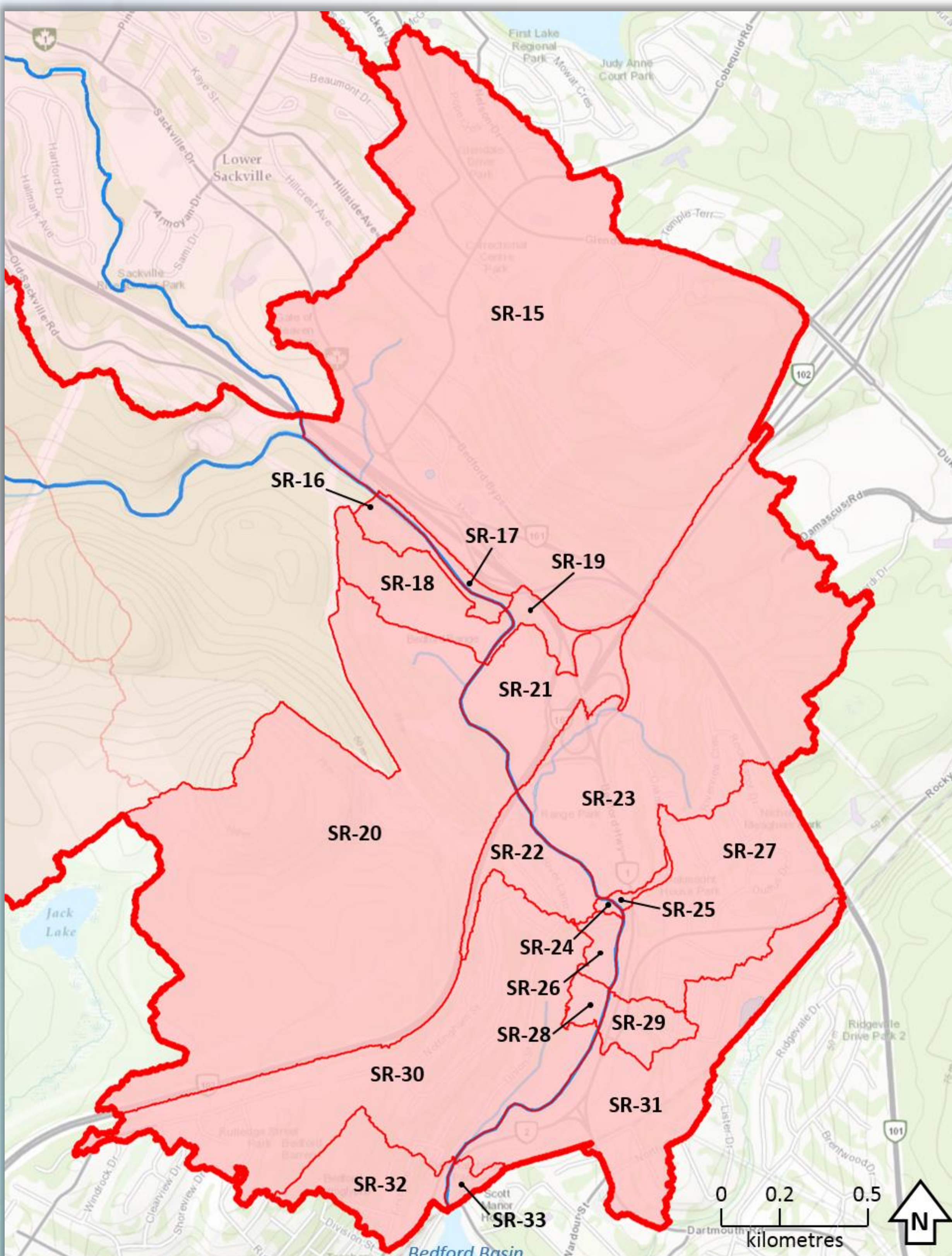


Figure 2.3: **Sub-Watershed Delineation of the Lower Reach of the Sackville River** **(Background Map Source: Esri World Topographic Map)**

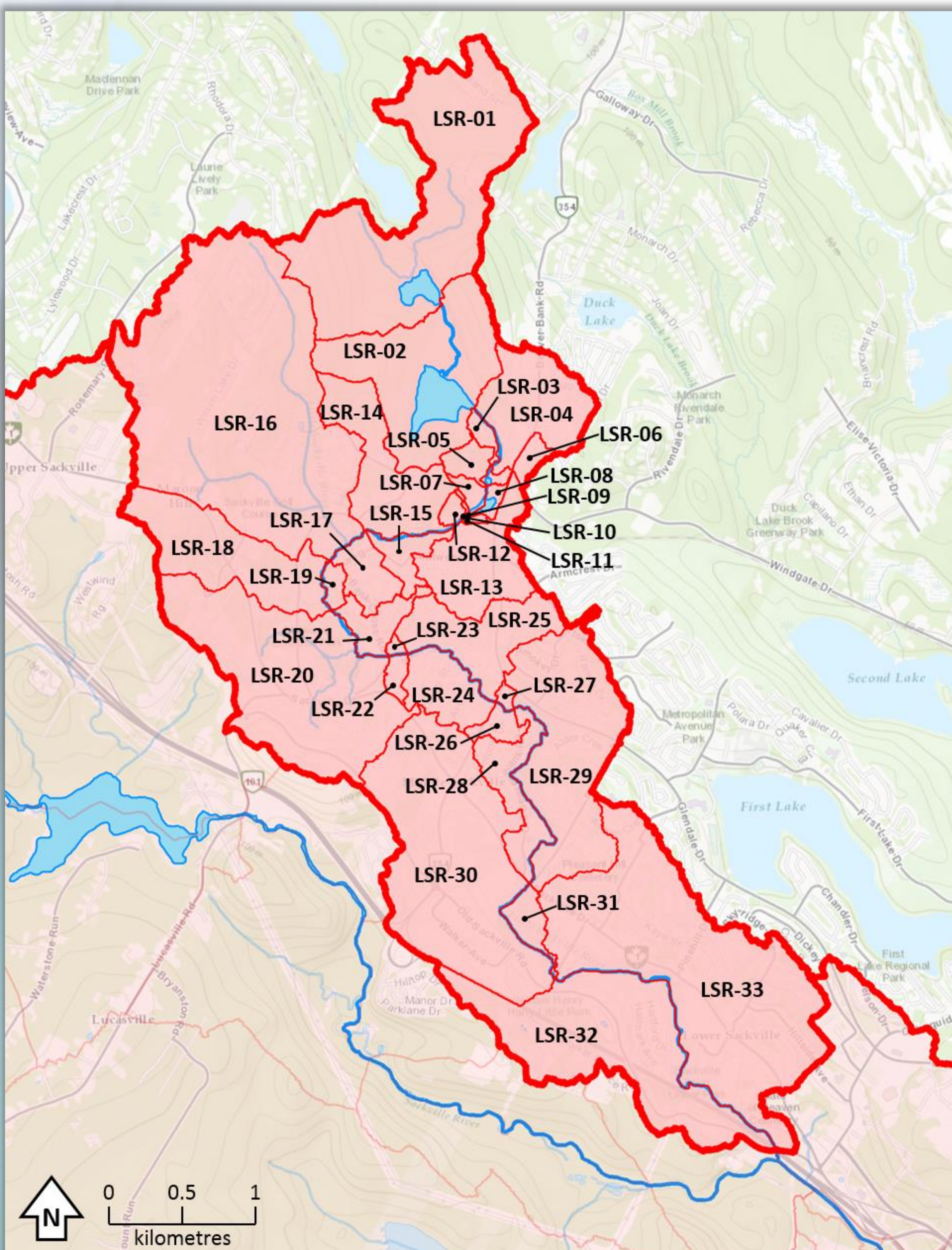


Figure 2.4: **Sub-Watershed Delineation of the Little Sackville River** **(Background Map Source: Esri World Topographic Map)**

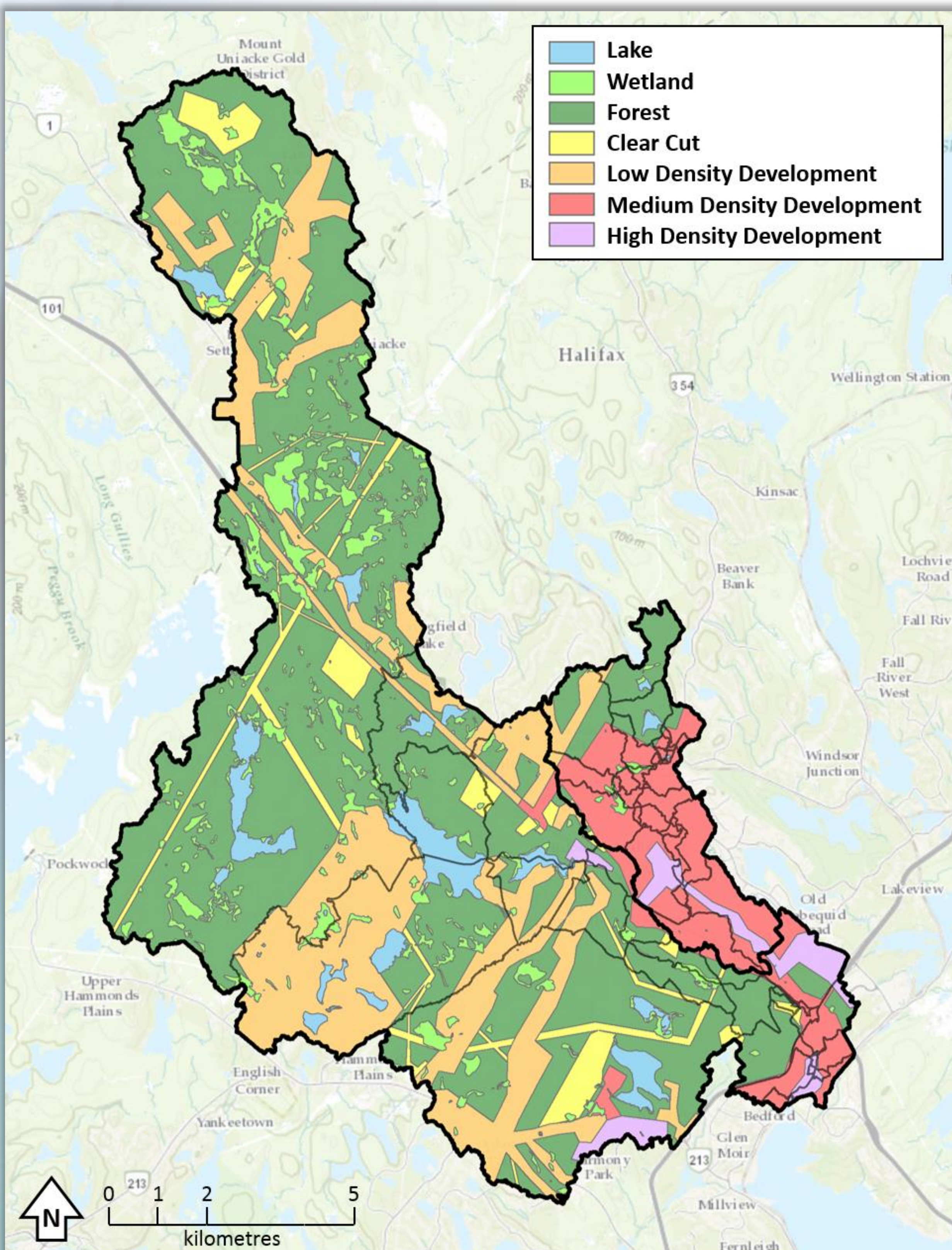


Figure 2.5: Land Cover Mapping of the Sackville River and Little Sackville River Watersheds for Existing Development Conditions **(Background Map Source: Esri World Topographic Map)**

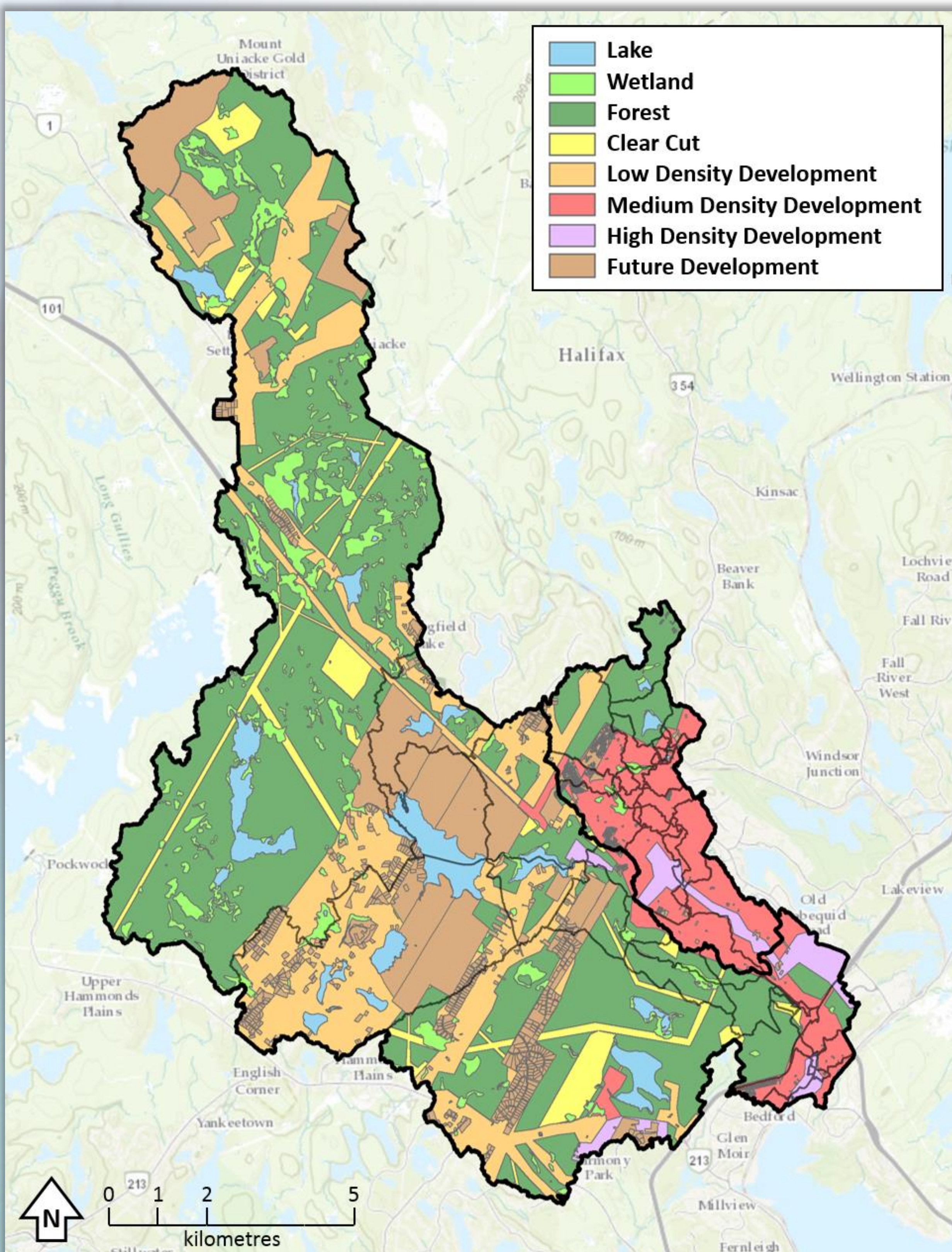


Figure 2.6: Land Cover Mapping of the Sackville River and Little Sackville River Watersheds for Future Development Conditions **(Background Map Source: Esri World Topographic Map)**

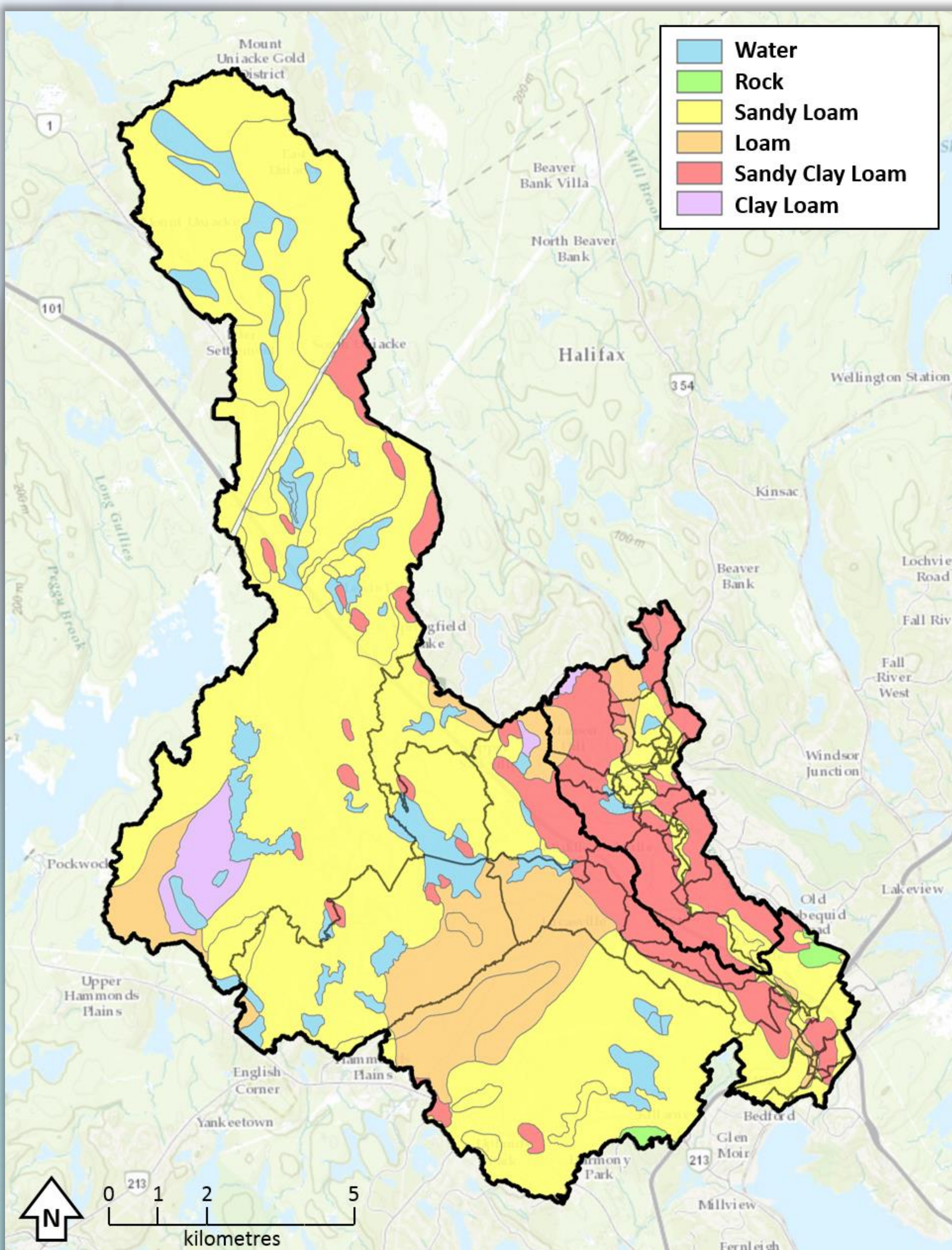


Figure 2.7: Soil Mapping of the Sackville River and Little Sackville River Watersheds (Background Map Source: Esri World Topographic Map)

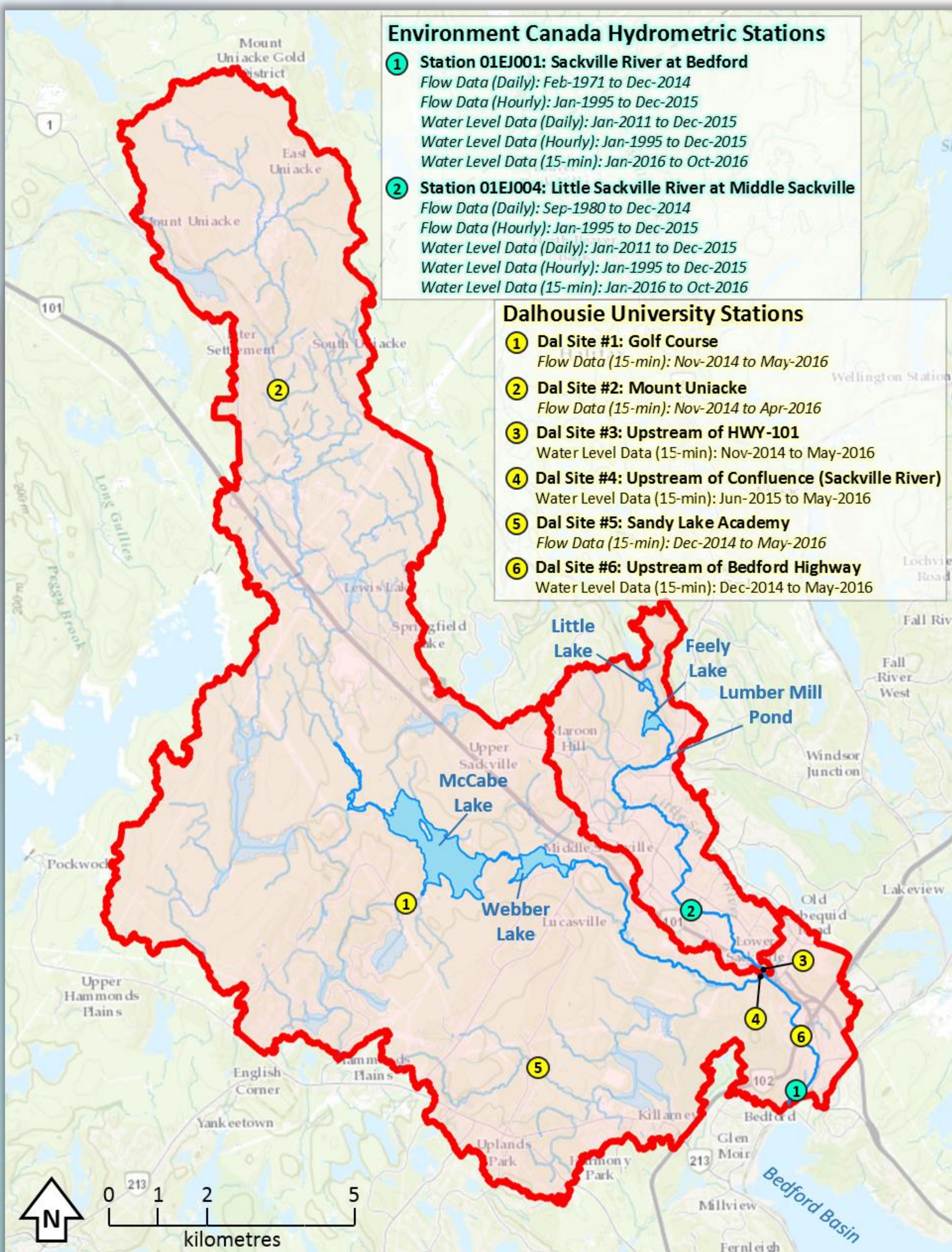


Figure 2.8: **Map of Flow and Water Level Gauging Stations and Data Availability** **(Background Map Source: Esri World Topographic Map)**

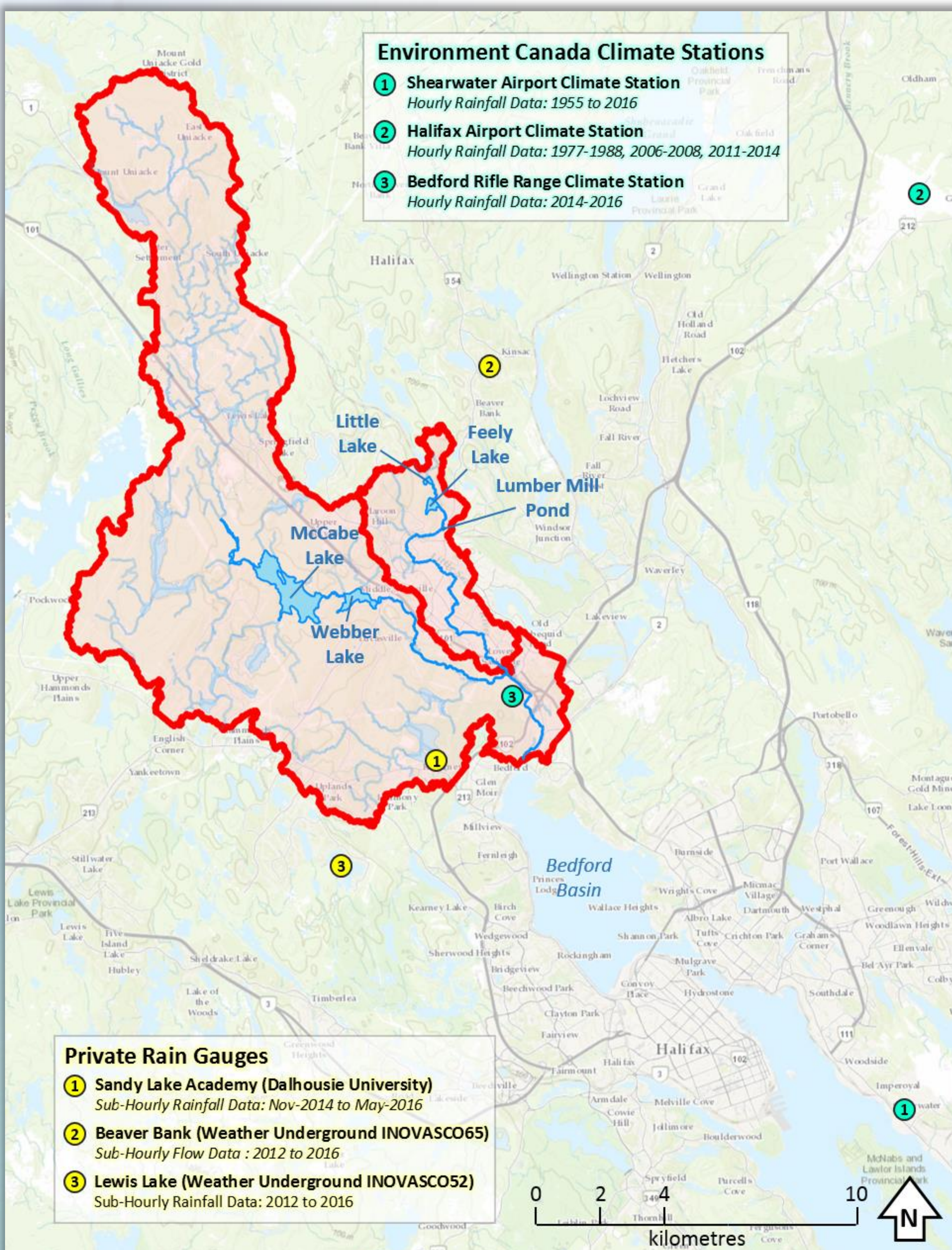


Figure 2.9: Map of Rain Gauging Stations and Data Availability (Background Map Source: Esri World Topographic Map)

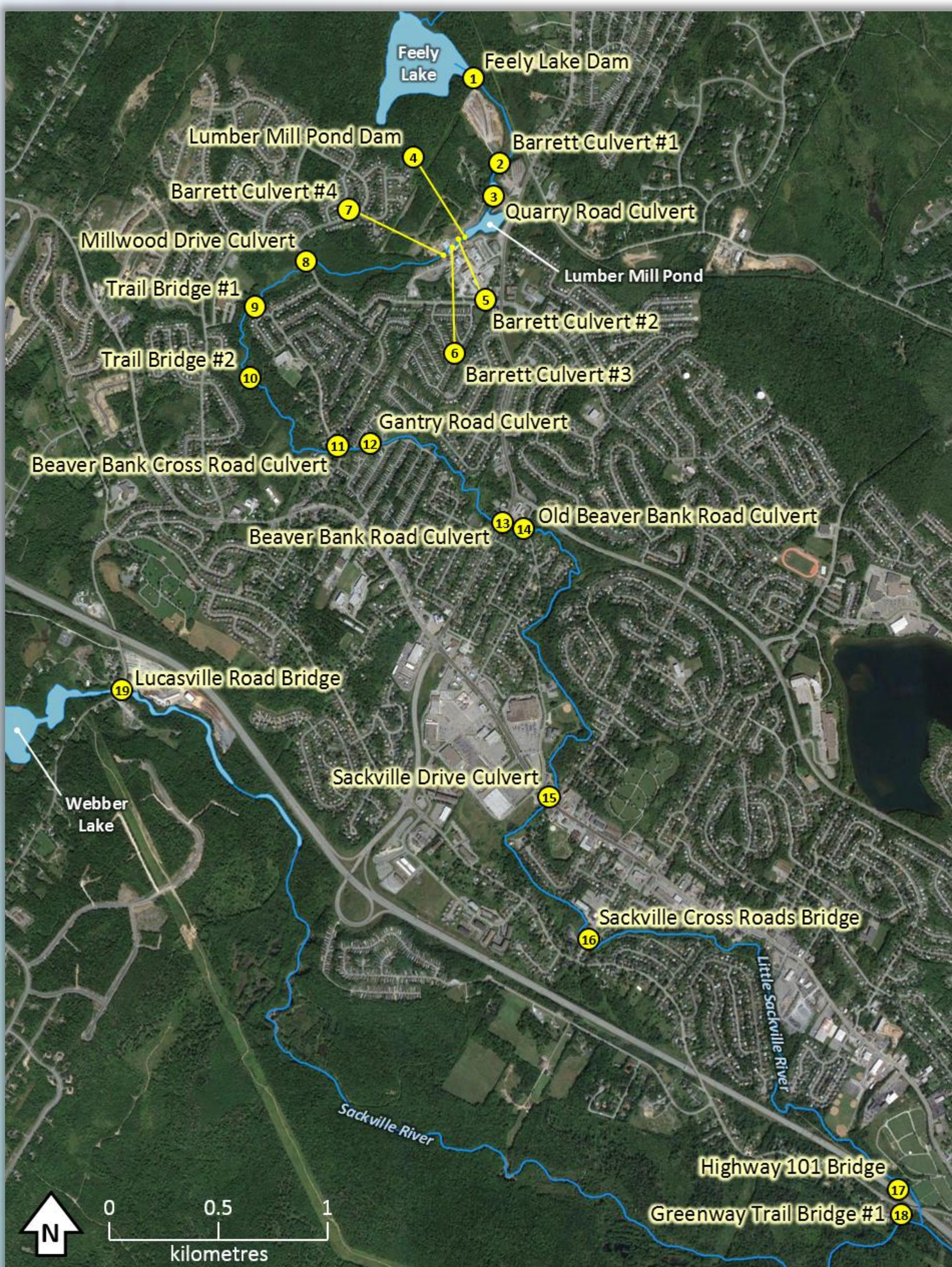


Figure 2.10: Map of Hydraulic Structure Locations (Little Sackville River) (Background Map Source: Google Earth)

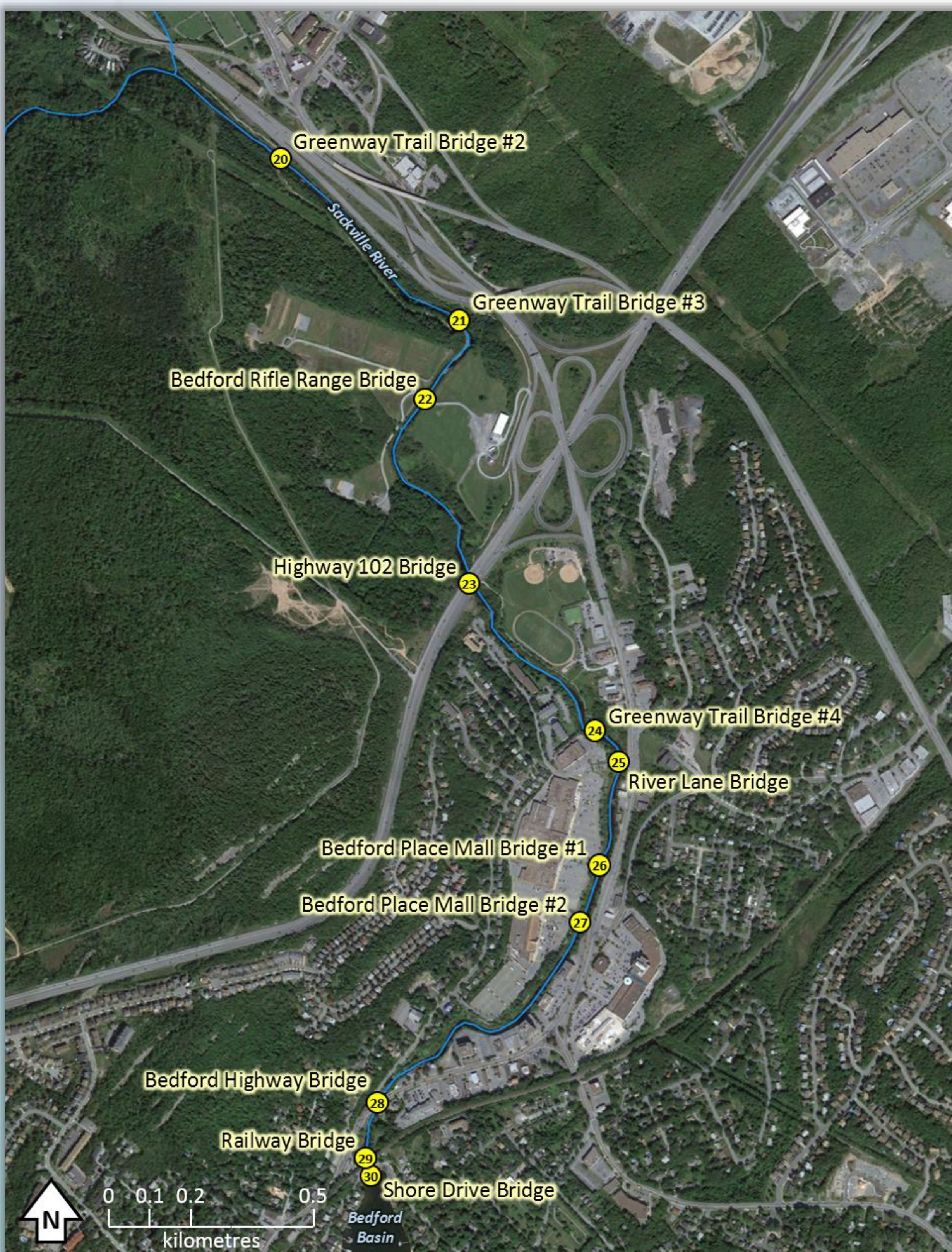


Figure 2.11: Map of Hydraulic Structure Locations (Sackville River) (Background Map Source: Google Earth)

Table 2.2: Summary of Hydraulic Structures within Study Area

Structure #	Structure Name	Bridge ID Number	Available Data[1,2]
1	Feely Lake Dam	-	• CBCL measurements (2016) • Nolan Davis measurements (1987)
2	Barrett Culvert 1	-	• CBCL Measurements (2016)
3	Quarry Road Culvert	-	• CBCL measurements (2016) • Nolan Davis measurements (1987)
4	Lumber Mill Pond Dam	-	• CBCL measurements (2016) • Nolan Davis measurements (1987)
5	Barrett Culvert 2	-	• CBCL Measurements (2016)
6	Barrett Culvert 3	-	• CBCL measurements (2016) • Nolan Davis measurements (1987)
7	Barrett Culvert 4	-	• CBCL measurements (2016)
8	Millwood Drive Culvert	103	• Record drawings (1989)
9	Trail Bridge #1	-	-
10	Trail Bridge #2	-	-
11	Beaver Bank Cross Road Culvert	61	• Nolan Davis measurements (1987)
12	Gantry Road Culvert	-	• Nolan Davis measurements (1987)
13	Beaver Bank Road Culvert	126	• Nolan Davis measurements (1987)
14	Old Beaver Bank Road Culvert	42	• Nolan Davis measurements (1987)
15	Sackville Drive Culvert	118	• CBCL measurements (2013)
16	Sackville Cross Road Bridge	43	• Nolan Davis measurements (1987) • Record drawings (2016)
17	Highway 101 Bridge	40	• Nolan Davis measurements (1987)
18	Greenway Trail Bridge #1	-	• Survey data (2015)
19	Lucasville Road Bridge	41	• Interprovincial Engineering measurements (1981)
20	Greenway Trail Bridge #2	-	• CBCL measurements (2016) • GHD measurements (2015) • Survey data (2015)
21	Greenway Trail Bridge #3	-	• CBCL measurements (2016) • GHD measurements (2015) • Survey data (2015)
22	Bedford Rifle Range Bridge	-	• CBCL measurements (2016) • Survey data (2015)
23	Highway 102 Bridge	-	• GHD measurements (2015) • Survey data (2015)
24	Greenway Trail Bridge #4	-	• GHD measurements (2015) • Survey data (2015)
25	River Lane Bridge	144	• GHD measurements (2015) • Survey data (2015) • Record drawings (1974)
26	Bedford Place Mall Bridge #1	148	• GHD measurements (2015) • Survey Data (2015)
27	Bedford Place Mall Bridge #2	147	• GHD measurements (2015) • Survey Data (2015)
28	Bedford Highway Bridge	15	• GHD measurements (2015) • Survey Data (2015) • Record drawings (1982)
29	Railway Bridge	-	• GHD measurements (2015) • Survey Data (2015)
30	Shore Drive Bridge	44	• GHD measurements (2015) • Survey Data (2015)

[1]Omitting data sources that were found to have outdated information.

[2]Lidar data available at all hydraulic structures.

CHAPTER 3 HYDROLOGIC & HYDRAULIC MODEL DEVELOPMENT

3.1 Modelling Software

Hydrologic and hydraulic modelling was carried out for this study to quantify the flood risks along the Sackville River and the Little Sackville River and delineate flood lines for the flood scenarios presented in **Chapter 4**. All hydrologic and hydraulic calculations were performed using PCSWMM, with the exception of ice jam flood simulation and the encroachment analysis where the hydraulic simulations were instead performed using HEC-RAS.

3.1.1 PCSWMM

PCSWMM is a modelling program developed by Computational Hydraulics International (CHI) that integrates Version 5 of the Storm Water Management Model (SWMM) with a GIS engine and is capable of performing 2D hydrodynamic simulations. SWMM is a hydrologic and one-dimensional hydraulic model produced by the United Stated Environmental protection Agency to study urban drainage systems and is capable of performing unsteady flow calculations to simulate water backup, pooling and culvert hydraulics by dynamically solving the continuity and momentum equations with a finite difference scheme.

This model was selected over the model proposed in the Request for Proposal (RFP) (HEC-RAS) because it is a dynamic model (allows flows and water levels to change in time), it integrates hydrologic and hydraulic calculations (the runoff flows are gradually input over time into the river system that is constantly responding to it), it is more numerically stable (therefore a higher level of confidence in the results) and most importantly, it calculates the impacts of water storage and flow restrictions on the overall flows and water levels in the entire river system, for each time step (in the order of 0.5 seconds for this model). This is critical in a river system that includes a number of lakes and flow constrictions, as well as being tidally influenced in its lower sections. The noted limitations of this model are that it does not include ice jam calculations, and is not set up to carry out encroachment analyses.

3.1.2 HEC-RAS

HEC-RAS is a one-dimensional model developed by the US Army Corps of Engineers and contains an Ice Jam module programmed by the Cold Regions Research Engineering Laboratory. The model is the most advanced and recognized ice jam model available and allows the assessment of risks of ice jam formation, as well as the testing of various ice jam and flood mitigation options. Ice jam simulations in HEC-RAS are carried out by first inputting initial ice cover thickness on the river and its banks prior to the jam and the ice surface roughness. For wide river ice jams, the user also inputs the internal friction angle, porosity, cohesion and maximum average velocity under the ice cover. HEC-RAS then calculates ice build-up in the river, as it is extracted from upstream sections, pushed and packed together by the flow of water, and its effect on water levels.

For these reasons, HEC-RAS was selected for this study to carry out the ice jam modelling tasks. HEC-RAS was also used for this study to carry out the encroachment analysis, following the United States Federal Emergency Management Agency (FEMA) guidelines for encroachment analyses (FEMA 2013).

3.2 Hydrologic Model Development

A hydrologic model of the Sackville River and Little Sackville River sub-watersheds was developed using PCSWMM to estimate runoff flows from each sub-watershed for input into the hydraulic model. Initial watershed characteristics for existing development and future development conditions were estimated for each sub-watershed in the model based on the lidar data, aerial photography, land cover mapping, future development mapping and soil mapping using GIS techniques. Imperviousness and roughness coefficients were estimated for each land cover type and applied to the watersheds using area-weighted averages. The impervious percentages were estimated by measuring and averaging impervious areas for each land cover type, and roughness coefficients were estimated based on values suggested by McCuen *et al*. (1996). The capillary suction head and saturated hydraulic conductivity of the soil were estimated for each soil class from the soil mapping based on values suggested by Rawls *et al.* (1983) and then applied to the watersheds using area-weighted averages. Maximum overland flow lengths were estimated by manually measuring the flow path from the highest point of each sub-watershed to the outlet. Average surface slopes were estimated by calculating the slope between two points at every 20 m throughout the sub-watersheds and then weighing the slopes by drainage area. The percentage of runoff routed from the impervious area to the pervious area was estimated manually for each sub-watershed. The resulting watershed characteristics estimated for each sub-watershed under existing conditions and future development conditions are presented in **Appendix B**. Groundwater was not directly modelled in the hydrologic model since it only represented a small fraction of the peak flow, and also that the software is not currently able to model cross-watershed aquifers or the transfer of water from one aquifer to another. It was however accounted for by modifying the impervious percentage and the pervious surface roughness, simulating a portion of the shallow sub-surface flow that re-emerges in the watercourse.

Since both the hydrologic model and the hydraulic model are integrated into a single model with PCSWMM, calibration of the hydrologic model was performed at the same time as the hydraulic model, as discussed in **Section 3.4**. Watershed characteristics following model calibration are also presented in **Appendix B**.

3.3 Hydraulic Model Development

A hydraulic model of the Sackville River and the Little Sackville River was developed using PCSWMM to estimate the flows and water levels throughout the rivers. The model was developed using the following steps:

1. Converting the existing HEC-RAS model into a PCSWMM model;
2. Updating the model with the compiled hydraulic structure data;
3. Creating and inputting floodplain cross sections based on the 2015 survey data into the model and using the lidar data to modify the floodplain regions of the cross sections;
4. Using the lidar data to create cross sections for the upper reaches;
5. Inputting the lake bathymetry data into the model; and,
6. Using the lidar data to input secondary flow paths and floodplain storage for berm overtopping.

3.3.1 HEC-RAS to PCSWMM Model Conversion

The hydraulic model was developed by first converting the HEC-RAS model from Phase I of the Sackville Rivers Floodplain Study (GHD 2016) into a PCSWMM model. The purpose of the HEC-RAS conversion was to input the information for the 11 hydraulic structures that were previously modelled during Phase I of the study based on record drawings and field measurements. The conversion was carried out using a GIS tool available in PCSWMM.

3.3.2 Inputting Hydraulic Structures

The 11 hydraulic structures input from the HEC-RAS model were then updated using the 2015 survey data, and the remaining 19 hydraulic structures were input into the model based on the 2015 survey data, field measurements, record drawings and measurements from previous reports. Initial estimates of roughness coefficients and loss coefficients were selected based on available photos and inlet/outlet configurations. The lidar data was then used to generate cross sections for the overflow path of hydraulic structures.

3.3.3 Inputting Floodplain Cross Sections from 2015 Survey Data

The 2015 survey points were then used to create floodplain cross sections for the model between the hydraulic structures. A total of 59 cross sections were collected from the survey and input into the model.

Lidar data was then used to add detail to the floodplain area of some of the surveyed cross sections, as the 2015 survey had a lower resolution than the lidar data outside of the river channel. Lidar was therefore used for the floodplain area where it was found that topographical features that could potentially impact floodplain hydraulics were missing from the survey data. A comparison of the survey data and lidar data is presented in **Figure 3.1** at survey cross section #11 (“XS-11”), which is located along the Sackville River immediately downstream of Bedford Place Mall. As shown in the figure, using the lidar data decreases the hydraulic opening between the river banks by approximately 15 m, and also provides important information for the survey gap between stations 217 m and 315 m.

The lidar data was then used to extend most of the surveyed cross sections beyond the limits of the survey. This was carried out because the 2015 survey only included the estimated extent of the 1 in 100 year floodplain and therefore did not include the floodplain extent of larger storm events included in Phase II of the study such as the 1 in 500 year storm, the Probable Maximum Precipitation (PMP) and the 1 in 100 year storm under future climate change and future development scenarios (described in **Chapter 4**). For example, water levels were found to be at close to 10 m elevation at the cross section shown in **Figure 3.1** (XS-11) during the PMP event, whereas the survey data only extents to 7.8 m elevation on one side and 7.3 m elevation on the right bank.

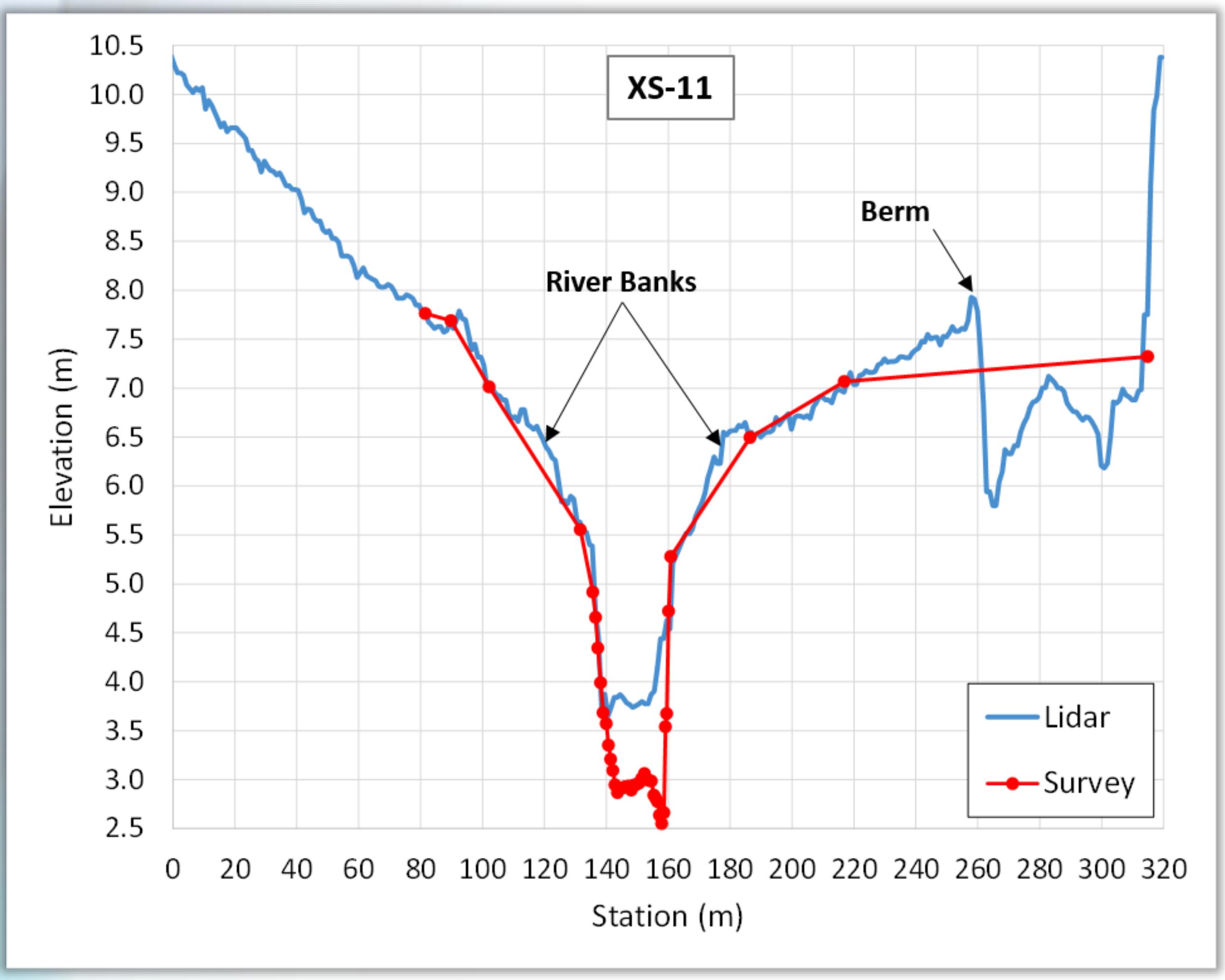


Figure 3.1: Comparison of Lidar Data and Survey Data at Cross Section #11 ("XS-11"), Located Immediately Downstream of Bedford Place Mall

3.3.4 Inputting Floodplain Cross Sections Based on Lidar Data

River cross sections for the Sackville River and the Little Sackville River upstream of the 2015 survey data extents were then assembled and input into the model using the lidar data at a 30 m spacing (see **Figure 3.2**). A resolution of 30 m was needed for these rivers to capture all major changes in the floodplain and channel geometry.

Since standard lidar measurements are unable to penetrate the water surface, the cross sections developed from the lidar data did not include the geometry of the river that was below the water surface on the day that the lidar data was collected. However, based on field observations and survey data, the normal water levels in the two rivers upstream of the confluence were found to be shallow in the range of approximately

0.3 m, which is attributed to the increased river slope in the upstream reaches. Thus, the missing river channel data below the lidar surface was estimated to have minimal impact on evaluating large storm events. Furthermore, the lidar cross sections were modified upstream and downstream of hydraulic structures where survey data was available to improve the estimation of river bathymetry.

A few additional lidar cross sections were then added between the surveyed cross sections for select locations where there were significant changes in floodplain geometry or the spacing between surveyed cross sections was irregularly long. For these additional cross sections, lidar data was used for the floodplain and interpolated survey data was used for the river channel.

3.3.5 Inputting Lake Bathymetry Data

The bathymetry data for McCabe Lake, Webber Lake and Feely Lake were converted into area-depth curves and then input into the model, as SWMM uses area-depth curves to perform storage calculations. Bathymetric data was not available for Little Lake or for the Lumber Mill Pond. For Little Lake, the bathymetry was estimated assuming a depth of 3 m by extrapolation of surface contours. For the Lumber Mill Pond, the bathymetry was estimated based on discussions with Barrett Truss and Building Supplies.

It should be noted that water levels in Feely Lake and the Lumber Mill Pond are regulated by Barrett Truss and Building Supplies. According to the operational procedures carried out by the company, Feely Lake is lowered prior to large rainfall events and during periods of frequent rainfall or snow accumulation throughout the year by removing stop logs at the dam. The stop logs are then replaced during dry periods to ensure continuous flow from the lake, with low flow openings installed during drought conditions. Thus, for the purposes of flood event modelling, it was assumed that stop logs were removed prior to the storm events such that the channel through the dam had an opening height of 0.6 m.

3.3.5.1 Comparison of Lake Modelling Methods

To ensure the most applicable lake modelling approach was used, a comparison was made between modelling lakes as a storage area or as a large channel. McCabe Lake was therefore also modelled as a series of channels instead of a storage area with an area-depth storage curve using the same bathymetric data source.

The comparison was carried out by simulating the 1 in 100 year design rainfall event (described in **Chapter 4**) using both methods and then evaluating the lake flow and water level results. As shown in **Figure 3.3**, the lake flows and water levels estimated by the model using the channel method were sensitive to the channel roughness assigned to the channels. A roughness coefficient of 0.03 was selected as an initial lower limit estimate for the channel roughness based on site observations. This value was then increased in the model until the model produced the same lake flows and water levels as those estimated using the area-depth storage curve method. As shown in the graph, a roughness coefficient of 0.77 was needed to produce the same results. This roughness coefficient also remains within the range of possible values (Chow 1959). Since both methods were able to generate the same results, the storage area method was selected for its higher computational efficiency. Furthermore, the calibration conducted on the June 2016 lake level measurements (see **Appendix C**) demonstrates the applicability of using the storage area method.

3.3.6 *Inputting Secondary Flow Paths and Floodplain Storage*

Berm overtopping was found to occur at two locations in the study area during several of the model scenarios presented in **Chapter 4**: (1) along the Sackville River near its confluence with the Little Sackville River and (2) along the Sackville River at Bedford Place Mall. Secondary flow paths and floodplain storage beyond these berms therefore needed to be modelled as separate but connected channels and storage units. This is illustrated in **Figure 3.1**, where the floodplain cross section for XS-11 needed to be split at the berm. The two dimensional channel configuration in the model for these two locations is presented in **Figure 3.4**.

3.3.6.1 Comparison of 1D and 2D Modelling Approach

An integrated 1D-2D model was developed for the floodplain area near the confluence of the Sackville River with the Little Sackville River to compare a 2D modelling approach with the 1D modelling approach for this location, as well as to check the quality of the results. The 2D model component was developed in PCSWMM 2D using a 5 m hexagonal grid based on the lidar data and using the same channel and floodplain roughness coefficients used for the 1D model. To simulate flows in the channel below the elevation of the lidar surface, an integrated 1D channel was modelled.

The comparison between the 1D and 2D modelling approach was carried out by inputting the 1 in 100 year design rainfall event (described in **Chapter 4**) and the design model parameters (described in **Section 3.5**) into the respective models. Peak water levels estimated by both modelling approaches were then compared at multiple locations along the floodplain area near the confluence. A map of the locations that were compared is presented in **Figure 3.5**, which also includes the peak water depths estimated by the 2D model for the 1 in 100 year rainfall event. As shown in **Table 3.1**, the 1D modelling approach generally estimated slightly higher water levels than those estimated by the 2D modelling approach. Ultimately, the 1D modelling approach was selected for the area near the confluence since it produced more conservative results and is consistent with the modelling approach of the remainder of the two rivers.

Table 3.1: Comparison of Peak Water Levels for 1D & 2D Modelling Approaches at Confluence

Location #	1 in 100 Year Peak Water Level [geodetic] (m)		Difference in Water Level (m)
	1D Modelling Approach	2D Modelling Approach	
1	14.40	14.17	-0.23
2	13.89	13.44	-0.45
3	10.77	10.83	+0.06
4	12.85	12.68	-0.17
5	12.85	12.62	-0.23

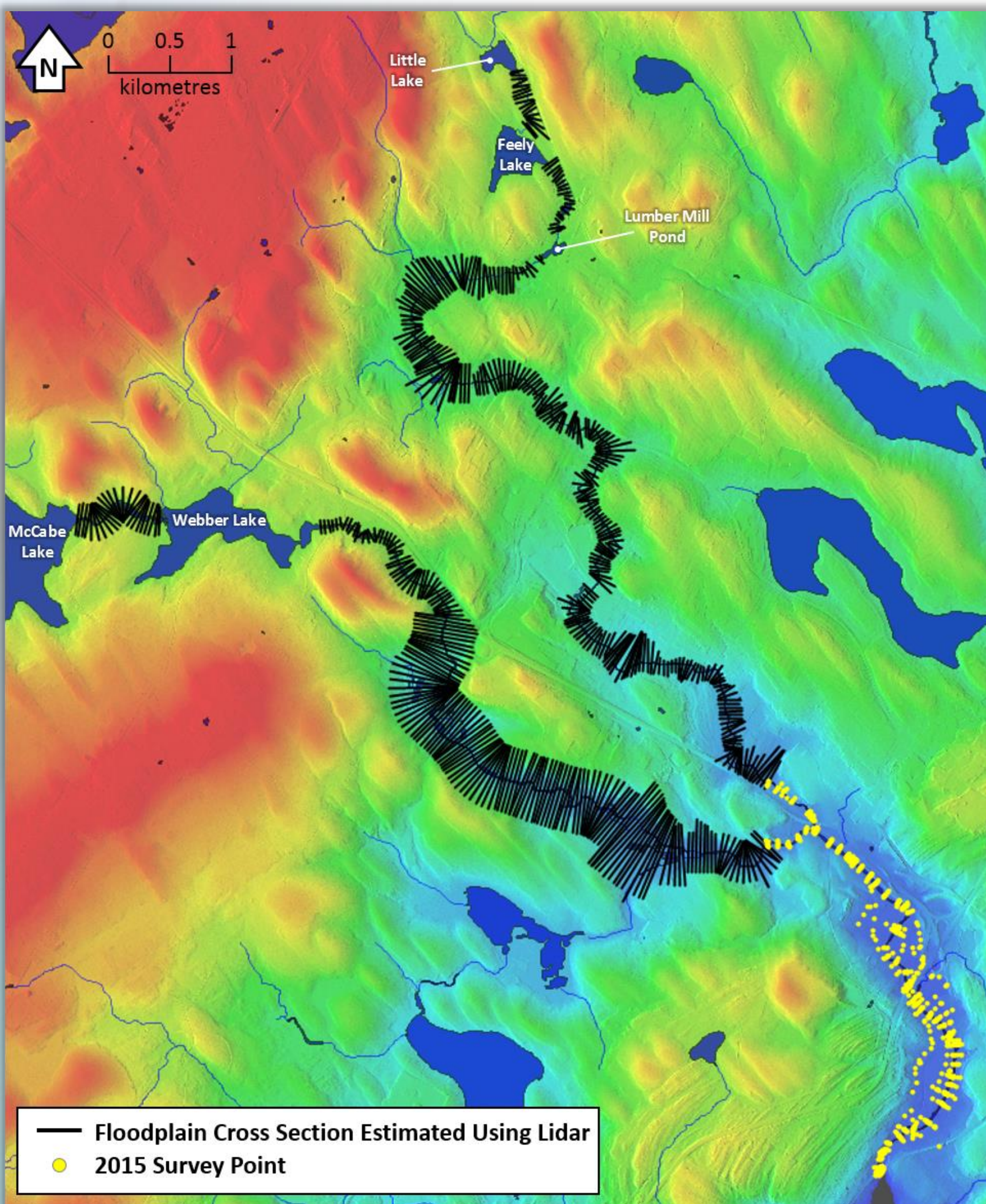


Figure 3.2: Lidar Cross Sections and Survey Points used to Develop Hydraulic Model

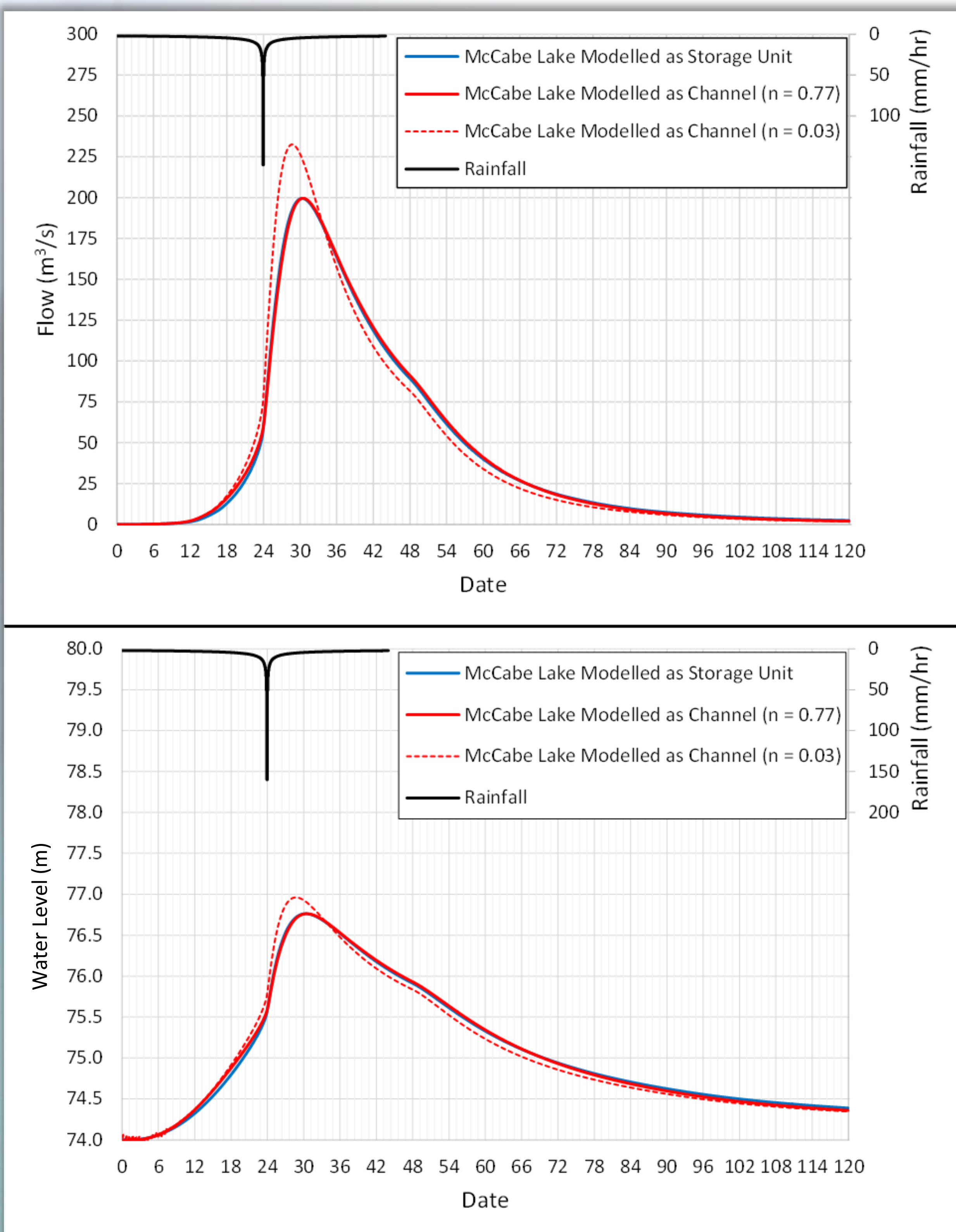


Figure 3.3: Comparison of Methods for Simulating Hydraulics of McCabe Lake based on 1 in 100 Year Rainfall Event

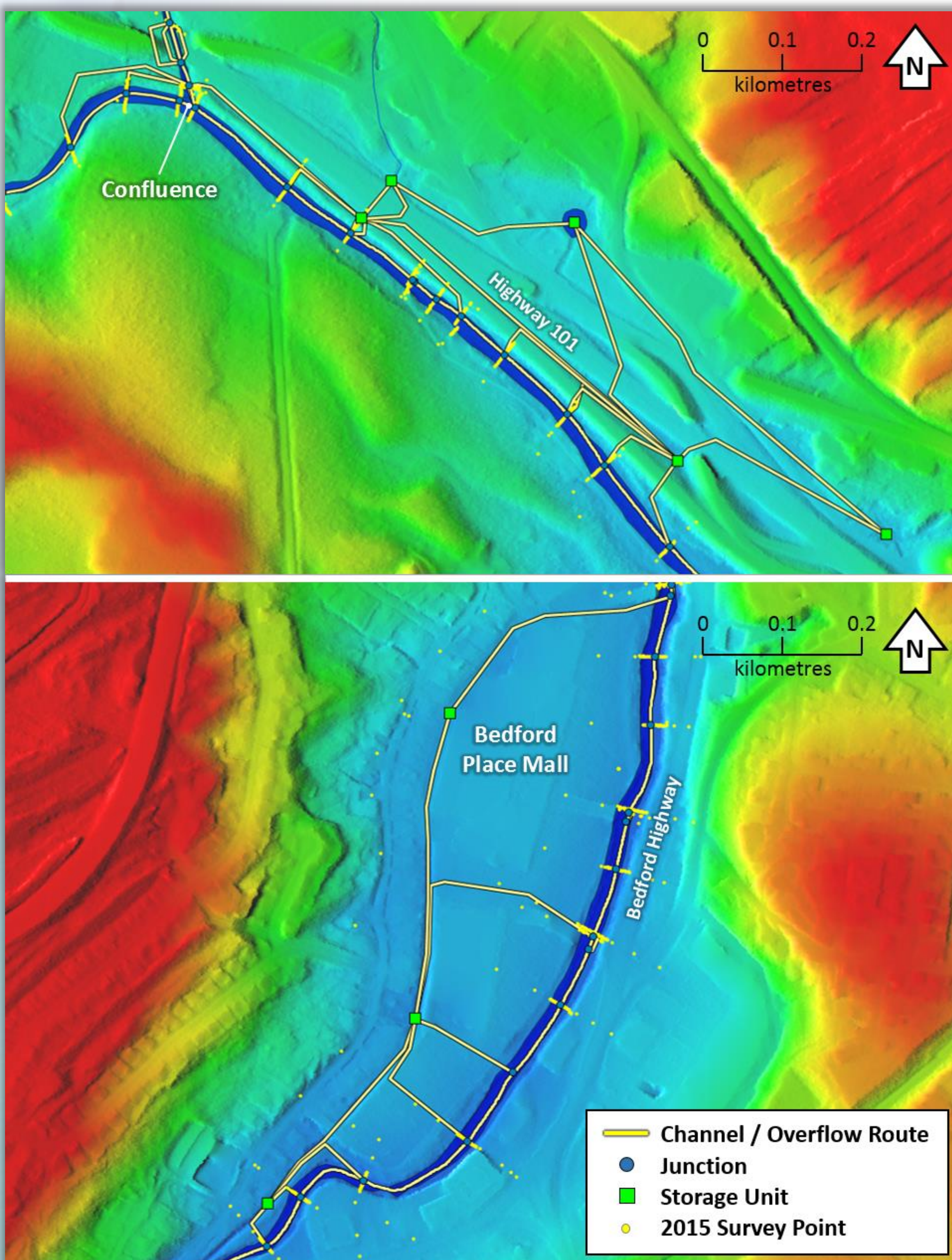


Figure 3.4: Secondary Flow Paths, Floodplain Storage and Berm Overflow Routes in Model for the River Confluence (top) and Bedford Place Mall (bottom)

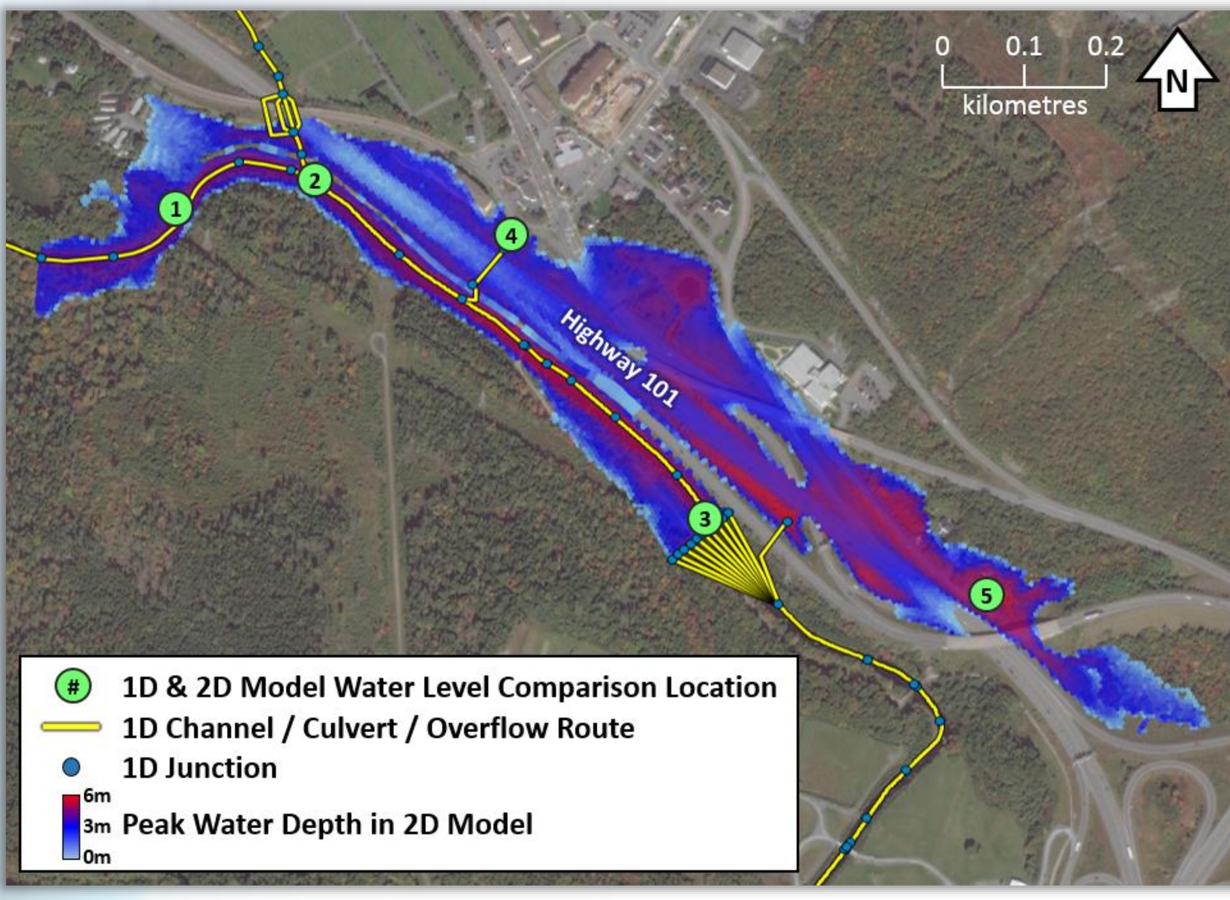


Figure 3.5: Map of 1D and 2D Model Result Comparison Locations, Including Peak Water Levels Estimated by 2D Model for 1 in 100 Year Storm **(Background Map Source: Bing Maps Aerial)**

3.4 Hydrologic & Hydraulic Model Calibration and Validation

Hydrologic and Hydraulic model calibration was carried out by inputting historical rainfall data and initial flow conditions for selected flood events into the PCSWMM model and then modifying the initial estimates of watershed characteristics and channel roughness coefficients until the flows and water levels simulated by the model were representative of historical values. Model validation was then performed by simulating secondary flood events and comparing the model results with the historical data. It should be made clear that model validation does not involve modifying the model parameters, it simply involves comparing the calibrated model results against measured data. One calibration event and one validation event was selected for each season, and for each of the two rivers. Event selection was based on both the magnitude of the flood and the availability of rainfall, flow and water level data. Events with the largest amounts of flooding and data availability were selected for model calibration, and events with the second largest amounts of flooding and data availability were selected for model validation. A summary of the events selected for model calibration and validation is presented in **Table 3.2**.

Table 3.2: Selected Calibration and Validation Events

Season	River	Calibration or Validation	Event Date	Best Available Rainfall Data	Available Flow Data	Available Water Level Data	Snowmelt Event?
Spring	Little Sackville River	Calibration	Apr-2015	Radar Data	01EJ004	01EJ004, Site 3	yes
		Validation	Mar-2003	Shearwater A	01EJ004	01EJ004	yes
	Sackville River	Calibration	Mar-2003	Shearwater A	01EJ001	01EJ001	yes
		Validation	Apr-2015	Radar Data	01EJ001	01EJ001, Site 6	yes
Summer	Little Sackville River	Calibration	Aug-1971	Shearwater A	[1]none	[1]none	no
		Validation	Jul-1981	Shearwater A	01EJ004	none	no
	Sackville River	Calibration	Aug-1971	Shearwater A	01EJ001	none	no
		Validation	Jul-1981	Shearwater A	01EJ001	none	no
Fall	Little Sackville River	Calibration	Nov-2015	Bedford Rifle Range	01EJ004	01EJ004, Site 3	no
		Validation	Nov-2004	Shearwater A	01EJ004	01EJ004	no
	Sackville River	Calibration	Nov-2010	Shearwater A	01EJ001	01EJ001	no
		Validation	Nov-2015	Bedford Rifle Range	01EJ001	01EJ001, Site 4, Site 6	no
Winter	Little Sackville River	Calibration	Dec-2000	Shearwater A	01EJ004	01EJ004	no
		Validation	Jan-2016	Radar Data	none	01EJ004, Site 3	yes
	Sackville River	Calibration	Dec-2014	Radar Data	01EJ001	01EJ001, Site 6	no
		Validation	Jan-2016	Radar Data	none	01EJ004, Site 4, Site 6	yes

[1]No significant summer storm event was identified for the Little Sackville River where either flow or water level data was available. Parameter adjustment factors used for the August 1971 calibration for the Sackville River were therefore applied to the Little Sackville River.

3.4.1 Initial Conditions for Calibration & Validation Events

Initial flow conditions input into the model for each calibration and validation event were set such that the flow rates in the model at the available flow gauging stations matched those recorded at the respective stations. This was performed by running the model with historical rainfall data prior to the respective storm event until the initial conditions were met. The process of running the model to develop initial conditions for successive model runs allows for the watershed flows, snowmelt, river water levels and river flows throughout the watershed to be in a stable and dynamic state at the beginning of the successive model run.

For calibration and validation events where snowmelt was present, the initial conditions set in the model included historical snow depths. Snow depth amounts were obtained from the Environment Canada website on historical weather data. Snowmelt calculations were then performed by the hydrologic model by inputting historical rainfall, temperature and wind data. Model parameters for snowmelt events were set such that the model would simulate the historical daily snow depths obtained from Environment Canada for the Shearwater Airport and Halifax Airport climate stations.

3.4.2 Radar Rainfall Data

This section describes the applicability of radar rainfall data for this study, and how the radar rainfall data obtained from Environment Canada was calibrated to local rain gauging stations such that it could be used to improve the calibration of the hydrologic and hydraulic model. It is noted that since radar rainfall analyses are a topic of current research, the information presented below makes frequent use of references to ensure that statements and analyses are supported by current research papers.

3.4.2.1 Applicability of Radar Rainfall Data in HRM

Rain gauges are fairly accurate and one of the best available tools for rainfall measurements, but inherently have limited spatial coverage. In all of HRM, only a handful of stations are active, and with intermittent operation. Only one Environment Canada climate station falls within the Sackville River Watershed, and none are found within the Little Sackville River Watershed. When no other information is available, rain gauges are typically assumed to be representative of the watersheds in which they lie as well as surrounding watersheds, and they are used as the basis of stormwater assessments and flood studies (Cristiano *et al*. 2016). However, this is problematic because rainfall is known to be highly spatially variable, and rainfall at one location can be vastly different than at another location even 1 km away (Berne and Krajewski 2013). **Figure 3.6** demonstrates the differences in rainfall measurements at different locations as a storm passes through the Sackville River and Little Sackville River watersheds. It is well known that spatiotemporal variability in precipitation (i.e., due to storm characteristics, speed, and direction) affects drainage system response (e.g., Cristiano *et al*. 2016). This has been shown to be especially important in urban areas, where hydrological processes are characterized by high variability in space and time (Bruni *et al*. 2015). Rainfall is one of the main sources of uncertainty in flood studies (Rico-Ramirez *et al*. 2015), so poor representation of rainfall patterns can be detrimental, and effort should be dedicated to obtaining the best possible representation of rainfall.

Resolution from radar data (1 km^2 grid) is vastly superior to what can be obtained using a network of rain gauges, so radar data can supplement rain gauging data by providing more detail on the spatial distribution and movement of storm cells across watersheds (Finney & Blades, 2014). The use of radar to estimate ground level rainfall is not new (Hunter 1996), and application of weather radar in urban hydrological studies has evolved significantly during the past decade (see Berne and Krajewski (2013) for review; Thorndahl *et al*. 2016). Since rain gauges and radar both have their advantages and disadvantages, it is best to merge estimates from the two to combine their strengths and minimize their weaknesses (Smith *et al*. 2012; McKee 2015).

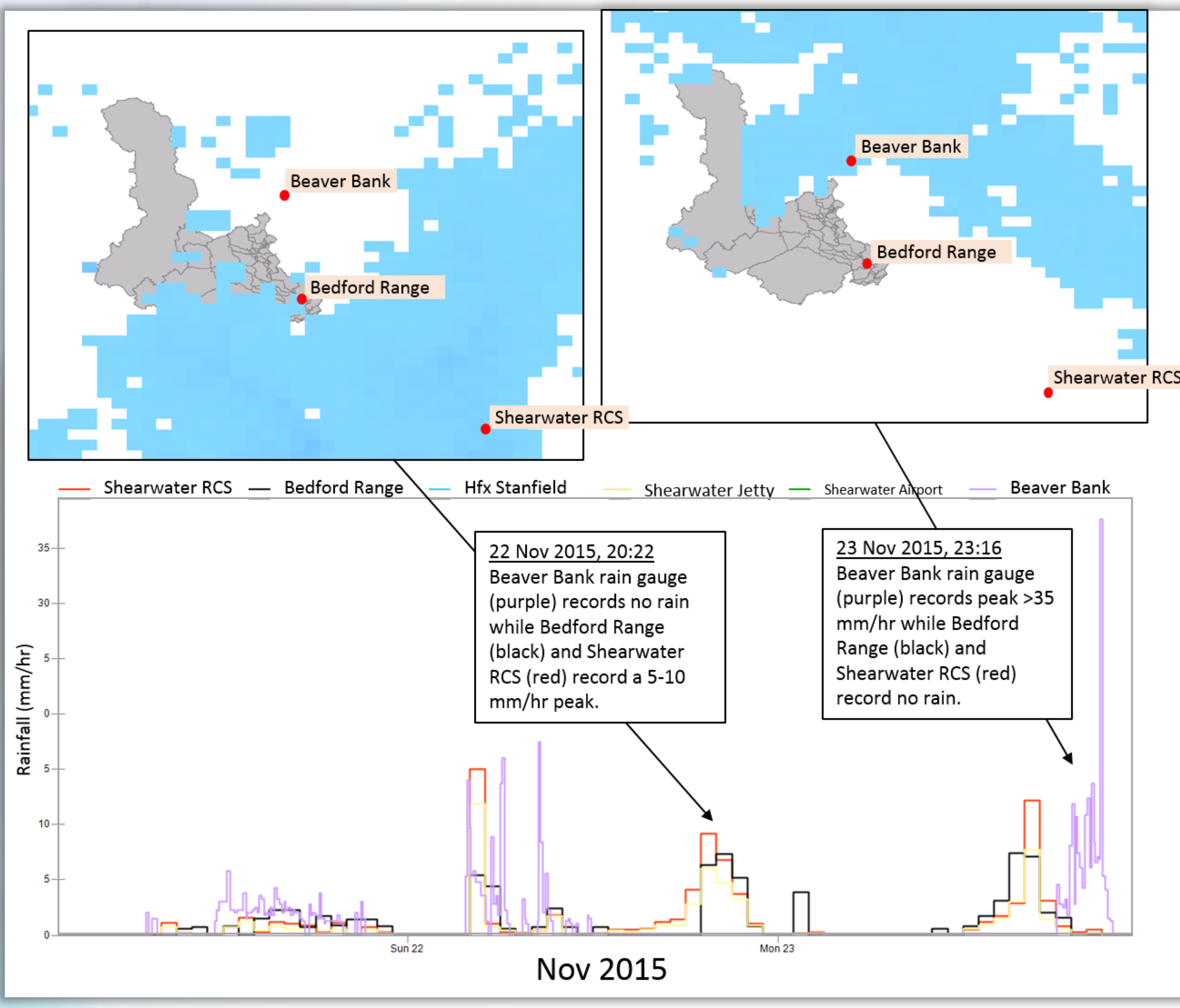


Figure 3.6: Spatial Variability of Rainfall Demonstrated by Rain Gauge Measurements at Different Locations near the Sackville River and Little Sackville River Watersheds

CHI, in partnership with Halifax Water (HW), compared several methods for merging radar data with rain gauge measurements in HRM, thus demonstrating the potential applicability of the method locally (Finney & Blades, 2014). CHI and HW used PCSWMM to process, calibrate, and integrate the radar with rain gauge data (see James *et al*. 2008 for description of PCSWMM radar capabilities). The CHI and HW study looked at 21 rainfall events, and results from three of these rainfall events are publically available (Finney & Blades, 2014). **Figure 3.7** shows the radar rainfall events that were calibrated to rain gauge measurements using the “Rain Gauge Values” method available in PCSWMM. Depending on the rainfall event, the calibrated radar rainfall was found to overestimate (Nov 2013 event, **Figure 3.7(a)**), closely match (Dec 2013 event, **Figure 3.7(b)**) or underestimate (Jan 2014, **Figure 3.7(d)**) the raw rain gauge rainfall. However, a strong correlation between radar-derived rainfall and rain gage rainfall can be seen when all rainfall events are combined (R^2 = 0.96; **Figure 3.7(d)**).

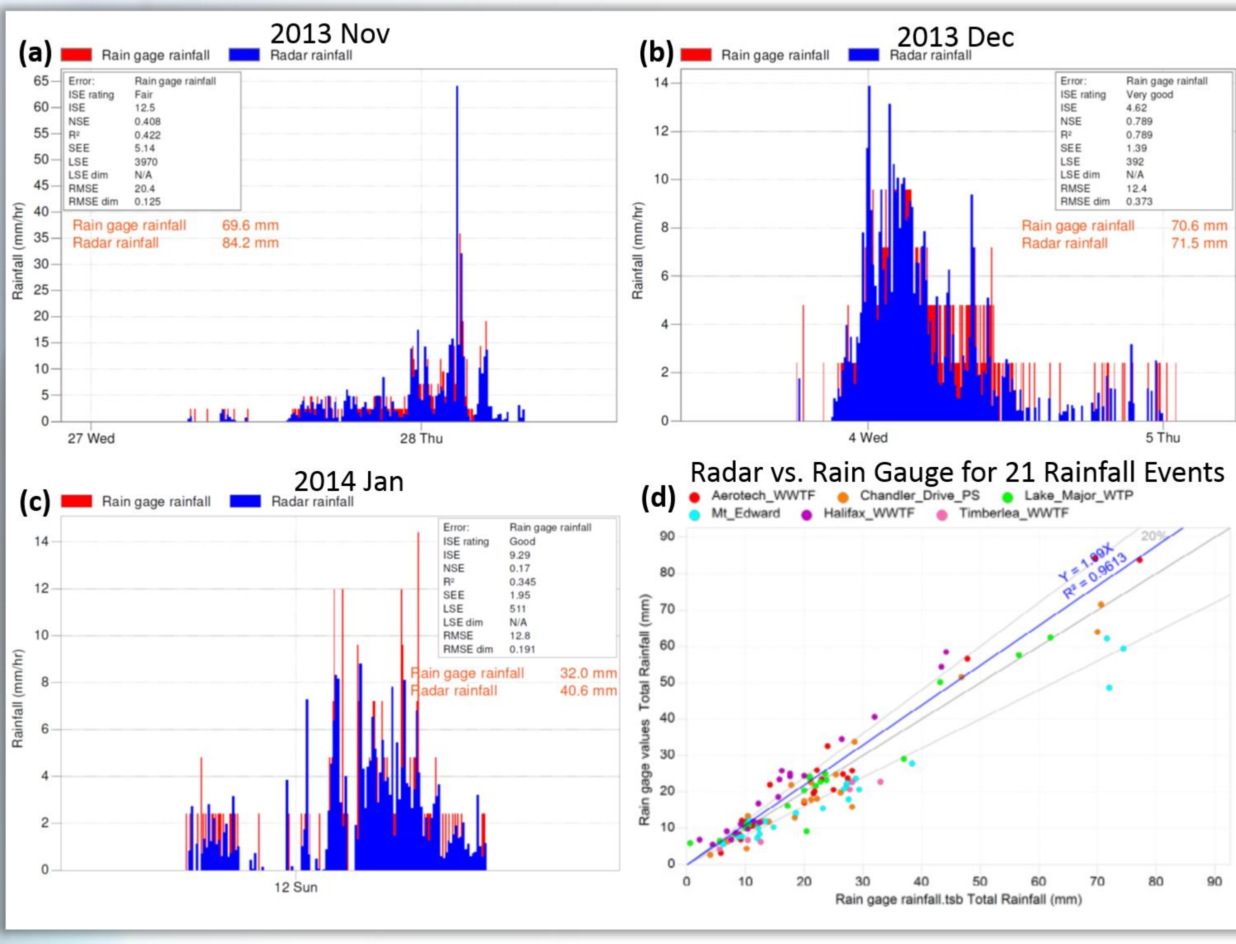


Figure 3.7: Comparison of Radar Rainfall Values Obtained using PCSWMM's "Rain Gauge Values" with Raw Rain Gauge Data (CHI-HW collaboration, Finney & Blades, 2014)

Although the use of radar in hydrological studies is a relatively new method (with many hydrological studies still using point source data), given that radar data is available and that its applicability has been tested locally, it is due diligence to use the state-of-the-art method to obtain the best possible representation of rainfall in the Sackville River and Little Sackville River watersheds during calibration events. It is also noted that this tool is not new to CBCL Limited, as we have previously used the method successfully for flood studies in the Truro, Falmouth, Eskasoni and Flatrock (NL) regions.

3.4.2.2 Steps Undertaken

The following radar rainfall data and rain gauging data was obtained for model calibration:

- Environment Canada radar products are available from the Meteorological Service of Canada Atlantic Operations. However, after initial discussions and trials with data samples, it was found that the format available through the Atlantic office cannot be manipulated without specialized software. The radar data was therefore obtained from the Environment Canada Canadian Meteorological Centre in Dorval, Quebec.
- Hourly rain gauging data was obtained through the Environment Canada Climate Atlantic office. Rainfall events were extracted by Environment Canada staff by querying their databases. For each calibration event, data was available from some of the rain gauges in the region, in either hourly or

6-hourly format, from either weighing gauges or tipping buckets, and in various stages of post-processing. It should be noted that some of the rain gauges had missing data.

- In addition to the Environment Canada rain gauging data, hourly rainfall data was extracted from private rain gauges using an in-house data extraction program. The private rain gauging data was then quality checked since it had not undergone post-processing by Environment Canada staff. This quality check involved ensuring that the total gauged monthly rainfall amounts were consistent with Environment Canada stations.

The radar data was then processed and calibrated to the rain gauging data using the following steps:

1. A radar rainfall storage database was assembled for the large radar rainfall datasets.
2. Processing and filtering of radar rainfall was optimized by testing different methods and settings. This was carried out using several reflectivity-rainfall relationships in PCSWMM to transform radar intensity readings into rainfall depths.
3. Removal of the "bias" in the radar rainfall data was conducted by comparing radar values with rain gauge measurements. This is also referred to as "ground truthing" or "calibration" of radar data using rain gauges measurements. This step was necessary to determine, for each event, how radar rainfall is best merged with rain gauge measurements.
4. Spatial weighting was then conducted within each sub-watershed to obtain a time series of rainfall at that particular sub-watershed.
5. For some of the calibration events, complications arose with the data processing that required extensive troubleshooting. These issues were resolved by working in close collaboration with CHI developers of the PCSWMM software. As a result, some changes were made to the PCSWMM software (fixed bug on data import, resolved incorrect grid conversion process). Through this troubleshooting, it was determined that complications with the radar data processing for three of the selected calibration storm events (November 2004, November 2010 and November 2015) were likely due to errors with the radar data. Rain gauging data was therefore used for these events in lieu of radar data.

For the three events where the data processing was able to be completed by the software (December 2014, April 2015 and January 2016), and for which the various corrections noted above were conducted, the resulting radar rainfall data was then scaled (the data was prorated to match the total daily rainfall amount of the calibration event at the closest rain gauge location) to improve the representativeness of the rain gauging observations in the study area. It should be noted that daily rainfall measurements are considered more accurate than hourly rainfall measurements, especially during heavy rainfall events. The ratios were then used to scale radar rainfall obtained for each of the sub-watersheds.

3.4.3 Model Parameter Adjustments

The adjustments that were made to the model parameters to calibrate the hydrologic and hydraulic model to the various storm events are presented in **Table 3.3** for the Sackville River and **Table 3.4** for the Little Sackville River. These tables are also represented graphically in **Figure 3.8**. The model parameters that were adjusted for calibration were the maximum overland flow lengths, impervious area percentages, watershed roughness coefficients for impervious areas and channel roughness coefficients. These parameters vary seasonally, as, for example, frozen ground will impact flow paths, infiltration and roughness. While the impact of other model parameters including hydraulic conductivity, capillary suction head, depression storage (for pervious and for impervious areas), percent routed (towards pervious or impervious areas) and hydraulic loss coefficients were tested during calibration, the initial estimates of these parameters were ultimately unchanged.

Table 3.3: Parameter Adjustments from Model Calibration for the Sackville River

Season	Event	[1]Parameter Adjustment (factor)			
		Maximum Overland Flow Length	Percent Impervious	Watershed Roughness [Impervious Area] (Manning's n)	Channel Roughness (Manning's n)
Spring	Mar-2003	5	2.10	1.50	2.22
Summer	Aug-1971	11	2.49	1	2.44
Fall	Nov-2010	1	2.41	1	1.77
Winter	Dec-2014	1	2.41	0.50	2.44

[1]Parameter adjustment of 1 indicates no change in initial estimate.

Table 3.4: Parameter Adjustments from Model Calibration for the Little Sackville River

Season	Event	[1]Parameter Adjustment (factor)			
		Maximum Overland Flow Length	Percent Impervious	Watershed Roughness [Impervious Area] (Manning's n)	Channel Roughness (Manning's n)
Spring	Apr-2015	1	1	1	1
Summer	Aug-1971	11	2.10	1	2.44
Fall	Nov-2015	1	2.04	1.70	1.11
Winter	Dec-2000	0.5	1	1	1.55

[1]Parameter adjustment of 1 indicates no change in initial estimate.

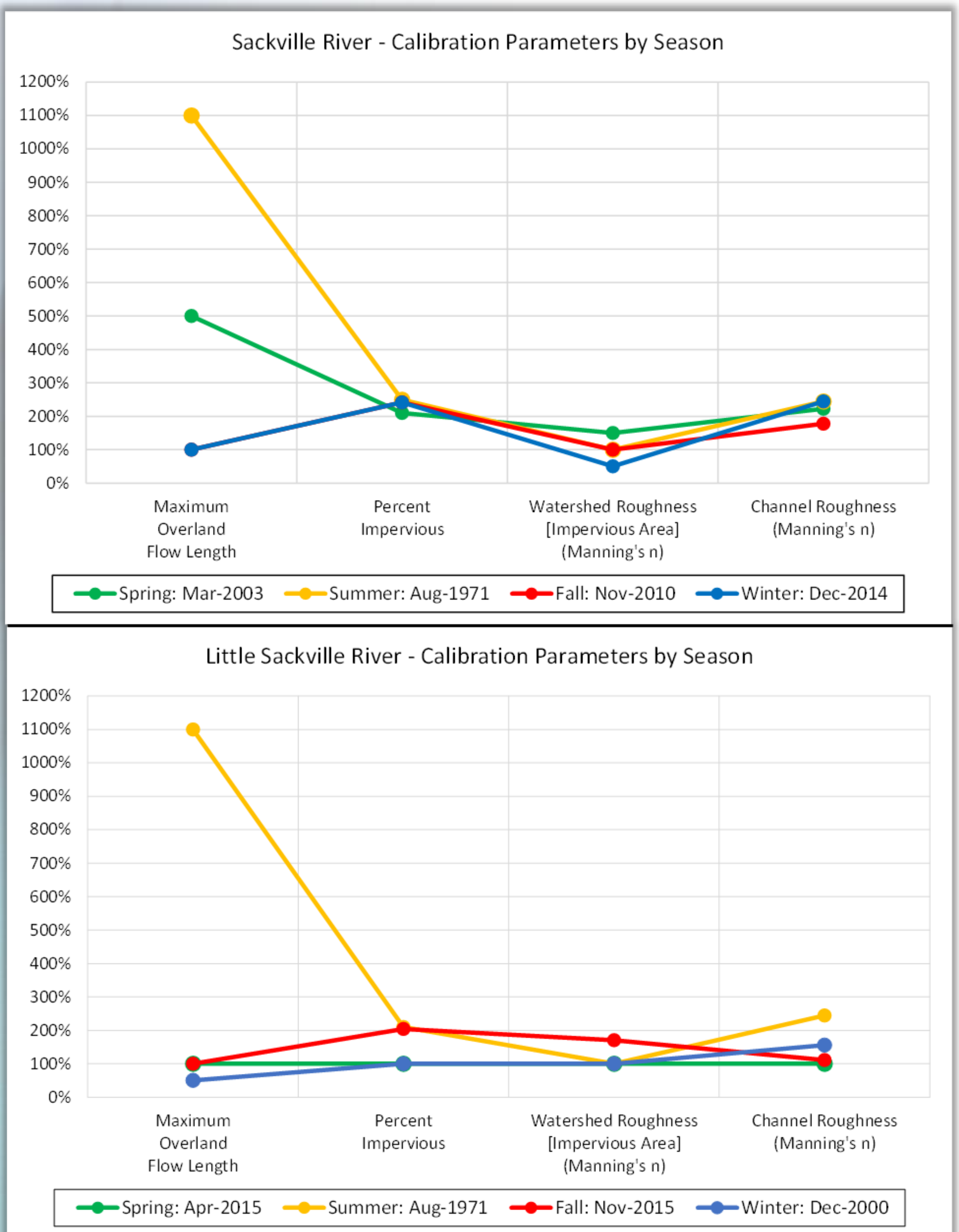


Figure 3.8: Model Parameter Calibration Adjustments for the Sackville River (top) and the Little Sackville River (bottom)

3.4.4 Calibration & Validation Results

The calibration and validation results for all storm events are presented graphically in **Appendix C**. A summary of the calibration and validation results with respect to peak flow rate is presented in **Table 3.5** for storm events where historical peak flow rates were available. As shown in the table, the model was able to satisfactorily reproduce the measured water level and flow data. This level of reproducibility is typical of hydrologic modelling analysis. Most of the percent differences in peak flows (measured versus calculated) are within 5%. The largest discrepancies were found to be for the Little Sackville River December 2000 event and for the Sackville River November 2010 event. In the calibration process, only seasonally variable parameters were adjusted, meaning that limitations were placed on the model's ability to closely reproduce the measured data. Fortunately, the events with the greatest discrepancies were not ultimately selected for the design model parameters, as described in the following section.

Table 3.5: Summary of Peak Flow Calibration and Validation Results

Season	River	Flow Gauging Station	Calibration or Validation	Event Date	Measured Peak Flow (m^3/s)	Simulated Peak Flow (m^3/s)	Percent Difference in Peak Flow
Spring	Sackville River	01EJ001	Calibration	Mar-2003	105.7	105.1	-1%
			Validation	Apr-2015	66.8	55.1	-18%
	Little Sackville River	01EJ004	Calibration	Apr-2015	14.5	14.4	-1%
			Validation	Mar-2003	[1] -	-	-
Summer	Sackville River	01EJ001	Calibration	Aug-1971	85.0	85.8	+1%
			Validation	Jul-1981	22.0	31.9	+45%
	Little Sackville River	01EJ004	Calibration	Aug-1971	[2] -	-	-
			Validation	Jul-1981	[3] 9.3	11.2	+20%
Fall	Sackville River	01EJ001	Calibration	Nov-2010	67.7	83.3	+23%
			Validation	Nov-2015	54.0	40.3	-25%
	Little Sackville River	01EJ004	Calibration	Nov-2015	9.7	9.8	+1%
			Validation	Nov-2004	15.8	9.1	-42%
Winter	Sackville River	01EJ001	Calibration	Dec-2014	85.0	88.7	+4%
			Validation	Jan-2016	[4] -	-	-
	Little Sackville River	01EJ004	Calibration	Dec-2000	21.6	28.9	+34%
			Validation	Jan-2016	[4] -	-	-

[1]Data for March 2003 event fragmented and incomplete for 01EJ004.

[2]Data for August 1971 event not available for 01EJ004.

[3]Measured peak flow for July 1981 event at 01EJ004 based on applying a peaking factor to the recorded average daily flow.

[4]Flow data for January 2016 event not yet available for 01EJ001 and 01EJ004 from Environment Canada.

3.4.4.1 Uncertainty Associated with the Models

Although models have been calibrated to reproduce historical events in a representative manner, there is still some uncertainty associated with the models. This is typical of any hydrologic and hydraulic model, and the typical sources of uncertainty include:

- Measurements of hydrologic parameters (soil infiltration, surface roughness, effective impervious area);
- Hydraulic loss parameters (channel roughness, energy losses at structures);
- Computational uncertainty (computational/iteration schemes used to resolve finite difference hydrodynamic equations);
- Calibration data uncertainty (flow data, water level data, amount and location of precipitation, groundwater contribution);
- Natural seasonal changes (most hydrological parameters change constantly throughout the year); and
- Climate change uncertainty.

Each of the above sources of uncertainty will compound the overall uncertainty of the model results. The exact uncertainty cannot be determined due to the wide variation of each of these sources. Hydrologic and hydraulic model assumptions and limitations are generally noted throughout the hydrologic and hydraulic model sections in this report.

3.5 Selection of Design Model Parameters

The model calibration results were then analysed to select design model parameters and initial conditions to be used for the modelling of a range of flood scenarios (described in **Chapter 4**). A comparison of the flows and water levels produced by the model using the adjusted model parameters for each season was therefore carried out based on the 1 in 100 year design rainfall event (described in **Chapter 4**).

3.5.1 Initial Conditions Selected for Each Season

The initial conditions and climate inputs used in the model for each season are presented in **Table 3.6.** Initial flow conditions used in the models were the average flow rates for the respective season estimated from the Environment Canada flow gauging data. Thus, the initial flow conditions were developed by inputting a 1 in 2 year storm event into the model and then running the model until flows reached the average flow rates at stations 01EJ001 and 01EJ004 after the storm event. The resulting flows output at every location in the model, when average flow rates were achieved, were then input back into the model as the initial flow conditions for simulating the 1 in 100 year storm.

For the winter and spring simulations, initial snow depth conditions were also input into the model based on an average of the climate normal values from the Shearwater Airport and Halifax Airport climate stations. Snowmelt calculations were performed in the model by inputting the average of the climate normal wind speeds, daily maximum temperature and daily minimum temperatures for the two climate stations. The temperatures were input such that the minimum temperature would occur at the beginning of the storm and maximum temperature would occur at the peak of the storm, allowing for a maximum snowmelt rate to occur during the peak of the storm.

Table 3.6: Initial Conditions and Design Climate Inputs for Seasonal Models

Season	Months	[1]Initial Flow (m^3/s)		[2]Initial Snow Depth (cm)	[2]Daily Max. Temperature (°C)	[2]Daily Min. Temperature (°C)	[2]Wind Speed (km/hr)
		01EJ001	01EJ004				
Spring	Mar/Apr/May	7.40	0.55	2.0	8.8	-0.2	16.9
Summer	Jun/Jul/Aug	2.21	0.19	0	21.9	12.5	12.9
Fall	Sep/Oct/Nov	4.43	0.41	[3]0.2	13.5	5.1	15.4
Winter	Dec/Jan/Feb	6.16	0.50	7.3	0.4	-8.2	18.0

[1]Initial flow conditions defined as the average flow rates for the respective season and flow gauging station.

[2]Initial snow depth, daily maximum temperature, daily minimum temperature and wind speed defined as the average of the climate normal values published for Shearwater Airport and Halifax Airport climate stations.

[3]Average initial snow depth for the fall season was considered to be minimal and was therefore not included in the model.

3.5.2 Seasonal Comparison Results

According to the simulation results for four seasonal models, water levels in the Sackville River were estimated to be highest using the winter characteristics, and water levels in the upper half of the Little Sackville River were estimated to be highest using the fall characteristics. Based on these findings, HRM selected the winter characteristics for the Sackville River and the fall characteristics for the Little Sackville River to be used as the design model parameters and initial conditions for the flood scenario modelling. Profiles comparing the peak water levels for the four seasons are presented in **Appendix D**.

3.6 Selection of Design Storm Durations

The duration of the design storms used for the Sackville River and the Little Sackville River were then selected based on a comparison of water levels estimated by 24-hour and 48-hour 1 in 100 year design storms. The comparison was conducted using the model characteristics selected for the design flood scenario modelling. Based on the model simulations results, water levels were approximately 0.17 m higher on average in the Sackville River and approximately 0.04 m higher on average in the Little Sackville River using the 48-hour storm compared to using the 24-hour storm. The higher impact on water levels resulting from the 48-hour storm for the Sackville River is likely attributed to the larger drainage area and lake coverage of the Sackville River watershed compared to the Little Sackville River watershed, which results in a longer time for flows to reach their peak and a higher influence of rainfall volumes on peak flows rates. Based on these findings, HRM selected a 48-hour duration for the Sackville River and a 24-hour duration for the Little Sackville River as the durations for the design storm events used for the flood scenario modelling.

3.7 Hydraulic Structure Head Losses

Head losses occurring at the hydraulic structures were analysed for the 1 in 100 year rainfall event (described in **Chapter 4**) to identify structures that could potentially cause flow restrictions during flood events. Head losses were defined as the difference between the peak water levels upstream and downstream of the respective structures. The estimated head losses for the 1 in 100 year rainfall event using existing IDF, sea level and development conditions is presented in **Table 3.7**. As shown in the table, the structures with the highest head losses (omitting structures that have significant drops in elevation at their outlets) were the Beaver Bank Cross Road Culvert, the Beaver Bank Road Culvert, the Sackville Drive Culvert and the Lucasville Road Bridge. According to the Phase I report (GHD 2016), a hydraulic constriction may be present near or upstream of the Little Sackville River hydrometric station (01EJ004), as the flows calculated by the flood frequency analysis for this station were found to be lower than those calculated by regional flood frequency analyses at this location from previous studies. However, while higher head losses were estimated at some of the structures upstream of the station (Beaver Bank Cross Road Culvert, Beaver Bank Road Culvert and Sackville Drive Culvert), the model estimated that minimal reduction in peak flow occurs through these structures during the 1 in 100 year rainfall event. Thus, while these structures may increase water levels, the model did not verify that they reduce peak flows.

Table 3.7: Head Losses at Hydraulic Structures for 1 in 100 Year Rainfall Event (Existing IDF & Sea Level, Existing Development)

Structure #	Structure Name	Head Loss (m)
1	Feely Lake Dam	[1]0.56
2	Barrett Culvert 1	0.62
3	Quarry Road Culvert	[1]0.72
4	Lumber Mill Pond Dam	[1]2.00
5	Barrett Culvert 2	[1]1.44
6	Barrett Culvert 3	0.50
7	Barrett Culvert 4	0.89
8	Millwood Drive Culvert	0.83
9	Trail Bridge #1	0.01
10	Trail Bridge #2	0
11	Beaver Bank Cross Road Culvert	1.38
12	Gantry Road Culvert	0.01
13	Beaver Bank Road Culvert	1.34
14	Old Beaver Bank Road Culvert	0.12
15	Sackville Drive Culvert	2.59
16	Sackville Cross Road Bridge	0.04
17	Highway 101 Bridge	0.14
18	Greenway Trail Bridge #1	0.07
19	Lucasville Road Bridge	2.45
20	Greenway Trail Bridge #2	0.06
21	Greenway Trail Bridge #3	0.01
22	Bedford Rifle Range Bridge	0
23	Highway 102 Bridge	0.11
24	Greenway Trail Bridge #4	0
25	River Lane Bridge	0.01
26	Bedford Place Mall Bridge #1	0.02
27	Bedford Place Mall Bridge #2	0
28	Bedford Highway Bridge	0.42
29	Railway Bridge	0.32
30	Shore Drive Bridge	0.45

[1]Head loss caused by elevation drop at structure outlet.

3.8 HEC-RAS Hydraulic Model Development

A simplified steady state HEC-RAS model of the Sackville River and the Little Sackville River was developed to carry out the ice jam and encroachment analyses. The calibrated PCSWMM hydraulic model was therefore converted to a steady state HEC-RAS model by converting and importing the cross sections and hydraulic structure characteristics from the PCSWMM model. Peak flows inputs for the HEC-RAS model were estimated from the PCSWMM model for the respective design storms, and downstream boundary conditions were set as fixed depths for the respective design sea levels. A three-dimensional view of the cross sections used for the HEC-RAS model is presented in **Figure 3.9** with an example flood simulation.

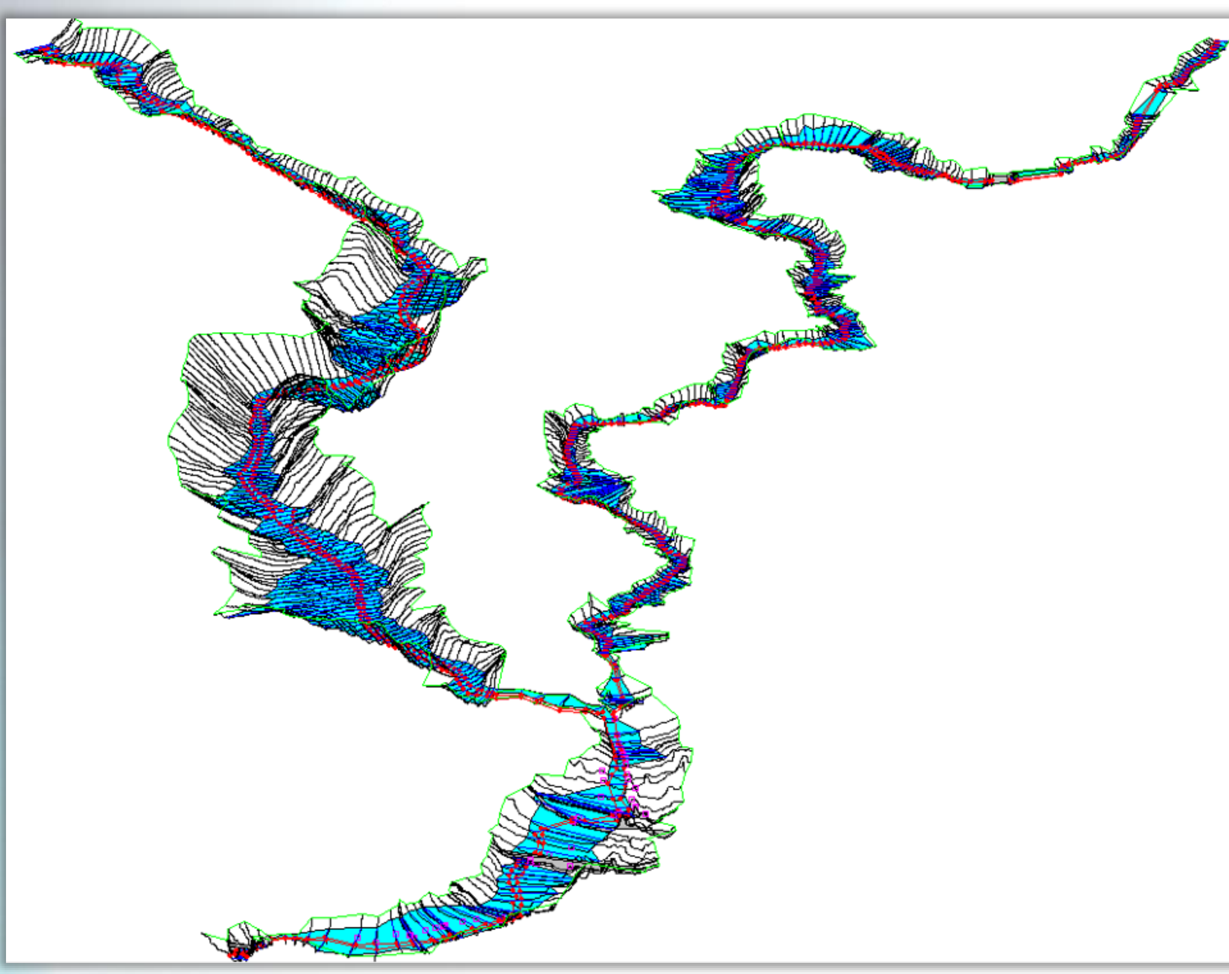


Figure 3.9: Three-Dimensional View of HEC-RAS Hydraulic Model

3.9 Hydrologic and Hydraulic Model Findings

The hydrologic model was calibrated on historic flow records. The results of the model are therefore consistent with the historical peak flows (e.g. the 1 in 100 year peak flow is calculated in the model to be a seasonal average of 38.5 m^3/s in the Little Sackville River, which compares to 26.3 m^3/s estimated from the flow gauging data frequency analysis, and 109.75 m^3/s in the Sackville River, which compares to 115 m^3/s estimated from the flow gauging data frequency analysis). Compared to the historical storm of

March 2003, the water level results are slightly higher throughout the river system, which is consistent with the finding that the March 2003 event was less significant than a 1 in 100 year event.

Other findings from this analysis include the identification of factors that lead to the flooding extents generated by the models. The analysis of structure constrictions only identified four structures that create notable impediments to the passage of water. Those structures are the Beaver Bank Cross Road, Beaver Bank Road and Sackville Drive structures along the Little Sackville River and the Lucasville Road structure along the Sackville River. Other than those structures, there are few anthropogenic impacts to the natural shape of the river channel, other than river diversions to circumvent development. This is a notable finding, because it demonstrates that flooding outside of the river channel (i.e. in the floodplain) is a natural phenomenon. Natural rivers create over time a natural channel whose size is reflective of average river flows. Flows above average values carve a natural floodplain in the landscape. The majority of floodplain extents in Nova Scotia rivers were created during the melting of the last ice age glaciers, approximately 10,000 years ago. These are natural floodplains, which rivers occupy in higher than average flows. The model results show that the current 1 in 100 year peak flood extents occupy a large portion of this natural "ice melt" floodplain. Notably, the model results also indicate that events of a greater magnitude, including the 1 in 500 year event, the PMP or future events influenced by development and climate change lead to increased floodplain width (as expected), but only by a small relative amount. This means that high flows will regularly fill the floodplain, but that extremely high flows will still stay within this main floodplain. It is important to note this because it means that the floodplain is necessary for the conveyance of high flows. Development within the floodplain will unavoidably be at risk of flooding, and any restriction of this floodplain will lead to higher upstream water levels. Notable development in the floodplain includes the road crossings noted above, the Downsview Mall, the development around Sackville Cross Road, the Contessa Ct. and Sami Dr. residential development, the Bedford Place Mall and adjacent residential development. The most notable infrastructure that alters the floodplain is the Highway 101 crossing and its interchange with Highway 102. All the above areas are at risk of flooding because they lie within the natural floodplain. Their impacts on flood levels seem to be limited, as seen from the water surface profiles in **Appendix D (Profile #7 to #9)**, but this has not been confirmed by modelling a scenario where this development does not exist.

The assessment of seasonal effects on flood risks also yielded interesting results. The Little Sackville River, being more urbanised, did not show notable seasonal variations in flood elevations (see **Appendix D, Profile #1)**. However, the Sackville River showed high sensitivity to seasonal changes, with close to a metre of difference in water levels, downstream of its confluence with the Little Sackville River. Development projections showed little influence, with an increase in the order of 100 mm at the downstream end of the Sackville River.

CHAPTER 4 FLOOD SCENARIO MODELLING, FLOOD LINE DELINEATION & FLOOD MITIGATION

4.1 Summary of Flood Scenarios Modelling & Results

The calibrated hydrologic and hydraulic models were used to simulate the various flood scenarios presented in **Table 4.1**. As shown in the table, the flood scenarios included variations in seasonal conditions, climate conditions, sea level conditions, development conditions and ice conditions for various rainfall events and sea level events. All flood scenarios were simulated using the PCSWMM model with the exception of the ice jam flood, which was simulated using HEC-RAS. The model simulation results were then used to produce water elevation profiles that show estimated water levels along the river channels and to delineate flood lines that show estimated flood extents overlaid on community and orthophotography maps. The resulting water elevation profiles are presented in **Appendix D**, and the flood line delineations are presented in **Appendix E**.

The process of delineating flood lines involved converting the model results to horizontal floodplain extents. This was achieved by creating a three-dimensional surface that connected the individual cross-sectional water level output values. The three-dimensional surface was then intersected with the high resolution lidar topographical surface to produce a flood line delineation. This process was followed for each of the scenarios modelled.

Table 4.1: Summary of Flood Scenarios, Model Simulations, Water Surface Profiles and Flood Line Delineations

Scenario Description	Model Parameters & Initial Conditions	[1,2]Design Rainfall Event	Peak Flow (m^3/s) 01EJ001	Peak Flow (m^3/s) 01EJ004	Design Sea Level Event	Total Sea Level [geodetic] (m)	[2]Water Surface Profiles	[2]Flood Maps
Seasonal Comparison (Existing IDF & Sea Level)	Spring	1 in 100 Year	89	40	1 in 2 Year	1.79	• 1 in 100 Year Spring/Summer/Fall/Winter Rainfall	• **Map 1**: 1 in 100 Year Spring/Summer/Fall/Winter
	Summer		76	31				
	Fall		135	43				
	Winter		139	40				
Historical Design Storm	March 2003	March 2003	151	32	March 2003	1.01	• March 2003 Rainfall	• **Map 2:** March 2003 Rainfall
Existing IDF & Sea Level, Existing Development	Fall (LSR) & Winter (SR), Existing Development	1 in 5 Year	92	26	1 in 2 Year	1.79	• 1 in 5/20/100/500 Year Rainfall & PMP • 1 in 5/20/100/500 Year Sea Level	• **Map 3:** 1 in 5/20/100/500 Year & PMP
		1 in 20 Year	112	34				
		1 in 100 Year	142	42				
		1 in 500 Year	184	57				
		PMP	457	254				
		1 in 2 Year	79	20	1 in 5 Year	1.93		
					1 in 20 Year	2.10		
					1 in 100 Year	2.29		
					1 in 500 Year	2.47		
Existing IDF & Sea Level, Future Development	Fall (LSR) & Winter (SR), Future Development	1 in 5 Year	96	26	1 in 2 Year	1.79	• 1 in 5/20/100 Year Rainfall (Future Development)	• **Map 4:** 1 in 5/20/100 Year (Future Development)
		1 in 20 Year	118	34				
		1 in 100 Year	152	43				
		1 in 2 Year	80	20	1 in 5 Year	1.93		
					1 in 20 Year	2.10		
					1 in 100 Year	2.29		
Future IDF & Sea Level, Existing Development	Fall (LSR) & Winter (SR), Existing Development	1 in 5 Year (2100) - IDF-CC Tool Median	97	28	1 in 2 Year (2100)	2.90	• 1 in 5/20/100 Year Rainfall (2100) – IDF-CC Tool Median • 1 in 5/20/100 Year Rainfall (2100) – IDF-CC Tool Upper Bound • 1 in 5/20/100 Year Rainfall (2100) – Clausius-Claperyon Upper Bound • 1 in 5/20/100 Year Sea Level (2100)	• **Map 5**: 1 in 5/20/100 Year (2100) – IDF-CC Tool Median • **Map 6**: 1 in 5/20/100 Year (2100) – IDF-CC Tool Upper Bound • **Map 7**: 1 in 5/20/100 Year (2100) – Clausius-Claperyon Upper Bound
		1 in 20 Year (2100) - IDF-CC Tool Median	124	37				
		1 in 100 Year (2100) - IDF-CC Tool Median	167	49				
		1 in 2 Year (2100) - IDF-CC Tool Median	80	21	1 in 5 Year (2100)	3.04		
					1 in 20 Year (2100)	3.21		
					1 in 100 Year (2100)	3.40		
		1 in 5 Year (2100) - IDF-CC Tool Upper Bound	125	38	1 in 2 Year (2100)	2.90		
		1 in 20 Year (2100) - IDF-CC Tool Upper Bound	202	68				
		1 in 100 Year (2100) - IDF-CC Tool Upper Bound	305	117				
		1 in 2 Year (2100) - IDF-CC Tool Upper Bound	90	25	1 in 5 Year (2100)	3.04		
					1 in 20 Year (2100)	3.21		
					1 in 100 Year (2100)	3.40		
		1 in 5 Year (2100) - Clausius-Claperyon Upper Bound	151	44	1 in 2 Year (2100)	2.90		
		1 in 20 Year (2100) - Clausius-Claperyon Upper Bound	221	76				
		1 in 100 Year (2100) - Clausius-Claperyon Upper Bound	302	116				
		1 in 2 Year (2100) - Clausius-Claperyon Upper Bound	115	35	1 in 5 Year (2100)	3.04		
					1 in 20 Year (2100)	3.21		
					1 in 100 Year (2100)	3.40		
Future IDF & Sea Level, Future Development	Fall (LSR) & Winter (SR), Future Development	1 in 5 Year (2100) - IDF-CC Tool Median	101	29	1 in 2 Year (2100)	2.90	• 1 in 5/20/100 Year Rainfall (2100, Future Development) – IDF-CC Tool Median • 1 in 5/20/100 Year Rainfall (2100, Future Development) – IDF-CC Tool Upper Bound • 1 in 5/20/100 Year Rainfall (2100, Future Development) – Clausius-Claperyon Upper Bound	• **Map 8**: 1 in 5/20/100 Year (2100, Future Development) – IDF-CC Tool Median • **Map 9[3]: 1 in 5/20/100 Year (2100, Future Development) – IDF-CC Tool Upper Bound Result** • **Map 10**: 1 in 5/20/100 Year (2100, Future Development) – Clausius-Claperyon Upper Bound
		1 in 20 Year (2100) - IDF-CC Tool Median	130	38				
		1 in 100 Year (2100) - IDF-CC Tool Median	179	49				
		1 in 2 Year (2100) - IDF-CC Tool Median	83	21	1 in 5 Year (2100)	3.04		
					1 in 20 Year (2100)	3.21		
					1 in 100 Year (2100)	3.40		
		1 in 5 Year (2100) - IDF-CC Tool Upper Bound	132	38	1 in 2 Year (2100)	2.90		
		1 in 20 Year (2100) - IDF-CC Tool Upper Bound	215	67				
		1 in 100 Year (2100) - IDF-CC Tool Upper Bound	321	115				
		1 in 2 Year (2100) - IDF-CC Tool Upper Bound	83	25	1 in 5 Year (2100)	3.04		
					1 in 20 Year (2100)	3.21		
					1 in 100 Year (2100)	3.40		
		1 in 5 Year (2100) - Clausius-Claperyon Upper Bound	163	45	1 in 2 Year (2100)	2.90		
		1 in 20 Year (2100) - Clausius-Claperyon Upper Bound	234	76				
		1 in 100 Year (2100) - Clausius-Claperyon Upper Bound	318	114				
		1 in 2 Year (2100) - Clausius-Claperyon Upper Bound	120	35	1 in 5 Year (2100)	3.04		
					1 in 20 Year (2100)	3.21		
					1 in 100 Year (2100)	3.40		
Ice Jam Flood	1 in 100 Year Ice Jam	1 in 2 Year	79	20	1 in 2 Year	1.79	• 1 in 100 Year Ice Jam Flood	• **Map 11**: 1 in 100 Year Ice Jam Flood
Comparison of Existing and Future IDF & Sea Level Flood Lines	-	-	-	-	-	-	-	• **Map 12**: 1 in 100 Year (Existing, IDF-CC Tool Median, IDF-CC Tool Upper Bound, Clausius-Claperyon Upper Bound)
Comparison of Previous Flood Line Delineations	-	-	-	-	-	-	-	• **Map 13**: 1 in 20 Year (Current, 1980s, 1990s) • **Map 14**: 1 in 100 Year (Current, 1980s, 1990s)
Phase I River Flow Frequency Analysis Results	Phase I Flows	-	62.1	17.5	-	-	• 1 in 5/20/100 Year Phase I River Flow Frequency Analysis Results	• **Map 15**: 1 in 5/20/100 Year Phase I River Flow Frequency Analysis Results
			78.7	21.7				
			97.4	26.3				

[1]Rainfall total amounts in mm are presented in Table 4.2 below. [2]IDF-CC Tool abbreviated from "Western University Intensity-Duration-Frequency Climate Change Tool". [3] This is the recommended Base Flood Map

4.2 Design Rainfall, Sea Level & Ice Accumulation Events

This section describes how the various design rainfall, sea level and ice accumulation events presented in **Table 4.1** were developed for the various flood scenarios model simulations.

4.2.1 Design Rainfall Events

Design rainfall hyetographs that follow the Chicago Distribution with 5-minute discretization intervals were developed for 24-hour duration and 48-hour duration storm events. The design hyetographs used for flood scenarios included the 1 in 2, 5, 20, 100 and 500 year storms, the PMP and the 1 in 2, 5, 20 and 100 year storms under climate change conditions for the year 2100. A summary of all design rainfall events and their associated total rainfall amounts is provided in **Table 2.3**.

Table 4.2: Summary of Design Rainfall Events and Associated Total Rainfall Amounts

Return Period	Total Rainfall Amount (mm)							
	Existing Climate Conditions (Year 2016)		[1]Climate Change Conditions (Year 2100)					
			Western University IDF Climate Change Tool (Median Result)		Western University IDF Climate Change Tool (Upper Bound Result)		Clausius-Clapeyron Equation (Upper Bound Result)	
	24 hr	48 hr	24 hr	48 hr	24 hr	48 hr	24 hr	48 hr
1 in 2 Year	81	110	85	116	100	135	136	185
1 in 5 Year	103	141	112	153	149	203	175	238
1 in 20 Year	133	182	147	200	212	289	226	307
1 in 100 Year	167	227	186	254	284	387	282	385
1 in 500 Year	199	271	-		-		-	-
PMP	483	483	-		-		-	-

[1]Climate change conditions based on Representative Concentration Pathway (RCP) 8.5 from the Intergovernmental Panel on Climate Change (IPCC) Fifth Assessment Report (2013).

4.2.1.1 Design Storm Events for Existing Climate Conditions

The design storm events for existing climate conditions were based on Intensity-Duration-Frequency (IDF) curves for the 1 in 2, 5, 20, 100 and 500 year storms, which were developed for this study following the same procedures used by Environment Canada. The IDF curves were therefore produced by fitting a Gumbel statistical distribution for the Shearwater Airport annual maximum rainfall amounts and then fitting an exponential curve for each return period. It is noted that since the 1 in 500 year return period is calculated using 53 year of data, there is some uncertainty with this result. The upper and lower 95% confidence limits for this estimate are calculated at 200.5mm and 165.6mm respectively. The resultant IDF Curves are presented in **Figure 4.1**. It is noted that the upper bound result is close to the 1 in 500 year value from the extrapolation of the IDF curve. The reason for this is that the IDF curve (as seen below) is an interpolated estimate or the results of the individual statistical analyses for the various storm event durations. This interpolation does not fall exactly at the values of the statistical analysis (i.e. the lines on the graphs do not follow exactly the crosses), but in the case of the 1 in 500 year event, the value falls within the 95% confidence limits.

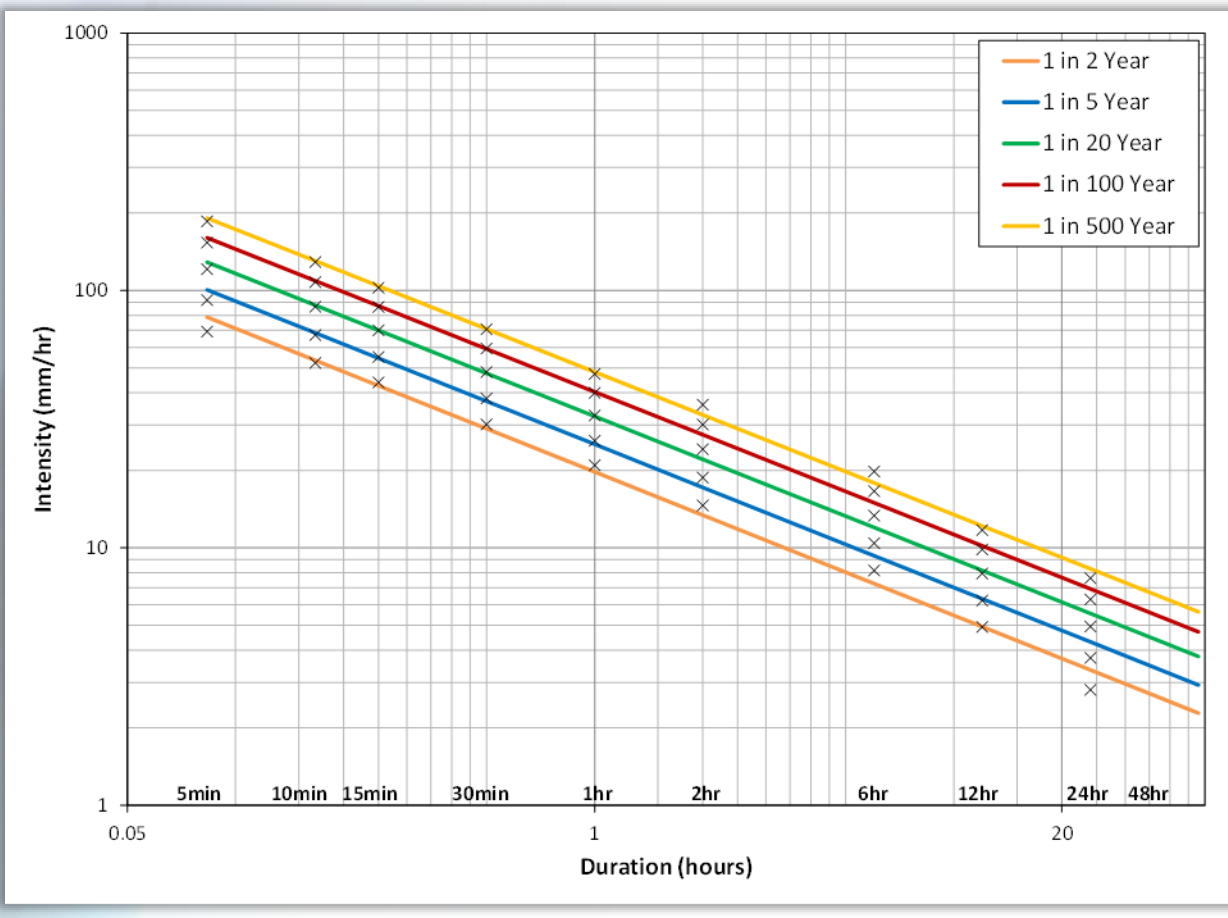


Figure 4.1: Intensity-Duration-Frequency (IDF) Curves Developed for the Shearwater Airport Climate Station

It should be noted that annual maximum rainfall amounts for the Shearwater Airport climate station were selected over those for the Halifax Airport climate station due to the Shearwater Airport having significantly more data available for statistical analysis (53 years of data at Shearwater Airport versus 18 years of data at Halifax Airport). It should also be noted that the reason why the official IDF curves published by Environment Canada were not used for the design rainfall events was because Environment Canada does not provide IDF curves for the 1 in 20 year and 1 in 500 year return periods. However, the 1 in 2, 5 and 100 year IDF curves published by Environment Canada exactly matched those developed for this study.

Chicago Distribution design rainfall hyetographs were then produced from the IDF curves for 24-hour and 48-hour duration storm events, and are presented in **Figure 4.2**. Environment Canada conducts statistical analyses based on a number of specific storm durations: 5 minutes, 15 minutes, etc up to 24 hours. It is assumed that the exponential relationship between rainfall duration and average rainfall intensity is maintained up to the 48 hour rainfall durations, and that therefore the equations proposed are valid to estimate 48 hour average rainfall intensities. Although there is some uncertainty associated with this approach, a more complete analysis would be a significant undertaking, which is not within the scope of this assessment and which is not expected to produce significant differences.

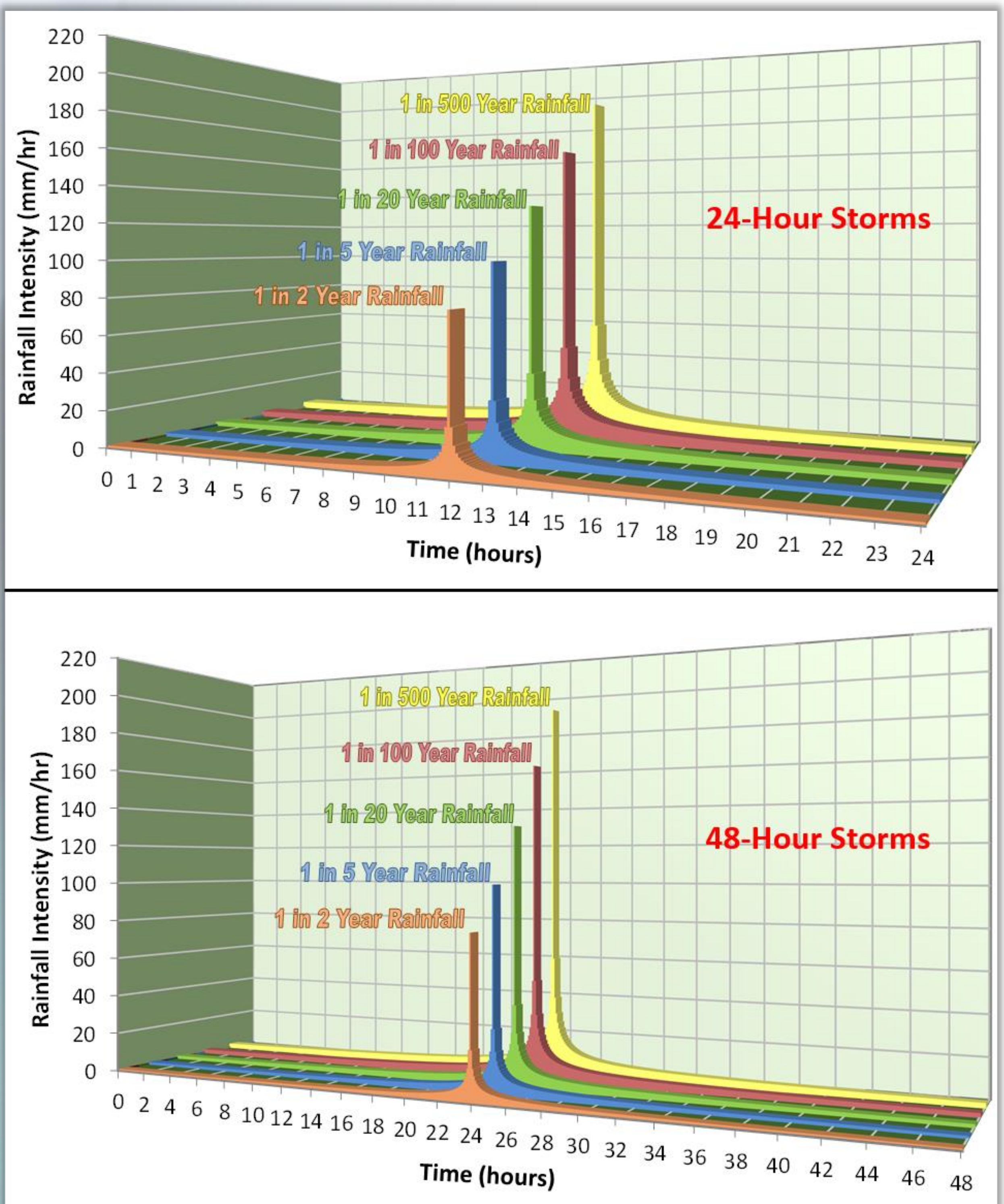


Figure 4.2: Chicago Distribution Design Rainfall Hyetographs for 24-Hour Duration (above) and 48-Hour Duration Storm Events

4.2.1.2 Design Storm Events for the Probable Maximum Precipitation (PMP)

The PMP is the theoretical maximum precipitation for a given duration under modern meteorological conditions (Hogg and Carr 1985). Several in-depth theoretical approaches are available for estimating the PMP; however, there is still a wide disparity between theoretically estimated amounts and actual observations, despite considerable investigation into the mechanisms of rainfall production over the last few decades (WMO 2009). As noted in the document, procedures for estimating PMP cannot be standardized. They vary with the amount and quality of data available, basin size and location, basin and regional topography, storm types producing extreme precipitation, and climate. It should be noted that due to the physical complexity of the phenomena and limitations in data and the meteorological and hydrological sciences, only approximations are currently available for the upper limits of storms and their associated floods. The accuracy of PMP/PMF estimation rests on the quantity and quality of data on extraordinary storms and floods and the depth of analysis and study. Nonetheless, it is impossible to give precise values for PMP and PMF. As yet, there are no methods to quantitatively assess the accuracy of PMP and PMF.

Statistical procedures for the PMP are considered the most appropriate method for small basins (1000 m^2), although they have also been used for much larger areas (Hogg and Carr 1985). Statistical approaches are particularly useful where other meteorological data, such as dew point and wind records, are lacking (WHO 2009). The approach used for estimating the PMP for this study used the empirical relationships developed by Hershfield (1965), which are based on several hundred thousand station-years of data from many countries. While the Hershfield procedure is not the only statistical approach available, it is the process that has received the widest acceptance (WHO 2009). However, it is noted that this approach assumes that the PMP has been historically observed at the station, and that it only estimates point values. The PMP was therefore calculated using the following equation provided by Hershfield (1965):

$$PMP = \bar{X}_n + K_{M24} S_n$$

Where:

$\bar{X}_n = mean\ annual\ maximum\ 24\ hour\ rainfall\ amount = 71.6\ mm$

$S_n = standard\ deviation\ of\ annual\ maximum\ 24\ hour\ rainfall\ amounts = 25.4\ mm$

$K_{M24} = 19(10)^{-0.000965\bar{X}_n}$

Using the annual maximum rainfall amounts published by Environment Canada for the Shearwater Airport climate station, the mean and standard deviation were found to be 71.6 mm and 25.4 mm, respectively. Thus, the PMP was estimated to be 483 mm using the above equation. Design hyetographs for the PMP were then developed for 24-hr and 48-hr durations following the Chicago distribution.

4.2.1.3 Design Storm Events for Climate Change Conditions

Climate change (and particularly the increase in precipitation intensity) is one of several uncertain factors (e.g., development, population growth, infrastructure performance, etc.) expected to impact future flooding vulnerabilities. It is therefore important to consider and compare different approaches to assessing the impacts of climate change on projected rainfall (within the context of their assumptions) and then use the resulting range in projections to test the sensitivity of the hydrological system. For this study, the future changes in extreme rainfall amounts were evaluated using the Western University Intensity-Duration-Frequency Climate Change (IDF-CC) Tool, which downscales the results of 22 Global Climate Change Models (GCMs), and the Clausius-Clapeyron Equation applied to temperature projections from the UPEI climate change tool. All climate change projections used for this analysis are based on Representative Concentration Pathways (RCPs) from the Intergovernmental Panel on Climate Change (IPCC) Fifth Assessment Report (AR5, IPCC 2013).

Western University Intensity-Duration-Frequency Climate Change (IDF-CC) Tool

A tool that has emerged at the forefront of recent discussions in future rainfall intensities is the Western University IDF-CC Tool. This tool develops IDF curves based on various GCM projections of changes in daily rainfall, assuming that the relationship between sub-daily and daily precipitation will be unchanged in the future (Srivastav *et al*. 2014). The tool can be used to obtain a range of possible future IDF curves by varying settings such as GCMs used, the time period of forecast over which the projection is done, and the emission scenario considered.

There is emerging evidence, however, showing that sub-daily precipitation is more sensitive to temperature changes than daily precipitation, as storms of different durations are controlled by different atmospheric mechanisms (see PCIP 2015). Therefore, the assumption of stationarity on which the tool is based is likely inaccurate, and the tool's results must be interpreted within this limitation.

Clausius-Clapeyron Equation

Another approach to estimating future changes in the intensity of extreme rainfall is based on the relationship of rainfall to atmospheric temperature, because warmer air is capable of holding more water than cooler air. The capacity of the atmosphere to hold water is governed by the Clausius-Clapeyron equation, which can be approximated as an increase in precipitation intensity by 6% to 7% per degree Celsius. Temperature projections were made using the UPEI climate change tool. The tool was originally created by Environment Canada and has been redeveloped and upgraded by the Climate Lab at UPEI, managed by Associate Professor Adam Fenech, PhD.

It is noted that the use of the theoretical scaling rates assumes that relative humidity remains constant (i.e., there is sufficient water availability) and that there are no changes to the atmospheric circulation patterns that produce rainfall. The actual rates vary with latitude and altitude as well as seasonal temperature (see Westra *et al*. 2014 for a list of the regions where the theoretical scaling rate has been observed). Lastly, storms of different durations scale with temperature at different rates because they are governed by different atmospheric mechanisms (PCIP 2015). Although a number of complicating factors affect the Clausius-Clapeyron empirical relationship, these equations can be used to obtain a "general tendency" of changes in precipitation intensity associated with changes in temperature (Westra *et al*. 2014).

Estimated Range of Possible Outcomes

Estimated 1 in 2, 5, 20 and 100 year future rainfall amounts were then obtained for Shearwater Airport, for three climate normal periods (2010-2039, 2040-2069 and 2070-2099) and for two RCPs (RCP 4.5 and RCP 8.5) using the Western University IDF-CC Tool and the Clausius-Clapeyron Equation. The IDF-CC Tool was used to obtain downscaled total precipitation amounts from 22 GCMs, and the Clausius-Clapeyron equation was used to estimate precipitation amounts based on temperature projections obtained using the University of Prince Edward Island Climate Change Tool. These temperature projections are an ensemble of results from all available global climate models for each climate normal period and RCP. The Clausius-Clapeyron scaling rate was applied to the temperature projections.

For the 1 in 100 year storm, total precipitation ranges for the 2070-2099 period for both RCP 4.5 and RCP 8.5 estimated using the IDF-CC tool and the Clausius-Clapeyron equation were 134 mm to 284 mm and 175 mm to 250 mm, respectively. **Figure 4.3** shows these results as a histogram for the 2070-2099 period as grouped bins of 10 mm. In the figure, the x-axis shows the number of GCMs from the IDF-CC Tool that predict a total precipitation for 2070-2099 within a given bin.

It should be noted that there are significant concerns associated with multi-decadal GCM projections, as they simulate what "might happen under some conditions" and they cannot be interpreted as predictions (Brown and Wilby 2012). Uncertainty is compounded when GCM projections are downscaled. Hence, the results presented here should be seen as a subset of all possible futures. They are best used as a range of inputs to the hydrologic models, producing multiple flood lines and a better understanding of the sensitivity of the system (how much it would respond to possible changes in climate). Hence, these downscaling results should inform, and not drive, the selection of an appropriate design storm for flood mapping of the Sackville River and the Little Sackville River.

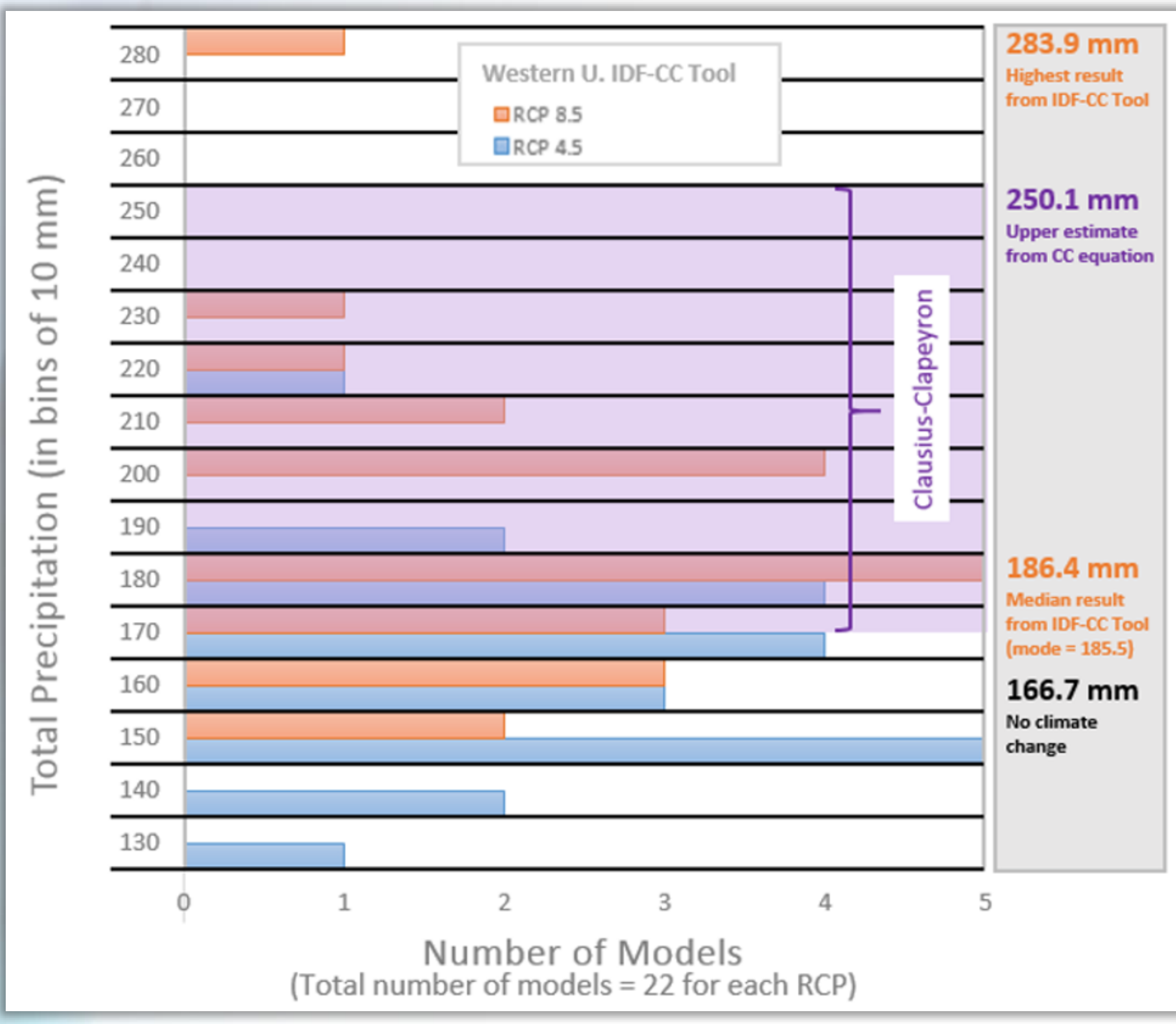


Figure 4.3: Estimated Future Total Precipitation Amounts for the 1 in 100 Year Storm for the 2070-2099 period at Shearwater Airport

Ultimately, predictions for the 2079-2099 period and for RCP 8.5 were selected for this study because they were generally the most conservative. Additionally, recent emissions were estimated to track more closely to RCP 8.5 (Zhai *et al*., 2014). The RCP 8.5 2079-2099 predictions were therefore compared for the 1 in 2, 5, 20 and 100 year storms, and HRM selected the following three climate change scenarios to be analysed for this study based on the results:

1. Western University IDF-CC Tool Median Result
2. Western University IDF-CC Tool Upper Bound Result
3. Clausius-Clapeyron Equation Upper Bound Result

All future climate change rainfall amounts estimated for the 1 in 2, 5, 20 and 100 year storm for the three selected climate change scenarios are presented in **Table 4.2**. For the purposes of this report, these rainfall estimates for the 2079-2099 period were assumed to be the same as for the year 2100.

The design storm hyetographs for future climate change conditions were then developed by scaling the existing 24-hr and 48-hr hyetographs to the estimated climate change rainfall amounts.

4.2.2 Design Sea Level Events

Design sea level events, which are defined in this study as extreme coastal water level occurrences that are caused by a combination of astronomical tide cycles, storm surge events and seiching (oscillations in a partially closed body of water), were developed for the flood scenarios for the 1 in 2, 5, 20, 100 and 500 year return periods. Design sea levels were estimated for both existing sea level conditions (year 2016) and for future sea level rise conditions (year 2100). A summary of the estimated design sea levels is presented in **Table 2.4** in both chart datum and Canadian Geodetic Vertical Datum 1928 (CGVD28). According to the Canadian Hydrographic Service, the difference between chart datum and Canadian Geodetic Vertical Datum 1928 for the Bedford Basin is approximately 0.8 m.

Table 4.3: Design Sea Levels

Return Period	Design Sea Level (m)			
	Existing Sea Level Conditions (Year 2016)		Future Sea Level Rise Conditions (Year 2100)	
	Chart Datum	[1]Geodetic Datum	Chart Datum	[1]Geodetic Datum
1 in 2 Year	2.59	1.79	3.70	2.90
1 in 5 Year	2.73	1.93	3.84	3.04
1 in 20 Year	2.90	2.10	4.01	3.21
1 in 100 Year	3.09	2.29	4.20	3.40
1 in 500 Year	3.27	2.47	4.38	3.58

[1]Canadian Geodetic Vertical Datum 1928 (CGV28).

4.2.2.1 Design Sea Level Events for Existing Sea Level Conditions

The design sea levels used for existing sea level conditions were estimated based on a statistical analysis of the historical hourly and sub-hourly tide gauging data in the Bedford Basin from 1919 to 2016 (see **Figure 4.4**). The historical gauged data therefore includes all the applicable water level mechanisms, such as the effects of tides, storm surges and seiching since they are measured directly in the Bedford Basin. The statistical analysis was carried out by first de-trending the tide gauging data to remove gradual increases in mean sea level resulting from sea level rise, which can be observed in the tide gauging data presented in **Figure 4.5**. The de-trended data was then shifted such that its mean sea level was at the 1.0 m chart datum, which is the existing mean sea level published by the Department of Fisheries and Oceans Canada (DFO) Canadian Hydrographic Service (CHS) for the Bedford Basin. Annual maximum sea levels were then calculated from the shifted dataset and were fitted to several statistical distributions (Normal, Log-Normal, Three-Parameter-Log-Normal, Gumbel, Fréchet, Weibull and Log-Pearson III). The most representative distribution was then selected using statistical hypothesis testing (Chi-square test, T-test, correlation, and coefficient of determination). Based on the statistical hypothesis test results, the Weibull distribution was selected as the most representative distribution.

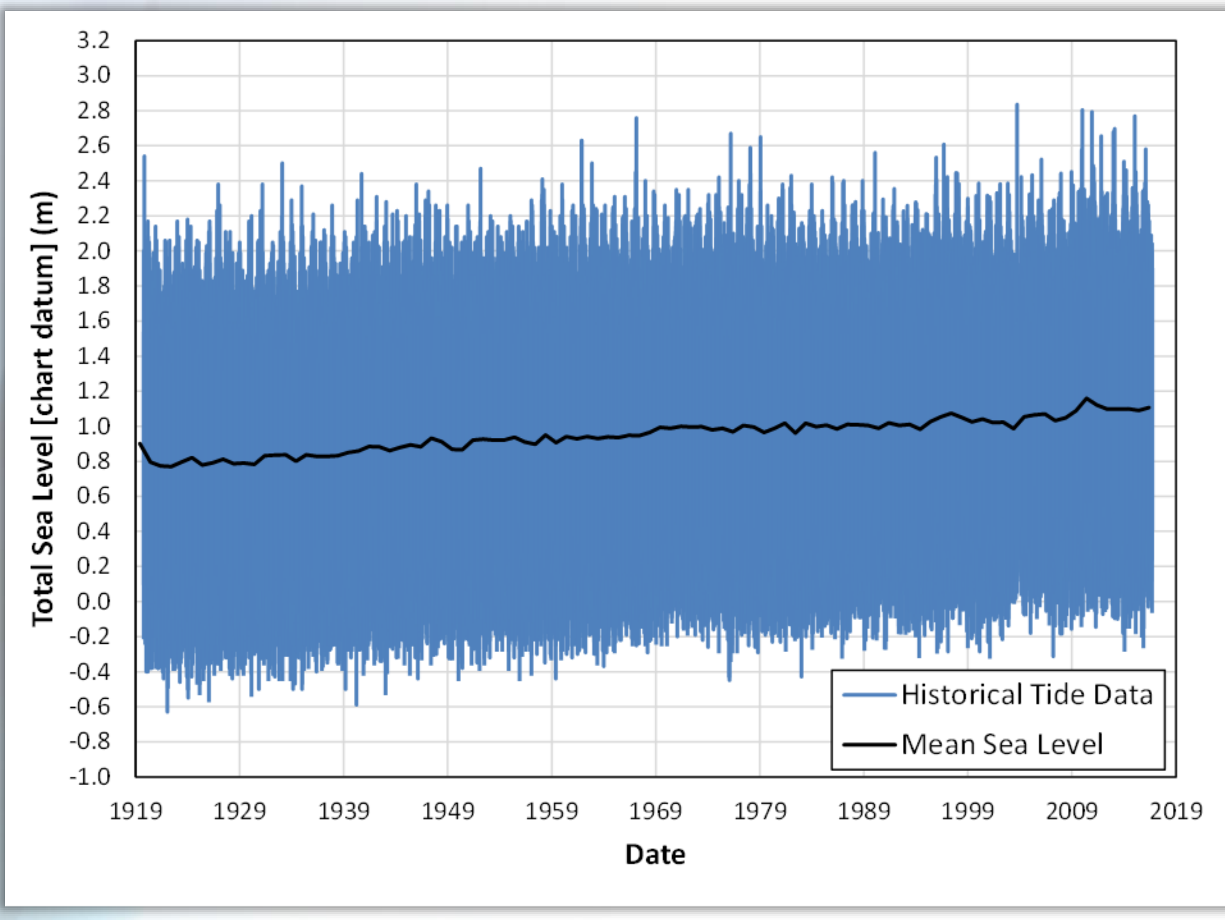


Figure 4.4: Historical Tide Gauging Data for the Bedford Basin Published by Canadian Hydrographic Services (CHS) in Chart Datum

The 1 in 2, 5, 20, 100 and 500 year design sea levels were then estimated using the Weibull distribution and by adding an additional 0.1 m to account for short term variations that occur in mean sea level. Finally, design sea level time series events were developed using 12-hour period sinusoidal curves that peak at the design sea level values, and are presented in **Figure 4.5**. The Lower Low Water Large Tide (LLWTL) value published by CHS for the Bedford Basin of 0.8 m geodetic was used as the minimum peak for all events.

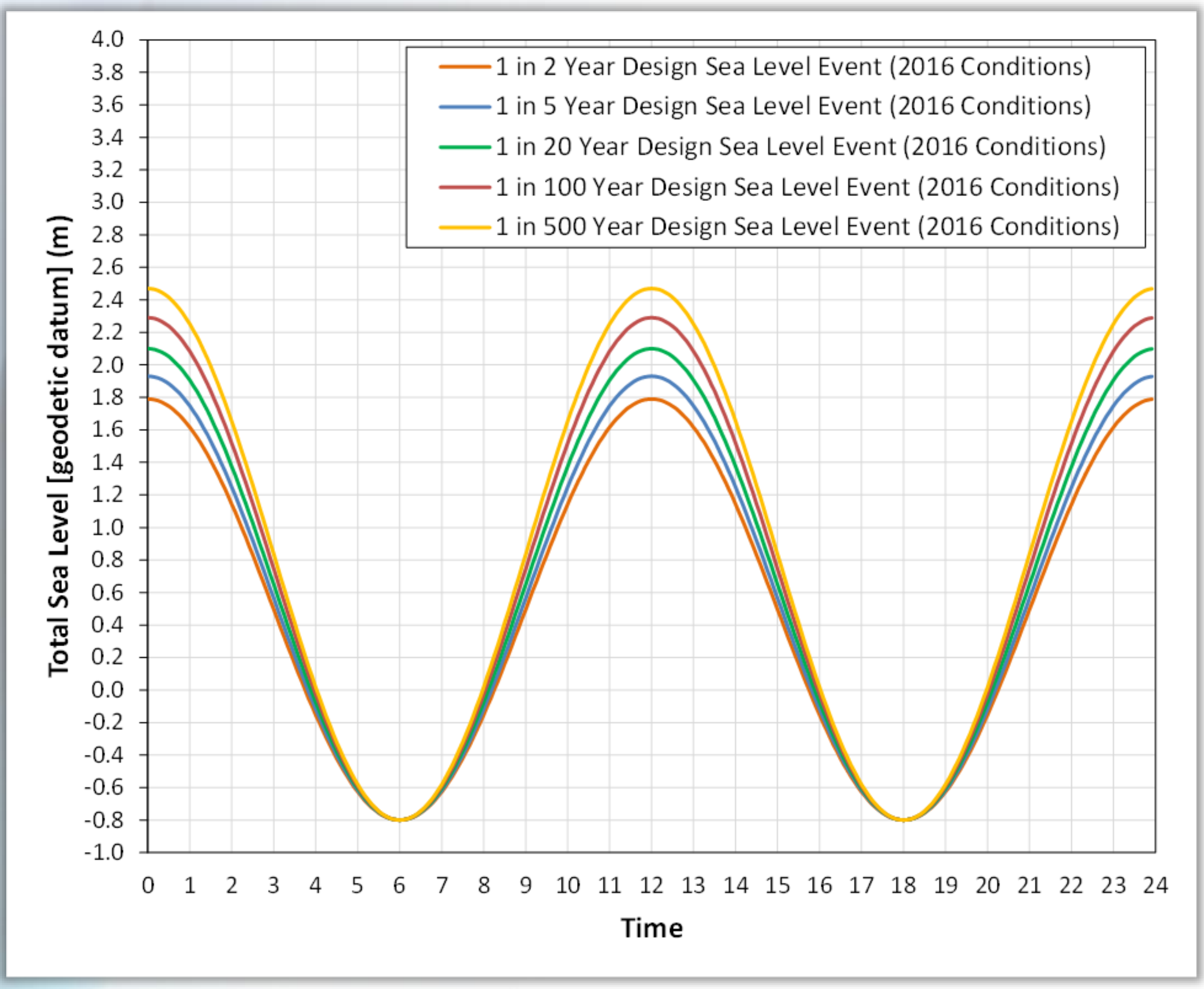


Figure 4.5: Design Sea Level Events for Existing Conditions (Year 2016)

4.2.2.2 Design Sea Level Events for Future Sea Level Rise Conditions

Sea level rise along eastern Canada's coast has been occurring since the end of the last ice age, about 10,000 years ago, when PEI was still linked to the mainland of Nova Scotia and New Brunswick. The rate of global mean sea level is accelerating in the 21st century due to global warming impacts and the melting of polar ice caps. At the time of the latest Intergovernmental Panel on Climate Change report (IPCC AR5, 2013), the *likely* range of global mean sea level rise for 2081-2100 relative to 1986-2005 was estimated at 0.98 m for the higher bound estimate for high emission scenario[1].

DFO's Canadian Technical Report of Hydrography and Ocean Sciences 300 (Zhai *et al.*, 2014) presents estimates of sea level rise allowances at different sites along the coasts of Canada. Allowances are

[1]There is currently insufficient evidence to evaluate the probability of specific levels above the assessed *likely* range. However, several subsequent studies indicate that the plausible upper bound level may exceed 2 m due to accelerated ice melting in Greenland and Antarctica (NOAA 2017).

estimated changes in the elevation of a site required to maintain the same frequency of inundation that the site has experienced historically. The allowances are determined based on RCP 8.5 from the latest projections of regional sea level rise from the IPCC Fifth Assessment Report (IPCC 2013).

Coastal flooding probabilities will increase with sea level rise. Therefore, deriving design values for coastal flooding could be based on 2 approaches:

1. **Probabilistic approach**, i.e. estimating the cumulative probability of occurrence of a flood event increasing with sea level rise over the lifetime of the project, or
2. **Deterministic approach**, i.e. using the N-year return storm plus a sea level rise allowance that would occur typically at the end of the project lifetime, which is more conservative than (1) above.

With the probabilistic approach, the cumulative probability of a flooding event was calculated for a range of elevations considering the individual probabilities of extreme storm surge elevations, which increase every year into the future due to sea level rise. The results are presented on **Figure 4.6**. This probabilistic approach should be used when determining risks to given infrastructure over a given lifetime into the next few decades. For example, it can show the probability of flooding reaching a certain design elevation by a certain timeline, at which point maintenance of a given structure would be required. The probabilistic approach is particularly suited when budgeting maintenance over a structure's lifecycle. The objective is somewhat different for flood mapping, where the focus is on planning for the worst case for the very long term.

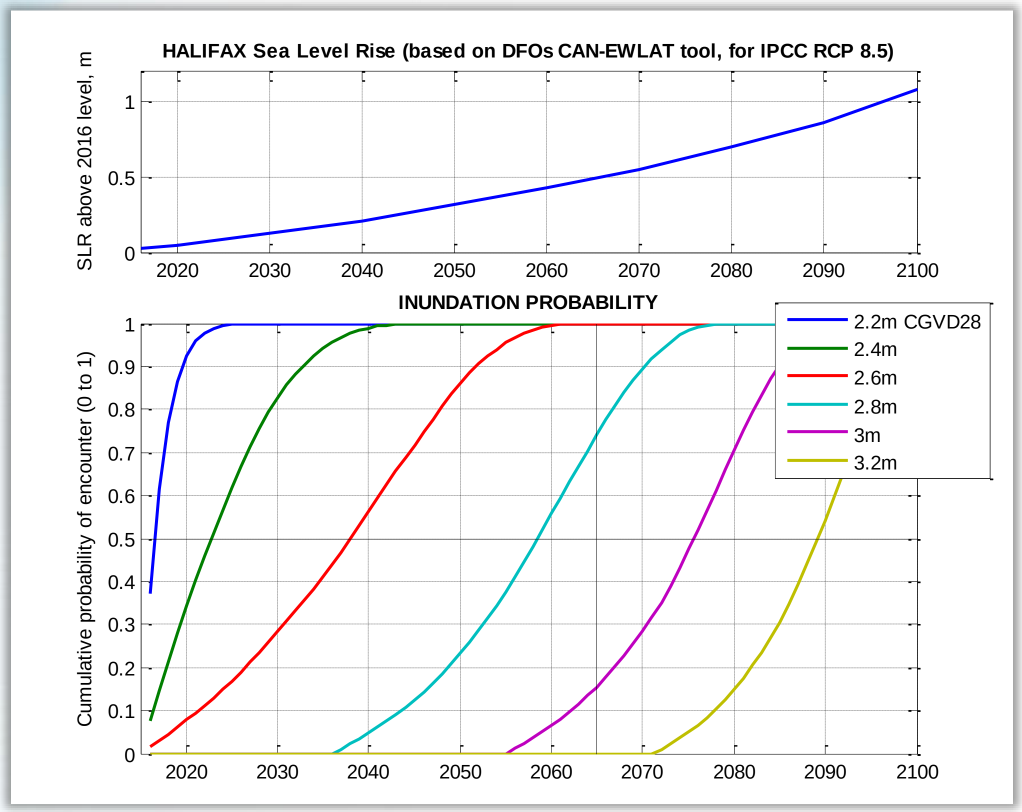


Figure 4.6: Probabilistic Risk Assessment Accounting for Sea Level Rise, Based on Storm Surge Statistics from the Halifax Tide Gauge Combined with Sea Level Rise Projections

For flood mapping, as a precautionary approach, and given that SLR may actually exceed IPCC upper-bound projections by 2100, we chose the deterministic approach to represent future conditions by year 2100, as follows. The allowance value of 1.11 m estimated from the DFO report (Zhai *et al*., 2014) for Halifax for the year 2099 was selected as the design future sea level rise value for this study. The design sea level time series for existing sea level conditions were therefore shifted by 1.11 m to develop the design sea level time series for future sea level rise conditions, as presented in **Figure 4.7**. For the purposes of this report, the sea level rise estimate for the year 2100 is defined as the allowance published for the year 2099.

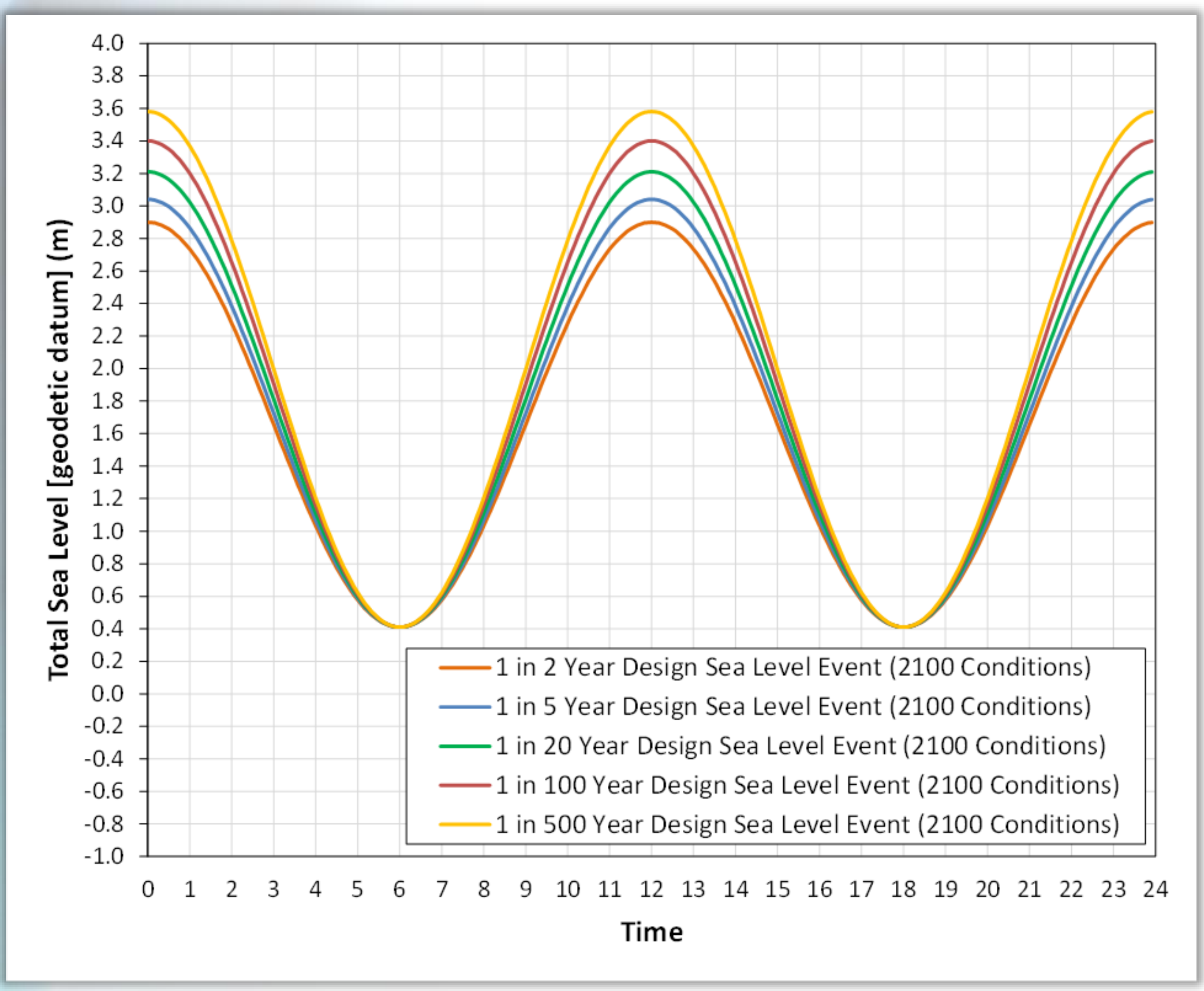


Figure 4.7: Design Sea Level Events for Future Sea Level Rise Conditions (Year 2100)

4.2.3 Design Ice Accumulation Event

4.2.3.1 Ice Jam Formation Characteristics

Ice jam formation can occur during the freeze-up period at the beginning of winter, or during the break-up period in spring. During the freeze-up period, ice forms on the river surface beginning at the banks. Ice crystals may also develop within the river as frazil ice, which is very common in rapids. The ice crystals tend to coalesce and accumulate, and may become attached to the underside of the ice cover or to the river bed as anchor ice.

Frazil pans and floes are major components in the formation of a river's initial ice cover. In tranquil reaches, this cover is a mere surface layer of ice floes and pans, but elsewhere it can be several layers thick.

Ice jams during the freeze-up period usually form where floating ice slush or blocks encounter a stable ice cover. There are, however, certain features that, in conjunction with ice cover, enhance the probability of ice jam formation: bridge piers, islands, bends, shallows, slope reductions, and constrictions.

During breakup in the spring, or during winter thaws, an ice jam results from the accumulation of ice from the breakup of the upstream ice cover. A rise in water levels may result from the spring snowmelt, or a sudden midwinter thaw, common in Atlantic Canada. Midwinter thaws are often accompanied by substantial rainfall, resulting in a rapid increase in water levels and severe ice jams. Compared to other flood events, ice jams can occur when minor rainfall events occur, or can even be due to flow caused by ordinary spring thaw, making them difficult to predict.

There are two main features of ice jams that can cause flooding. First, ice jam thickness can be considerable, amounting to several metres. Secondly, the underside of the ice cover is usually very rough. Under open water conditions, the only frictional resistance retarding the flow of the water is the streambed. The rougher the streambed, the greater the depth required to pass a given discharge. With an ice jam in place, the additional ice and very rough lower surface retard flow. Therefore, the flow depth has to be much greater than for open water.

An important factor to the level of ice build-up is the amount of ice existing on the banks just prior to the jam occurring. This amount is dependent on many factors, such as the variation in temperature and water levels in the entire winter period leading to the ice jam.

4.2.3.2 Ice Jam Accumulation

To evaluate the potential ice thickness that can be reached in the Sackville River and the Little Sackville River, a statistical analysis was carried out using the US Army Corps of Engineers' Ice Engineering publication: "Method to Estimate River Ice Thickness Based on Meteorological Data". This publication describes a formalised approach to estimating maximum potential ice thicknesses based on climate data and heat transfer processes, using the concept of "Accumulated Freezing Degree Days". The methodology included calibration against actual ice thickness measurements carried out by the Canadian Ice Service, the closest location being in Caraquet, NB. **Figure 4.8** shows the available ice thickness measurements at this location, from 1974 to 1986.

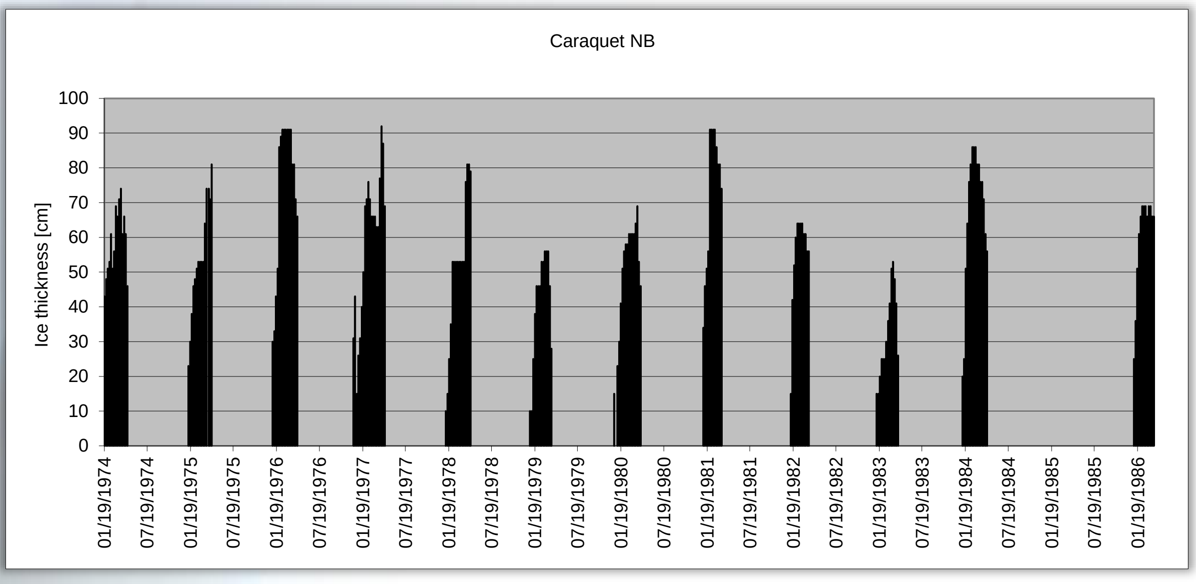


Figure 4.8: Ice Thickness Measurements in Caraquet, NB

Long term temperature data was obtained from the climate station at Shearwater Airport, and the maximum annual ice thicknesses for each year was then compiled and analysed with statistical distributions. The results from this analysis are presented in **Figure 4.9**.

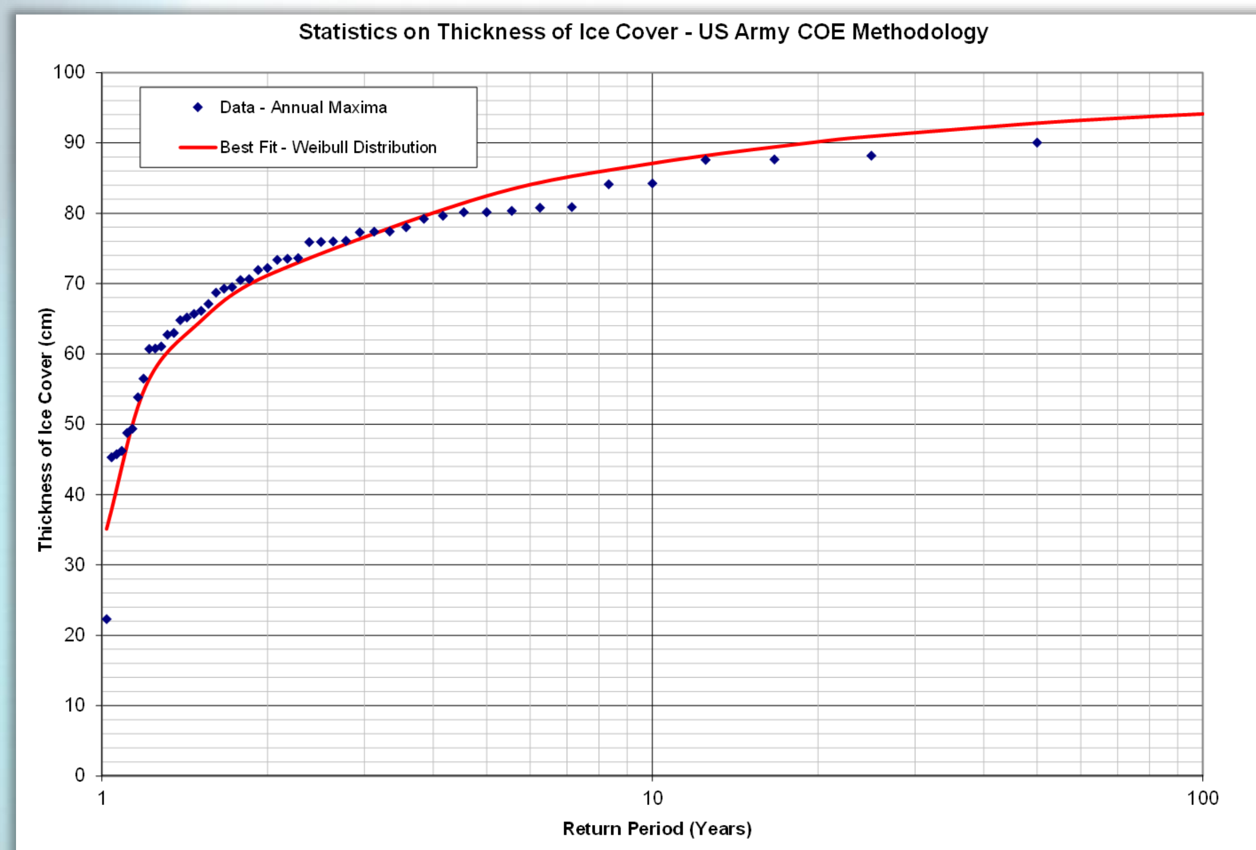


Figure 4.9: Estimation of 1 in 100 Year Ice Thickness Based on Statistical Analysis

Based on this method, the 1 in 100 year ice accumulation was estimated to 94 cm. This value was therefore input into the HEC-RAS model as the initial ice accumulation parameter to simulate the ice jam flood. As shown in **Table 4.1**, the ice jam flood was simulated by also inputting peak flows and sea levels corresponding to the average of the annual maximum events. Including a larger rainfall event (such as the 1 in 100 year rainfall event) at the same time as the 1 in 100 year ice accumulation would change the return period of this occurrence to a value significantly greater than 1 in 100 years.

It is noted that while the most accepted methods were used in this study to conduct simulations of ice jam processes, the results are still highly uncertain. The first reason is that there was no ice jam thickness data available for the Sackville River or Little Sackville River to calibrate the model on. Secondly, the results are still highly uncertain, and too variable to produce flood lines that can be relied upon with confidence. The results are therefore presented for information, and should be reviewed when any work in the river, including bridge repairs or upgrades, is conducted, to lessen potential risks.

In general terms, the flood lines generated by ice jams are of a similar order of magnitude of width to the 1 in 100 year flood lines generated by rainfall and sea level. The model does not seem to show any significant increase in ice thickness beyond the 1 m thickness input in the initial conditions, except in the undeveloped areas of the Sackville River, where the river braids significantly through wetland areas. In general, compared with the previous analysis results (1 in 100 year rainfall, no ice jams), the model results indicate that there is a slightly increased risk of flooding in the Little Sackville River caused by ice accumulation. As shown in the water level profiles and noted ice thickness accumulation (**Appendix D, Profile #33 to #35**), this is probably highest in the Gantry Road and Beaver Bank Cross Road culvert areas, but also present at the Lucasville Road Bridge, the Downsview Mall, the Sackville Cross Road and the baseball field by Highway 101. Other than those areas, the ice jam model does not seem to highlight any significant additional risk of flooding that is not present in the previous model (based on the rainfall and sea level analysis).

4.2.4 Historical Design Storm Event

The delineation of a historical design storm event can help compare theoretical model results to actual known and measured storm events. During Progress Meeting #4 on October 6th, 2016, the March 2003 storm event was selected, as it generated the highest peak flows in the Sackville River watershed.

4.2.5 Phase I River Flow Frequency Analysis

HRM requested that the flows estimated from the river flow frequency analysis completed in Phase I (GHD 2016) be used to delineate flood lines. These flows (presented in **Table 4.1**) were therefore input at the most upstream locations in the model (McCabe Lake and Little Lake). However, it should be noted that the flood lines delineated for this scenario poorly represent the flood extents throughout the rivers, as they are only based on flows estimated for single locations. A comparative analysis between the flood lines produced by the single station flow frequency analysis results and by the calibrated model is therefore not recommended.

4.3 Flood Line Delineation

As shown in **Table 4.1,** each flood line (with the exception of the ice jam flood line) presented in **Appendix E** was delineated based on a combination of the design rainfall and design sea level events. Since extreme rainfall events often bring storm surges and seiches, the 1 in 2 year sea level event, or the average annual maximum sea level event, was selected as representative water level conditions in the Bedford Basin during extreme storm events. Similarly, the 1 in 2 year rainfall event, or the average annual maximum rainfall event, was selected as representative river flow conditions during extreme sea level events. For a given return period, the maximum water level between each of the two scenarios at each location along the rivers was therefore selected to delineate the respective flood line. Thus, each flood line delineation consists of the maximum of the extreme rainfall flood event and extreme sea level flood event. The resulting flood lines therefore do not represent the flood extents for single events that can occur, but rather combined flood extents for two different types of events that can occur, each having the same return period. It is noted that the downstream sea level influence was found by the model to extend to just upstream of the Bedford Highway Bridge for both existing and future sea levels **(see Profiles #10 and #23 in Appendix D)**.

All flood lines were delineated by interpolating the water levels output by the model results and then intersecting them with the 1 m resolution lidar DEM using GIS tools.

4.3.1 Field Verification of Flood Lines

A field verification of the flood lines was carried out to verify that the floodplain and channel hydraulics observed in the field were representatively included in the model. However, since high resolution (1 m) lidar data was available for this study, features such as sudden changes in floodplain width, sudden drops along the floodplain profile, berms and secondary floodplain flow paths (see **Section 3.3.6**) were also able to be identified by the lidar data. Thus, as described in **Section 3.3.3**, lidar was used to fill data gaps in the survey data where topographical features that could potentially impact floodplain hydraulics were missing. These features, such as the hydraulics of the berms and secondary flow paths located at Bedford Place Mall and next to Highway 101 near the Sackville River and Little Sackville River confluence, were confirmed in the field. Notable other field verifications made during the site visits were as follows:

- The river channels upstream of the confluence of the Sackville River and the Little Sackville River were significantly shallower than further downstream, with water depths in the order of approximately 0.3 m on average. Thus, as noted in **Section 3.3.4**, the use of standard lidar data (which does not penetrate water) to develop cross sections for these upper reaches was verified by the shallow water observations to have minimal impact on evaluating large storm events.
- While visiting the Feely Lake Dam and the Lumber Mill Pond Dam, the operational practices of the dams and the bathymetry of the lake and pond were discussed with Barrett Truss and Building Supplies. As noted in **Section 3.3.5**, adjustments were made to the model to reflect the operational procedures implemented for when significant rainfall and snowmelt events are forecasted. Furthermore, Barrett Truss and Building Supplies noted that the overflow path of the Lumber Mill Pond is along the road and parking lot near the south corner of the pond, whereas minimal flooding should be expected near the northwest corner of the pond. These observations are reflected in the flood lines presented in **Appendix E**. Field observations and measurements taken of the hydraulic

structures on the property also ensured that the model was representative of current hydraulic conditions.

- While visiting the Bedford Rifle Range, staff at the Department of National Defence noted that the Bedford Rifle Range Bridge approaches overtop approximately three to four times per year on average. This observation therefore verifies the flooding of this area presented by the flood lines in **Appendix E**.

4.4 Encroachment Analysis

Encroachment analyses are used by the Federal Emergency Management Agency (FEMA) to show how activities such as infilling and development in the regulatory floodway will produce an increase in flood levels, based on by hydraulic modelling (FEMA 37 1993). The goal of the encroachment analysis is to therefore determine the reduction in floodplain width at every location along the river that will cause a specified increase in water level. For this study, a 5% increase in water level was used to define the encroachment, and the analysis was carried out for the 1 in 100 year rainfall event using the HEC-RAS model. The HEC-RAS User's Manual Version 5.0 (Brummer 2016) notes on page 10-1 that normally, the base flood is the one-percent chance event, and the designated height is one foot, unless the state has designated a more stringent regulation for maximum rise. In order to be consistent with this, a 5% rise was selected, which on average corresponds to approximately 1 foot. A percentage value was selected in order to generate results that show the relative sensitivity of flooding over the whole river system as a result of floodplain development, rather than an absolute value that does not support comparative analyses. Method 4 was selected in the HEC-RAS model for the analysis, which runs multiple steady state simulations to determine the encroachment widths throughout the rivers based on a user defined increase in water level.

A typical land use planning policy allows development to still occur between the 1 in 20 and 1 in 100 year floodplain extents, provided that the development is flood proofed. This analysis is made to identify specific locations where such a policy would result in increased flooding risks. The resulting 1 in 100 year encroachment floodplain width was therefore compared to the existing 1 in 20 year floodplain width (from the HEC-RAS model) to determine locations where development between the 1 in 20 year and 1 in 100 year flood lines would create increased upstream flooding risk. This comparison is presented graphically in **Figure 4.10** for the Sackville River and **Figure 4.11** for the Little Sackville River. As shown in the figure, the 1 in 100 year encroachment floodplain width was estimated to slightly exceed the existing 1 in 20 year floodplain width at several locations, with the most significant location being along the Little Sackville River between stations 9720 m and 9760 m (this is the section where the river borders Highway 101 upstream of the intersection with Highway 102). The floodplain is already restricted by the highway, and this analysis shows that any further restriction would cause significant impacts on upstream water levels.

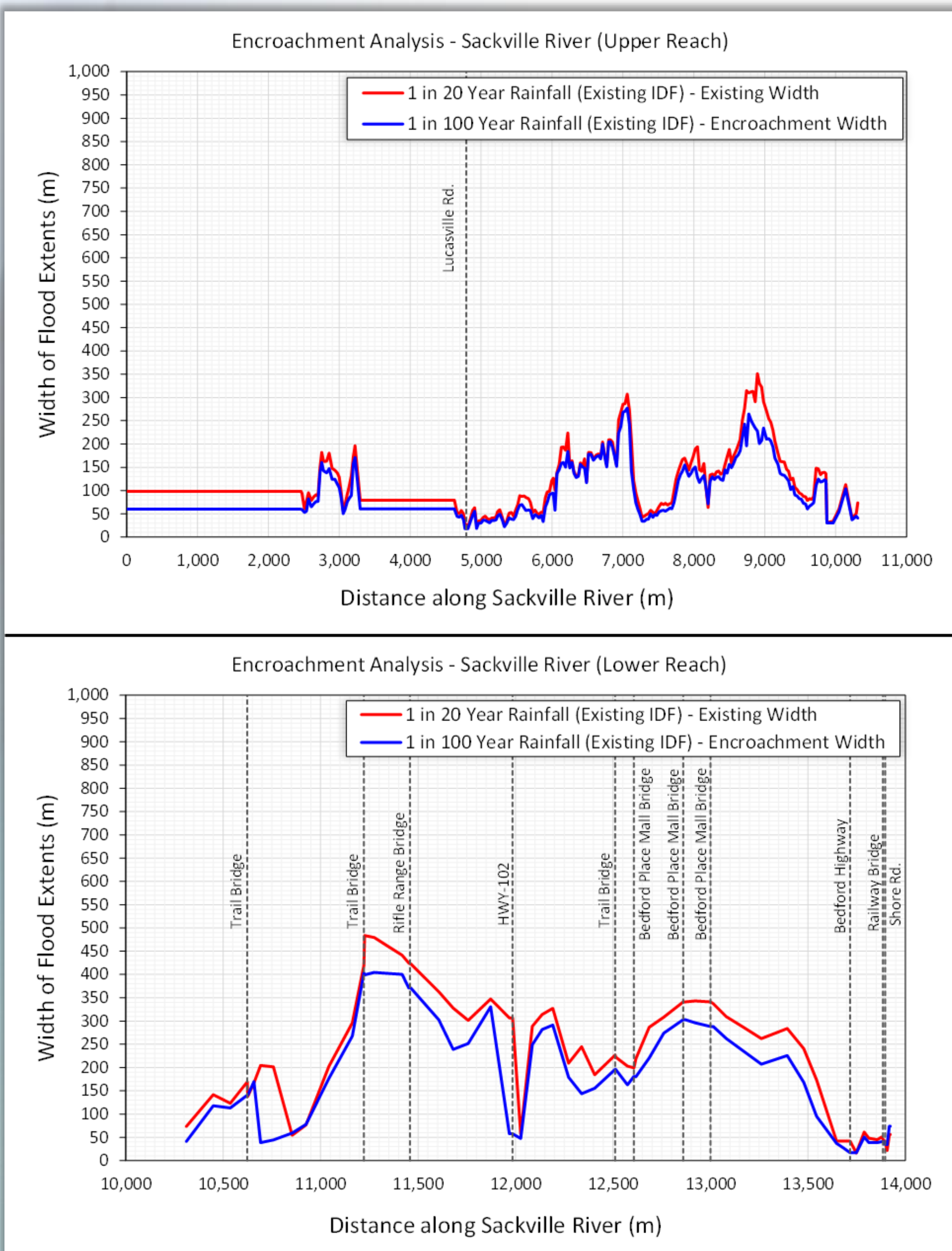


Figure 4.10: Encroachment Analysis Results (Sackville River)

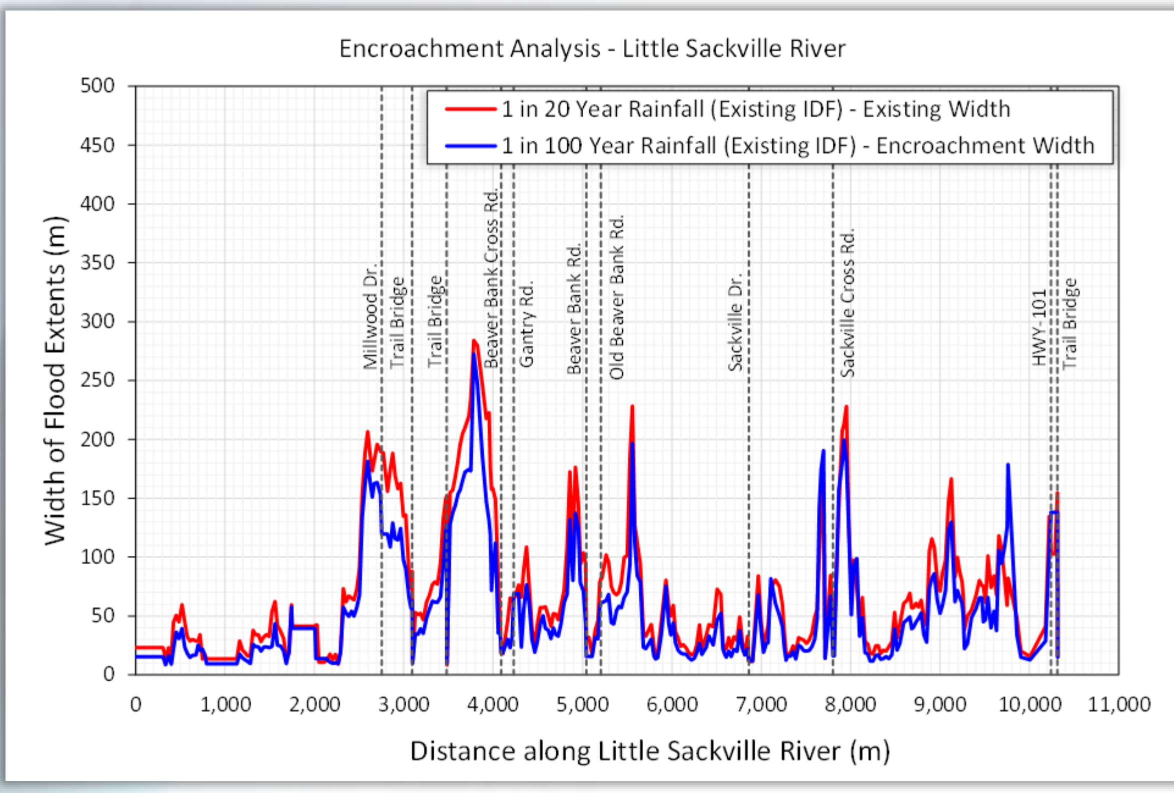


Figure 4.11: Encroachment Analysis Results (Little Sackville River)

4.5 Selection of Base Flood

The Request for Proposals included a requirement for a recommendation of the "Base Flood". This was defined by HRM as a pair of flood lines, for the floodway (1 in 20 year) and floodway fringe (1 in 100 year), for planning and regulatory purposes. Since the scope of this study does not include any stakeholder consultation, assessment of vulnerability of floodplain land uses, infrastructure and services, nor any review of existing and future planning challenges and opportunities, the current recommendation is strictly related to river hydrodynamics and the current state of climate change science.

In this respect, CBCL agrees with following HRM's proposition to select the most conservative model result to ensure that known risks to public safety are not being ignored.

This means that the future 1 in 20 year and 1 in 100 year flood lines in worst case climate conditions is recommended, which, in this instance, includes the following characteristics:

- Fall seasonal watershed characteristics for the Little Sackville River;
- Winter seasonal watershed characteristics for the Sackville River;
- 24-hour duration design storm event for the Little Sackville River;
- 48-hour duration design storm event for the Sackville River;
- Future development conditions for both watersheds (as known at the time of this study by HRM);

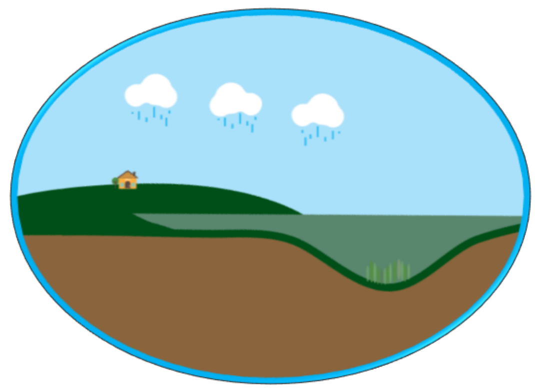
- Climate change conditions for the Western University IDF-CC Tool upper bound result for the 2070-2099 period; and,
- 1 in 20 year and 100 year return periods.

The "Base Flood" map showing the results of the modelling with the above characteristics is presented in Map 9 in Appendix E.

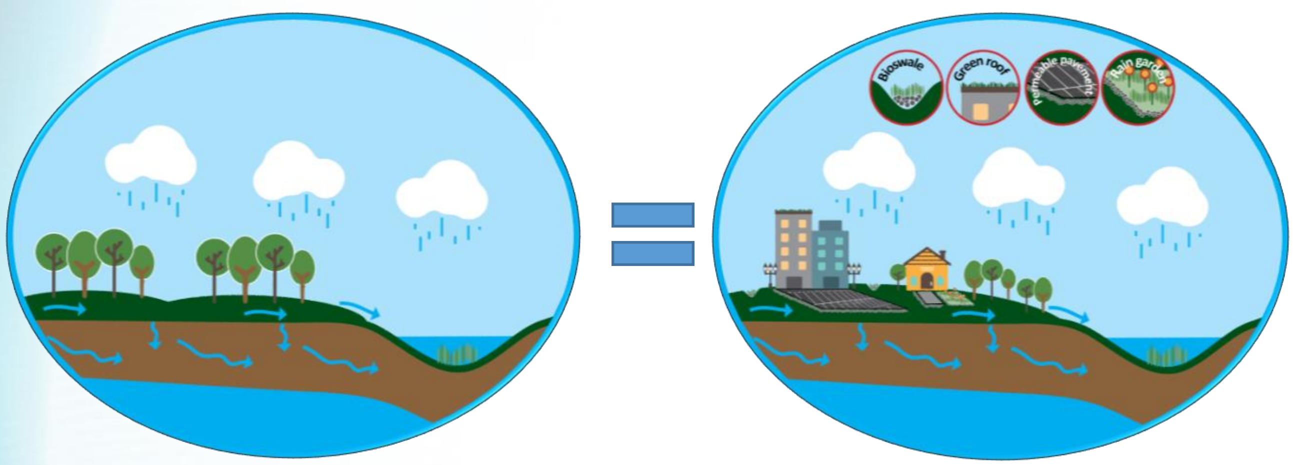
4.6 Analysis of Flood Mitigation Options

HRM requested a discussion of potential flood mitigation options. A high level flood mitigation review was carried out to discuss potential flood mitigation options throughout the community. In general, the goal of flood mitigation is to protect vulnerable areas from flood damage. Separating the water from the vulnerable areas can be achieved by taking some of the following main approaches. They are listed in order of recommended priority.

1. **Preventing future development in flood prone areas by planning and zoning by-laws:** This is the most effective and lowest cost strategy to avoid placing future services, land uses and infrastructure at risk.

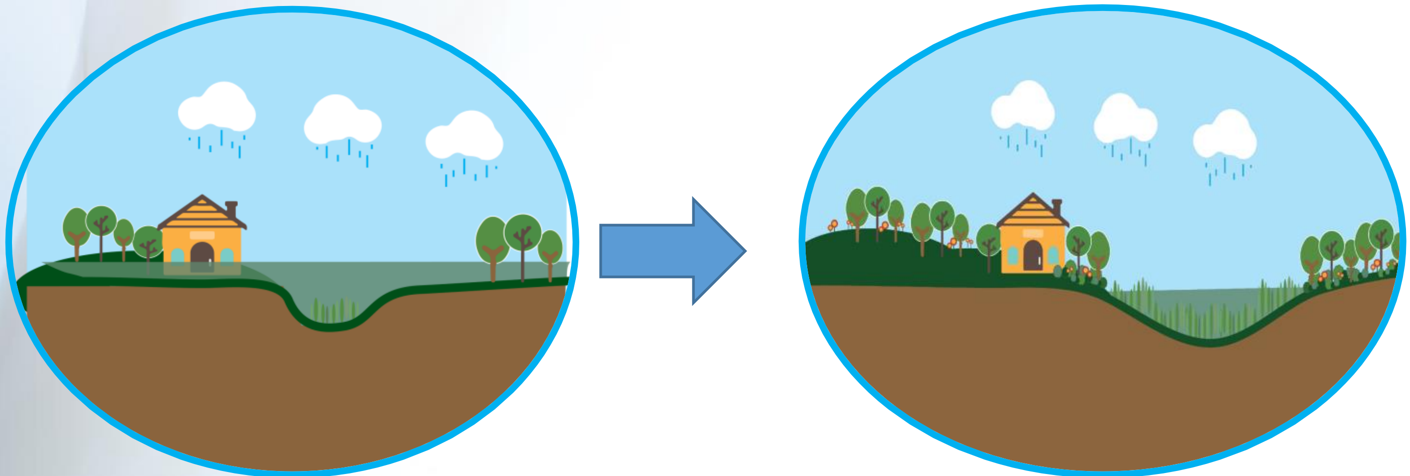


2. **Preventing excess water in the vulnerable areas from being generated in the first place**:

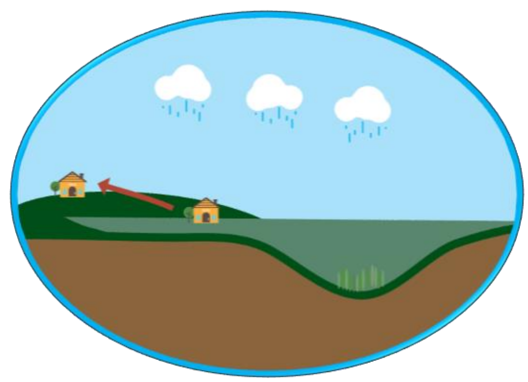


This is best achieved though measures such as Low Impact Development and Stormwater Best Management Practices (LID and BMPs). In-stream controls and detention ponds are only moderately effective;

3. **Increasing the capacity of the river using natural techniques**:

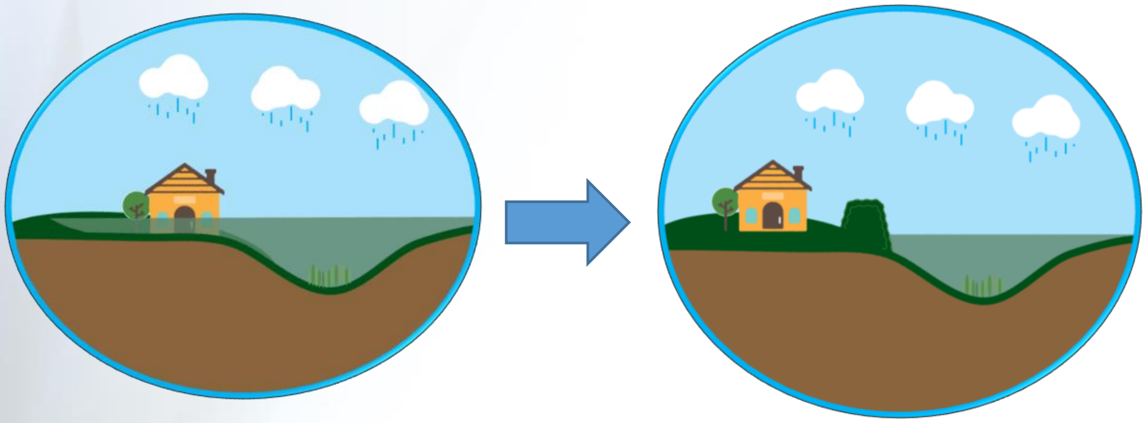


To avoid the high costs and risks of upgrading bridges and culverts, additional capacity can be provided by conducting river restoration and widening the floodplain where room is available;



4. **Retreating or displacing the vulnerable services, land uses and infrastructure**:
This is the most effective long term measure, but can come at a high cost and is a sensitive issue for homeowners;

5. **Protecting the vulnerable areas by building walls and berms**:
This is the least recommended approach, and would only be recommended if the approaches recommended above are not successful. This option can also include raising the ground level, as was done in the Ellenvale Run area of Dartmouth. This means, however, narrowing the natural floodplain and increasing upstream flooding risks, river velocities, risks of erosion and increased risks to public safety if the berms fail;



The following examples provide some more local context to the measures described above. Some advantages and drawbacks are presented, as well as some discussion on their potential suitability for various areas of the Sackville River and Little Sackville River watersheds. Options are presented in order of recommended priority.

4.6.1 Planning Measures such as Zoning and By-Laws

This is typically the first recommendation in flood mitigation studies: minimising potential future flood risks to vulnerable areas by controlling development within the floodplain. A common practice is to prevent any development within the "High Risk" zone, or 1 in 20 year floodplain, and allow only non-permanent uses that do not infringe on the floodplain in the "Moderate Risk" zone, or 1 in 100 year floodplain. It is worthwhile to note that any construction in the floodplain that somehow restricts the flow of water will increase flooding risks to the upstream areas. It is therefore fundamental that before the Municipality considers allowing any form of development within the 1 in 100 year floodplain, impacts of this action be studied, understood and accepted. Since planning is oriented towards controlling future development, the flood lines should consider future climatic conditions and therefore take into account climate change.

4.6.2 Reducing Peak Flows through infiltration

Reducing peak flows can be achieved by either storing water upstream, or infiltrating the rainfall before it becomes runoff. Direct storage of water may not be effective in this case since the major watersheds are relatively large and would therefore require a significant amount of excavation and land area to make an impact on the flows.

Infiltration of stormwater, however, may have good potential. This is a natural process, and one that has been tampered with through development of the watershed. Low Impact Development and Stormwater Best Management Practices (LID and BMPs) can restore the natural (pre-development) hydrologic balance of infiltration vs. rainfall, by constructing infiltration areas, permeable surfaces, perforated pipes, etc. Infiltration of rainfall is a natural process, and one that has been tampered with through development in the watershed. LID and BMPs can restore the natural (pre-development) hydrologic balance of infiltration vs. rainfall, by constructing infiltration areas, rain gardens, bioswales, permeable surfaces, perforated pipes, etc.

This can start very simply, by implementing a program of roof downspout disconnection, to be re-directed to a green space. Unfortunately, if implemented immediately, its cost would be prohibitive. However, if implemented as part of planning and building permit approval regulations, each time any modification is made, or a new construction is made, the incremental cost can be minimal, with significant improvements.

This approach is therefore a first recommendation, to be implemented wherever possible, whenever the opportunity presents itself. Since LID and BMPs can take many shapes and forms, it is recommended that a study be conducted to identify the best opportunities currently available, and optimise their implementation with planned capital works programs. Similarly, any future development should require such measures to maintain peak flows, runoff volumes, flows, water quality and increase biodiversity.

4.6.3 Increasing Capacity and Storage through River Restoration

A means of increasing the conveyance system capacity as well as storage would be to increase the channel size. Widening the rivers through the entire study area would involve large volumes of excavation and would need to be conducted in concert with upgrading the bridge and culvert structures. This approach would also be difficult to support from an environmental perspective and would not likely receive approval by Nova Scotia Environment.

However, if this is conducted as part of a river restoration effort, where previous activities have impacted the natural shape and meanders of the rivers, there is potential to provide increased drainage capacity, additional storage, as well as restore natural features of the river.

Flood reduction as part of river restoration is currently being carried out by CBCL Limited in Yarmouth and is effective where the river has been channelized by urbanisation, and wetland and fish habitat has been lost. In the Sackville River and Little Sackville River, it is worth investigating the potential for river restoration efforts, which can increase the drainage capacity and storage, while increasing biodiversity and restoring ecosystem habitat. Meanders can be reintroduced in the system, wide enough to carry the water efficiently, but also including pools, riffles and a low flow channel to provide fish passage. As carried out in Yarmouth, the Municipality may only need to carry out the excavation work, while community groups can conduct the planting of suitable species.

4.6.4 Purchasing the Impacted Properties

This approach, which is currently being pursued in Sydney, Nova Scotia, has clear benefits: the impacted individuals are now permanently safe, properties at risk can be restored to the natural floodplain, upstream flooding risks can be reduced, there is no further maintenance cost or residual risk, and the riverfront area can now be enhanced for public enjoyment.

It is the only permanent option that needs no further maintenance to be effective, and it can still be used for non-permanent uses such as park or recreational space. The main challenges to this option are therefore its cost and resistance from property owners who may not be receptive to selling and moving. If these challenges can be overcome, this option is recommended as the next priority.

4.6.5 Bridge and Culvert Upgrades

This has historically been the most common first step taken to reduce flooding risks. Flooding can be caused by a multitude of factors, including not only high flows and insufficient bridge or culvert capacity, but also high surface and channel roughness, low channel slope or insufficient room in the floodplain. Perhaps the simplest approach, if not the most cost-effective, is to assess whether bridge and culvert structures have an impact on the overall flood levels and whether or not upgrading these structures will address the flood risks.

The model results shown in the profiles of flood levels along the river reaches indicate that some structures do appear to be creating obstructions to the flow and increasing water levels on their upstream side. These include the Beaver Bank Cross Road, Beaver Bank Road, Sackville Drive and Lucasville Road structures. In the downstream reach of the Sackville River (downstream of the confluence of the two rivers), the model results do not seem to give evidence of flows being notably restricted by any given structure.

Upgrading the structures that create some level of obstruction to the flow, however, will be costly. Even though it would reduce water levels and risks of flooding on their upstream side, it would also increase risks of flooding on their downstream side. The decision to upgrade them will therefore need to be supported by an assessment of the balance between current risks of flooding of upstream areas, the potential increased risks of flooding of downstream areas, and the risk of the culverts or bridges being washed out. Millwood Drive and Lucasville Road are examples of structures which, if structurally safe, may actually be reducing flooding risks in vulnerable downstream areas.

The conclusion of this discussion is that upgrading structures is not a recommended option for the Sackville rivers. Options presented above should be pursued first, and if upgrading a structure is necessary for structural reasons, a detailed, river system wide modelling exercise should be carried out to ensure that no increased risks of flooding are created for upstream or downstream landowners.

4.6.6 Structural Flood Protection

Structural flood protection is a means of constructing berms to protect areas at risk. The mechanism is that berms will protect vulnerable areas by reducing the floodplain width. This reduction can lead to increased water levels upstream, and should therefore be considered only with careful thought and analysis. Berms also create a residual risk, in which the protected areas could still get flooded by an event greater than the design event, or by failure of the structure (leading to larger damage). It is very important to understand the concept of residual risk since it will have to be accepted by both the HRM and those who are protected. Constructing flood protection measures therefore means that not only capital costs will have to be incurred, but also operation and maintenance costs, as well as costs of the potential flood damage which might be of an even greater magnitude than if a flood mitigation structure had not been there.

Constructing flood protection berms would also require locally raising some of the roads to prevent water from going around the berms, and possibly require pumping stations. It is estimated that in most cases, relatively small areas would be draining towards the watercourses behind the new berms, and therefore culverts with check valves may be sufficient to convey local drainage into the watercourses and pumping may not be required.

While some areas may benefit from flood protection berms (wherever unwanted flooding is occurring), the berms would not be feasible in locations where they could increase upstream water levels. This risk would need to be identified by individual hydraulic modelling and analysis studies of the entire river system prior to the consideration of any such measure.

4.6.7 Property Raising

Raising the ground level of individual properties above the peak water levels is an alternative method of flood protection that has similar impacts on the floodplain hydraulics as constructing protective berms. In this manner, the flood waters from the river would stay in front of the properties and the surface drainage from the affected areas will flow by gravity naturally towards the river. An example of a raised property in Dartmouth, Nova Scotia is shown in **Figure 4.12**.

However, several difficulties are involved with grading land on private properties. As a first step, a survey should be conducted to determine the extents of the necessary land grading. Land grading may not be possible if homes are located on land which is too low. If determined possible, all features of the original property should be reinstated as close to the original conditions as possible, including sheds, fences, but also trees and shrubs. Landowners may offer resistance, and it is therefore necessary to take early steps to discuss and obtain buy-in from every affected land owner before proceeding with this option.

Figure 4.12: Example of Raised Property along Ellenvale Run in Dartmouth

4.6.8 House Raising

An alternative option to raising the entire property that is gaining popularity, especially after the recent wave of floods in the US and in the UK, is to raise just the home itself. Jacking companies have been developing products that can raise homes for an estimated average cost of $50,000 per house. This would be accompanied with infill to adjust the surrounding land to the level of the house and make it accessible. As with raising the property, close coordination with the homeowners will be needed.

As noted for the option of constructing berms, raising the level of the land is not recommended unless more recommended options have been exhausted.

4.7 Flood Scenario Modelling, Flood Mapping and Flood Mitigation Analysis Findings

Tidal effects (see **Appendix D, Profile #10 and Profile #23**), with and without climate change, were shown by the model to be limited to just upstream of the Bedford Highway.

Results of modelling rainfall impacted by climate change (see **Appendix D, Profile #14 to #22 and Profile #24 to #32**) were also generated. It was found that the large number of existing climate change models, combined with the various methods of transformation of the results into rainfall amounts, produced a wide range of results, with the highest rainfall amount calculated at 283.9 mm, a 70% increase compared to the existing 1 in 100 year rainfall amount (166.7 mm). Interestingly, while the water levels increased accordingly, the floodplain width did not significantly widen. This is mostly a result of the existing floodplain topography in which the floodplain edges have higher slopes, resulting in a small change of width when water levels increase.

The 1 in 500 year event results showed larger flooding extents than the 1 in 100 year event, but again, to a limited extent. Since the total rainfall amount in 24 hours is 199 mm for the 1 in 500 year event, it is only marginally higher than the 1 in 100 year total rainfall amount (166.7 mm), and notably lower than the climate change amount (283.9 mm). Results are therefore much closer to the 1 in 100 year event than the worst case climate change scenario.

A discussion of potential flood mitigation options is presented in **Section 4.6**, and reviews the benefits and challenges associated with each potential measure. Although this assessment did not investigate in detail, nor model, any flood mitigation option, certain high level aspects can be drawn from the results. The flood line delineation showed that climate change impacts clearly have the potential to increase flooding risks and should be considered in any future planning decision. While the upper reaches of the Sackville River are mainly undeveloped, its lower reaches, and most of the Little Sackville River, are quite highly urbanised, which is both increasing river flows as well as creating vulnerabilities. The least intrusive and most cost-effective flood mitigation option is to implement stormwater infiltration measures (LID and BMPs). Such measures, if implemented in future development and during resurfacing or repair works, can have a very low direct cost but make a clear impact in flood reduction. Other options discussed include conducting river restoration and increasing capacity in river sections that have been channelized, and purchasing properties at risk. The planning regulations will be central to managing future development and it is recommended that they include language on runoff control, flood proofing or limited uses in floodplain areas. Options such as upgrading bridge structures, building berms, or raising the level of the land or homes, should only be used after the above options have been exhausted. Stakeholder consultations and modelling should be carried out to identify the best compromise between protecting vulnerabilities, overall stakeholder needs, ecosystem protection and costs.

CHAPTER 5 SUMMARY AND CONCLUSIONS

A need to update the previous flood line delineation analyses was identified by the HRM. This need arose from the emergence of updated information and tools of much better quality (topography, flow and water level, rainfall, hydrologic and hydraulic computer models), as well as research on climate change, and pressure from the Sackville Business Association.

This study has assessed the hydrology and hydraulic regime of the Sackville River and the Little Sackville River, as well as their respective watersheds, in order to produce floodplain maps for various flood scenarios. Flood risks were evaluated based on a calibrated hydrologic and hydraulic model using PCSWMM, and an ice jam hydraulic model using HEC-RAS. Model calibration and validation for the PCSWMM model was carried out for flood events corresponding to each of the four seasons, and for each of the two rivers. Design flood scenarios included variations in seasonal conditions, rainfall conditions under climate change, sea level conditions under climate change, development conditions and ice conditions for various rainfall events and sea level events. The resulting flood lines delineated for this study are presented in **Appendix E** where two sets of maps are presented: one overlaid on community mapping and one overlaid on orthographic photography. **Table 4.1** lists each map produced for this study, including seasonal changes (**Map #1**), historical design storm (**Map #2**), existing climate, existing and future development (**Map #3 and Map #4**), various scenarios of climate change for existing and future development (**Map #5 to #10 and Map #14**), ice jam analysis (**Map #11**) and previous flood line comparison (**Map #12 and Map #13**). Mapping of the Phase I river flow frequency analysis results is presented as well on **Map #15**.

The thorough analysis presented in this report was carried out to support the flood extents produced by the hydrologic and hydraulic models. The flood extents may be incorporated into future planning documents, which warrants this thorough analysis. Included in this assessment was also an in-depth analysis of climate change impacts on rainfall and sea levels. Since climate change is to be considered in planning documents, it was essential to use the best science and tools available to evaluate those effects. This is presented in **Section 4.2.1**. Other significant inputs to this assessment included a radar-rainfall analysis used to improve the model calibration, presented in **Section 3.4.2**, an ice jam analysis, presented in **Section 4.2.3**, and model calibration and validation for each season in the year for both rivers, presented in **Section 3.4**.

The RFP required a recommendation for the selection of a Base Flood. This was defined by HRM as a pair of flood lines, for the floodway (1 in 20 year) and floodway fringe (1 in 100 year), for planning and regulatory purposes. Since the scope of this study did not include stakeholder consultation, assessment of vulnerability of infrastructure, floodplain land uses and services, nor any review of existing and future planning challenges and opportunities, the current recommendation is strictly related to river hydrodynamics and the current state of climate change science.

In this respect, CBCL agrees with following HRM's proposition to select the most conservative model result to ensure that known risks to public safety are not being ignored.

This means that the future 1 in 20 year and 1 in 100 year flood lines in worst case climate conditions is recommended, which, in this instance, includes the following characteristics:

- Fall seasonal watershed characteristics for the Little Sackville River;
- Winter seasonal watershed characteristics for the Sackville River;
- 24-hour duration design storm event for the Little Sackville River;
- 48-hour duration design storm event for the Sackville River;
- Future development conditions for both watersheds (as known at the time of this study by HRM);
- Climate change conditions for the Western University IDF-CC Tool upper bound result for the 2070-2099 period; and,
- 1 in 20 year and 1 in 100 year return periods.

The first result of interest is the comparison with the previously generated flood lines. **Map #13** in **Appendix E** presents the comparison of the 1 in 100 year flood lines from the 1980's regulated floodplain width, the Porter Dillon study from 1999 and the current modelling results. It is clear that the use of the lidar data and computer mapping techniques improved the resolution and consistency of the model results (previous results were drawn by hand). Beyond this, hydrotechnical modelling also shows the flow regime in steep sections allowing the river width to narrow (for example around the Downsview Mall), which was not identified in previous assessments. The other prominent difference is in the downstream areas of the Highway 101 and the Bedford Place Mall. The updated model results show significantly larger flood extents, where both locations are under extensive flooding during the 1 in 100 year event. Those changes are estimated to result more from the improved quality of calibration, hydrodynamic modelling and surface topographical data, rather than the increased extent of the flow monitoring record.

Other findings from this analysis include the identification of factors that lead to the flooding extents generated by the models. The analysis of structure constrictions only identified four structures that create notable impediments to the passage of water. Those structures are the following:

- Beaver Bank Cross Road (Little Sackville River);
- Beaver Bank Road (Little Sackville River);
- Sackville Drive (Little Sackville River), and
- Lucasville Road (Sackville River).

Other than those structures, there are few anthropogenic impacts to the natural shape of the river channel, other than river diversions to circumvent development. This is a notable finding, because it demonstrates that flooding outside of the river channel (i.e. in the floodplain) is a natural phenomenon. Natural rivers create over time a natural channel whose size is reflective of average river flows. Flows above average values carve a natural floodplain in the landscape. The majority of floodplain extents in Nova Scotia rivers were created during the melting of the last ice age glaciers, approximately 10,000 years ago. These are natural floodplains, which rivers occupy in higher than average flows. The model results show that the current 1 in 100 year peak flood extents occupy a large portion of this natural "ice melt" floodplain. Notably, the model results also indicate that events of a greater magnitude, including the 1 in 500 year event, the PMP or future events influenced by development and climate change lead to increased floodplain width (as expected), but only by a small relative amount. This means that high flows will regularly fill the floodplain, but that extremely high flows will still stay within this main floodplain. It is important to note this because it means that the floodplain is necessary for the conveyance of high flows. Development within the floodplain will unavoidably be at risk of flooding, and any restriction of this floodplain will lead to higher upstream water levels. Notable development in the floodplain includes:

- the road crossings noted above;
- the Downsview Mall;
- the development around Sackville Cross Road;
- the Contessa Ct. and Sami Dr. residential developments, and
- the Bedford Place Mall and adjacent residential development.

The most notable infrastructure that alters the floodplain is the Highway 101 crossing and its interchange with Highway 102. All the above areas are at risk of flooding because they lie within the natural floodplain. Their impacts on flood levels seem to be limited, as seen from the water surface profiles in **Appendix D (Profile #7 to #9)**, but this has not been confirmed by modelling a scenario where this development does not exist.

The assessment of seasonal effects on flood risks also yielded interesting results. The Little Sackville River, being more urbanised, did not show notable seasonal variations in flood elevations (see **Appendix D, Profile #1**). However, the Sackville River showed high sensitivity to seasonal changes, with close to a metre of difference in water levels, downstream of its confluence with the Little Sackville River. Development projections showed little influence, with an increase in the order of 100 mm in the downstream end of the Sackville River. Tidal effects (see **Appendix D, Profile #10 and Profile #23**), with and without climate change, were shown by the model to be limited to just upstream of the Bedford Highway.

The hydrologic model was calibrated on historic flow records. The results of the model are therefore consistent with the historical peak flows (e.g. the 1 in 100 year peak flow is calculated in the model to be a seasonal average of 38.5 m^3/s in the Little Sackville River, which compares to 26.3 m^3/s from the flow gauging data and 109.75 m^3/s in the Sackville River, which compares with 115 m^3/s estimated directly from the flow gauging data). Compared to the historical storm of March 2003, the water level results are slightly higher throughout the river system, which is consistent with the finding that the March 2003 event was less significant than a 1 in 100 year event.

Results of modelling rainfall impacted by climate change (see **Appendix D, Profile #14 to #22 and Profile #24 to #32**) were also generated. It was found that the large number of existing climate change models, combined with the various methods of transformation of the results into rainfall amounts, produced a wide range of results, with the highest rainfall amount calculated at 283.9 mm, a 70% increase compared to the existing 1 in 100 year rainfall amount (166.7 mm) and the lowest rainfall amount at 130mm, a 22% decrease. This wide range of results presents great challenges for selecting a value for climate change impacts. The selection should therefore be informed by the purpose of the value, i.e. in this case for planning purposes, and the notion of acceptable risk should be considered, as well as the impact of selecting one result over another. Interestingly, with the selection of the highest result in the range, while the water levels increased accordingly, the floodplain width did not significantly widen. This is mostly a result of the existing floodplain topography in which the floodplain edges have higher slopes, resulting in a small change of width when water levels increase. This provides some support for selecting this value.

The 1 in 500 year event results (see **Appendix E, Map #3**) showed larger flooding extents than the 1 in 100 year event, but again, to a limited extent. Since the total rainfall amount in 24 hours is 199 mm for the 1 in 500 year event, it is only marginally higher than the 1 in 100 year total rainfall amount (166.7 mm), and notably lower than the climate change amount (283.9 mm). Results are therefore much closer to the 1 in 100 year event than the worst case climate change scenario.

A discussion of potential flood mitigation options is presented in **Section 4.6**, and reviews the benefits and challenges associated with each potential measure. Although this assessment did not investigate in detail, nor model, any flood mitigation option, certain high level aspects can be drawn from the results. The flood line delineation showed that climate change impacts clearly have the potential to increase flooding risks and should be considered in any future planning decision. The planning regulations will be central to managing future development and it is recommended that they include language on setback limits, runoff control, flood proofing or limited uses (directed at recreational) in floodplain areas. Designating environmentally sensitive areas (e.g. Watercourse Greenbelt zoning in East Hants) is also recommended to prevent future development in water storage and undeveloped floodplain areas.

While the upper reaches of the Sackville River are mainly undeveloped, its lower reaches, and most of the Little Sackville River, are quite highly urbanized, which is both increasing river flows as well as creating vulnerabilities. The following list of factors have contributed to the prioritized recommendations noted below.

- Risks associated with climate change;
- Increased interest in sustainability;
- Increased awareness of liability;
- Increasing costs of maintenance, and
- General reduction in funding for infrastructure projects

Recommendations have been generally oriented towards more sustainable, low maintenance, more nature-oriented approaches, which provide not only solutions to flooding risks, but also additional advantages in terms of erosion protection, water quality improvements and overall aesthetics and protection/restoration of the natural character of the rivers. This is consistent with the Sackville Greenway Plan, the Halifax Regional Plan and the Halifax Green Network Plan (Greenbelting and Open Space Plan).

Recommendations for flood mitigation, beyond adopting the floodlines in this report into planning regulations, are the following:

1. **Stormwater Infiltration - Best Management Practices (BMPs) and Low Impact Development (LID):**
   The least intrusive and most cost-effective flood mitigation option is to implement stormwater infiltration measures (LID and BMPs). It is recommended that such measures be enforced for all future development (more effective than detention ponds) through planning regulations and during resurfacing or repair works. BMPs and LID can have a very low direct cost but make a clear impact in flood reduction, in a manner that mimics natural processes;
2. **Increasing channel capacity through river restoration**:
   Other recommended approaches include conducting river restoration to increase capacity and storage in river sections that have been channelized. Significant ecosystem benefits are also achieved;
3. **Purchasing properties at risk**:
   The impacted individuals are now permanently safe, properties at risk can be restored to the natural floodplain, upstream flooding risks can be reduced, there is no further maintenance cost or residual risk, and the riverfront area can now be enhanced for public enjoyment. The challenges are its cost and resistance from property owners. Where not yet developed, purchasing floodplain lands can ensure their protection in the future;
4. **Flood Protection Infrastructure**:
   Options such as upgrading bridge structures, building berms, or raising the level of the land or homes, should only be used after the above options have been exhausted. They will be expensive, require maintenance, will move the problem downstream and will place public safety at increased risk for events greater than the design event.

In all cases, stakeholder consultations and modelling should be carried out to identify the best compromise between protecting vulnerabilities, overall stakeholder needs, ecosystem protection and costs. The creation of a dedicated floodplain committee (possibly cross-municipal to include the Municipality of East Hants) with regular meetings can streamline this process.

Overall, this study has updated the current state of knowledge on rainfall, hydrologic (including seasonal) characteristics, river flow responses, impacts of structures and ice jams, mechanisms leading to flooding, potential climate change impacts and potential flood mitigation options. This study has brought very detailed data sets of high resolution and quality, combined with state-of-the-art modelling and analysis to inform the results and recommendations presented.

Recommendations to improve this analysis in the future would include conducting further flow gauging in various areas of the watershed, evaluating in more detail ground infiltration and exfiltration characteristics, being cognizant of the latest climate change research as it progresses, and trying to collect as much calibration data (water levels) as possible in the rivers during flood events.

In terms of recommended next steps for the HRM, the first goal of this study is to provide information to support an update to the planning regulations. An essential step, as noted by the HRM, is to make every effort to communicate the results and implications of this study and planning regulation to the public and all affected stakeholders, which is best achieved by using a wide range of approaches. Communication of flooding risks and emergency procedures, as well as flood proofing techniques, is also very valuable to help residents understand and deal with flooding risks. Warning systems, including flood forecasting and warning, can be very valuable tools to increase public safety. In terms of flood mitigation options, next steps will need to include conducting more detailed analyses and modelling of potential options. This can be done in parallel with an assessment of vulnerabilities along the river system, conducted through consultation with each of the relevant stakeholders. Vulnerabilities for land use, infrastructure and services can be obtained from stakeholders. Together with vulnerabilities in the management of emergency procedures (e.g. ensuring reliable communications or access to emergency services), these can be ranked by priority to define flood protection goals. How well each flood mitigation measures addresses each vulnerability can then be used to evaluate the efficiency of each flood protection measure.

Prepared By:

Jeff Marvin, P.Eng.
Water Resources Engineer
Phone: 902 421 7241 Ext 2536
Email: jeffmarvin@cbcl.ca

Léa Braschi, M.Sc., B.Sc.
Water Resources Scientist
Phone: 902 421 7241 Ext 2522
Email: lbraschi@cbcl.ca

Vincent Leys, M.Sc., P.Eng
Senior Coastal Engineer
Phone: 902 421 7241 Ext 2508
Email: vincentl@cbcl.ca

Reviewed By:

Alexander Wilson, M.Eng., P.Eng
Senior Water Resources Engineer
Phone: 902 421 7241 Ext 2502
Email: alexanderw@cbcl.ca

Andrew Gates, P.Eng
Vice President, Infrastrucure
Phone: 902 492 6755
email: andrewg@cbcl.ca

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APPENDIX A

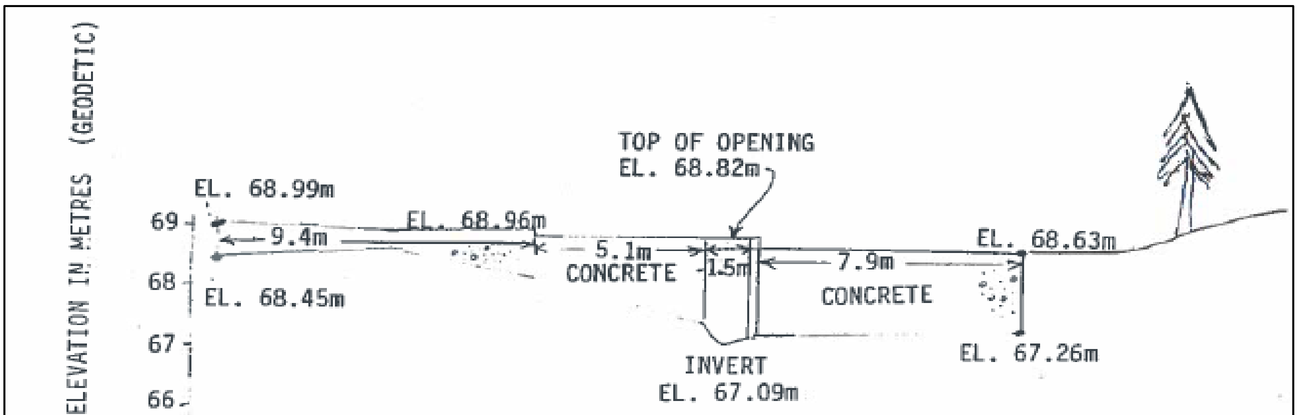
Hydraulic Structure Data Sheets

Structure Name:	Feely Lake Dam	Structure #: 1

Bridge ID: -
Structure Type: Dam with Rectangular Opening Containing Stop Logs
Overtopping Elevation (m): 68.82

Nominal Diameter (mm): -
Width (m): 1.5
Height (m): 1.73 (full height - stop log height varies)
Material: Timber
Length (m): -
Slope: -
Inlet Invert (m): 67.09
Outlet Invert (m): -

Photos/drawings/sketches:



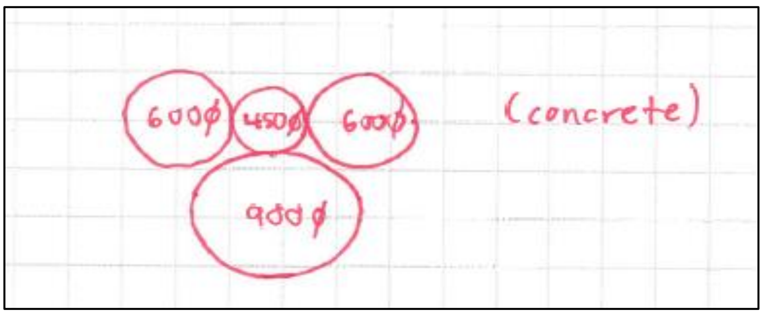
Source: Nolan Davis & Associates Limited. (1987). "Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	Barrett Culvert 1			Structure #: 2

Bridge ID:	-			
Structure Type:	Circular Culverts (x4)			
Overtopping Elevation (m):	65.62			

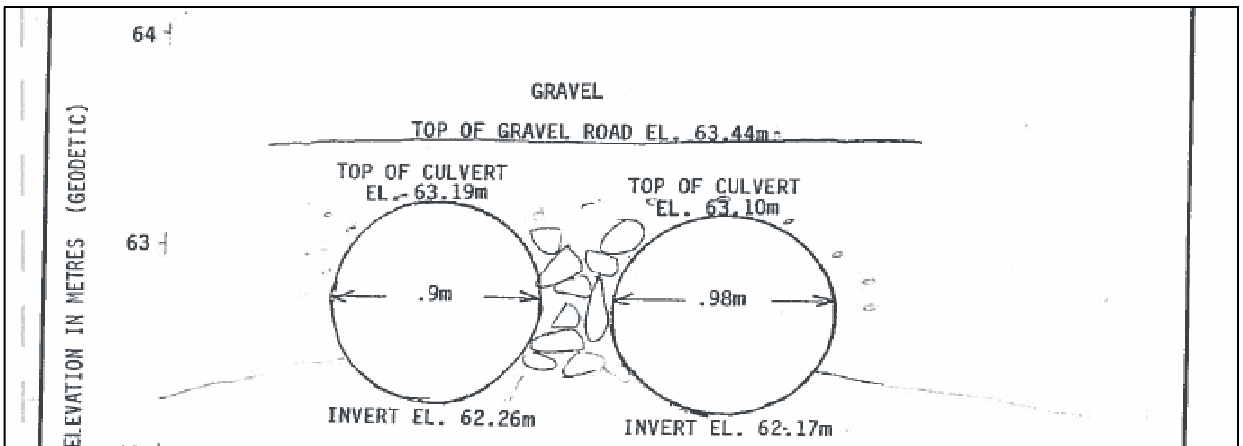
	Culvert 1	Culvert 2	Culvert 3	Culvert 4
Nominal Diameter (mm):	900	600	450	450
Width (m):	-	-	-	-
Height (m):	-	-	-	-
Material:	Concrete	Concrete	Concrete	Concrete
Length (m):	6	6	6	6
Slope:	0.8% (est.)	0.8% (est.)	0.8% (est.)	0.8% (est.)
Inlet Invert (m):	63.95 (est.)	64.75 (est.)	65.05 (est.)	65.05 (est.)
Outlet Invert (m):	63.90 (est.)	64.70 (est.)	65.00 (est.)	65.00 (est.)

Photos/drawings/sketches:



Structure Name:	Quarry Road Culvert 1		Structure #: 3
Bridge ID:	-		
Structure Type:	Circular Culverts (x2)		
Overtopping Elevation (m):	63.13		
	Culvert 1	Culvert 2	
Nominal Diameter (mm):	900	900	
Width (m):	0.9	0.98	
Height (m):	0.93	0.93	
Material:	Concrete	Concrete	
Length (m):	9	9	
Slope:	1.9% (est.)	0.9% (est.)	
Inlet Invert (m):	62.26	62.17	
Outlet Invert (m):	62.10 (est.)	62.10 (est.)	

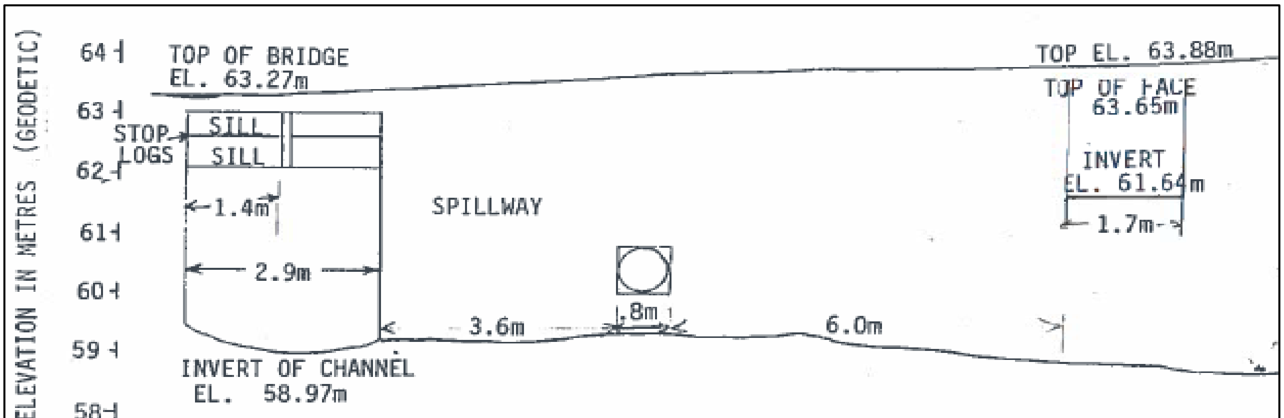
Photos/drawings/sketches:



Source: Nolan Davis & Associates Limited. (1987)."Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	**Lumber Mill Pond Dam**			**Structure #: 4**
Bridge ID:	-			
Structure Type:	Dam with Rectangular Openings (x3)			
Overtopping Elevation (m):	62.5			
	Opening 1	Opening 2	Opening 3	
Nominal Diameter (mm):	-	-	-	
Width (m):	2.4	2.4	1.7	
Height (m):	0.8	0.8	1.6	
Material:	Timber	Timber	Timber	
Length (m):	7	7	-	
Slope:	0% (est.)	0% (est.)	-	
Inlet Invert (m):	62.05 (est.)	62.05 (est.)	62.05 (est.)	
Outlet Invert (m):	62.05 (est.)	62.05 (est.)	-	

Photos/drawings/sketches:



Source: Nolan Davis & Associates Limited. (1987). "Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	Barrett Culvert 2			Structure #: 5
Bridge ID:	-			
Structure Type:	Circular Culverts (3)			
Overtopping Elevation (m):	60.35			
	Culvert 1	Culvert 2	Culvert 3	
Nominal Diameter (mm):	750	750	750	
Width (m):	-	-	-	
Height (m):	-	-	-	
Material:	Concrete	Concrete	Concrete	
Length (m):	15	15	15	
Slope:	2.5% (est.)	2.5% (est.)	3.2% (est.)	
Inlet Invert (m):	58.70 (est.)	58.70 (est.)	58.80 (est.)	
Outlet Invert (m):	58.32 (est.)	58.32 (est.)	58.32 (est.)	

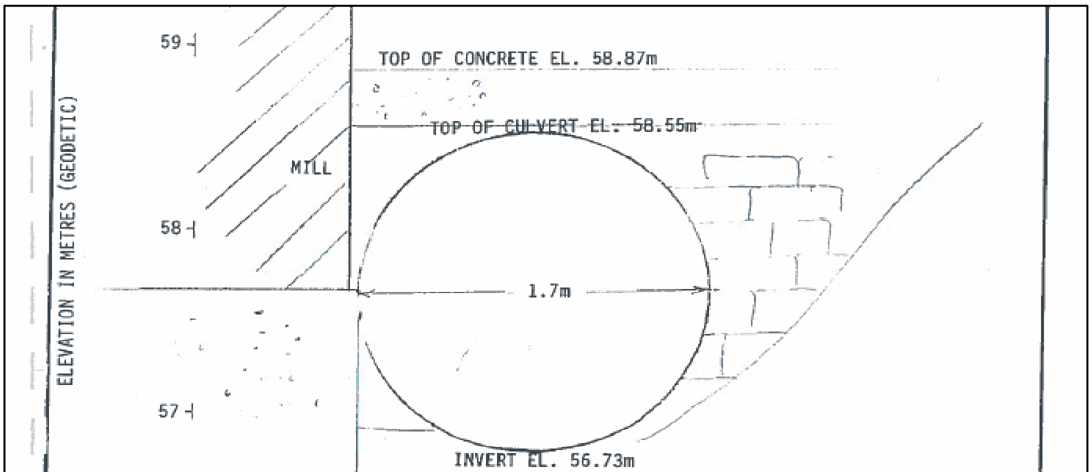
Photos/drawings/sketches:

Structure Name:	**Barrett Culvert 3**	**Structure #: 6**

Bridge ID: -
Structure Type: Circular Culvert
Overtopping Elevation (m): 58.62

Nominal Diameter (mm): 1800
Width (m): -
Height (m): -
Material: Concrete
Length (m): 60
Slope: 0.1% (est.)
Inlet Invert (m): 56.73
Outlet Invert (m): 56.65 (est.)

Photos/drawings/sketches:



Source: Nolan Davis & Associates Limited. (1987). "Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	Barrett Culvert 4	Structure #: 7

Bridge ID:	-
Structure Type:	Circular Culvert
Overtopping Elevation (m):	58.41
Nominal Diameter (mm):	2100
Width (m):	-
Height (m):	-
Material:	Concrete
Length (m):	10
Slope:	1.9% (est.)
Inlet Invert (m):	56.40
Outlet Invert (m):	56.21 (est.)

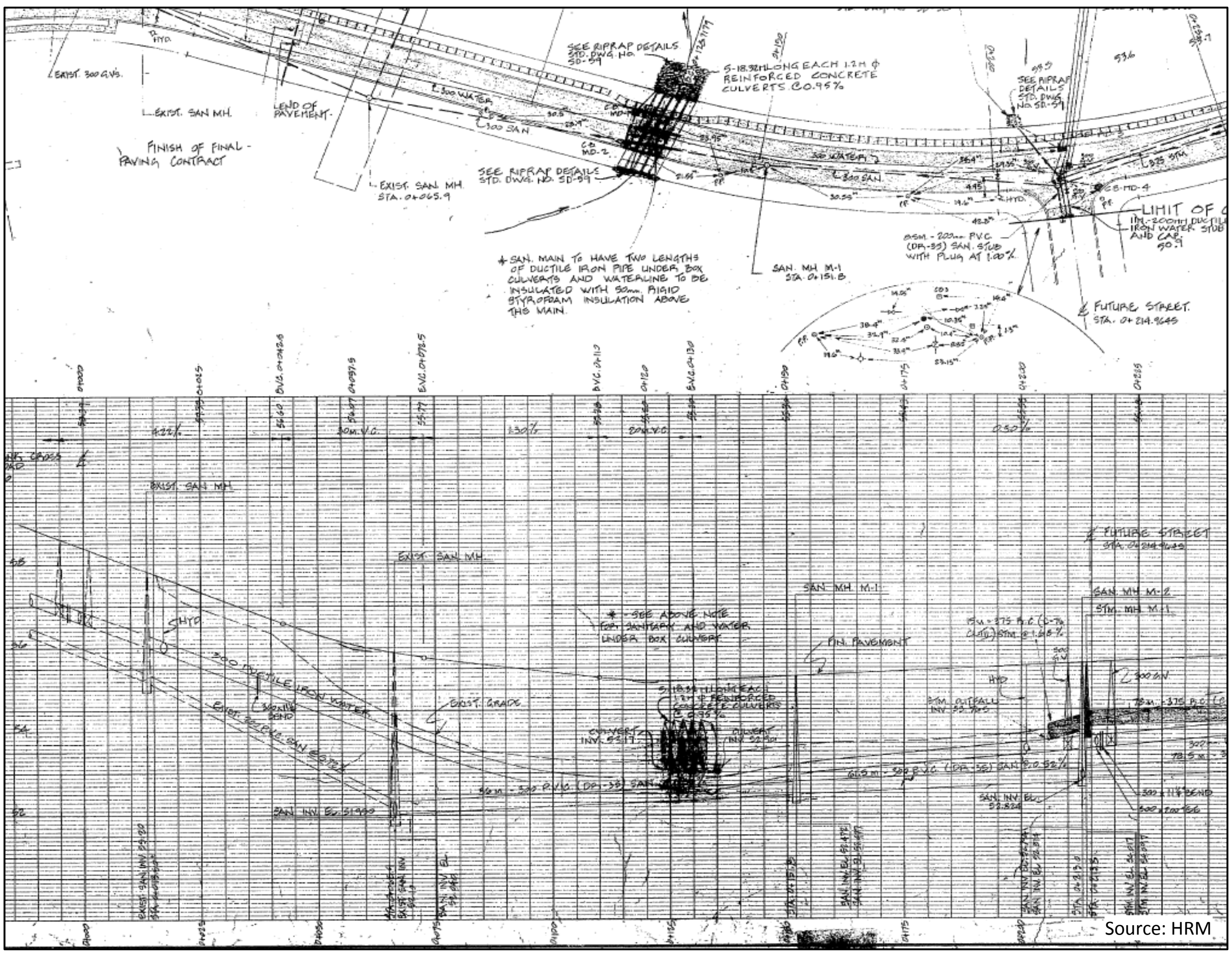
Photos/drawings/sketches:

Structure Name:	Millwood Dr ("Beaver Bank Cross Rd")	Structure #: 8

Bridge ID: 103
Structure Type: Circular Culvert (x5)
Overtopping Elevation (m): 55.09

	Culvert 1	Culvert 2	Culvert 3	Culvert 4	Culvert 5
Nominal Diameter (mm):	1200	1200	1200	1200	1200
Width (m):	-	-	-	-	-
Height (m):	-	-	-	-	-
Material:	Concrete	Concrete	Concrete	Concrete	Concrete
Length (m):	18	18	18	18	18
Slope:	1.0%	1.0%	0.01	1.0%	1.0%
Inlet Invert (m):	52.90	53.19	53.19	53.19	53.19
Outlet Invert (m):	52.90	53.00	52.996	53.00	53.00

Photos/drawings/sketches:



Source: HRM

Structure Name:	**Trail Bridge #1**	**Structure #: 9**

Bridge ID:	CRSS_ID 95
Structure Type:	Bridge
Overtopping Elevation (m):	53.08
Nominal Diameter (mm):	-
Width (m):	9.78 (est.)
Height (m):	1.51 (est.)
Material:	unknown
Length (m):	3
Slope:	0.3% (est.)
Inlet Invert (m):	52.29 (est.)
Outlet Invert (m):	52.28 (est.)

Photos/drawings/sketches:

Structure Name:	Trail Bridge #2	Structure #: 10

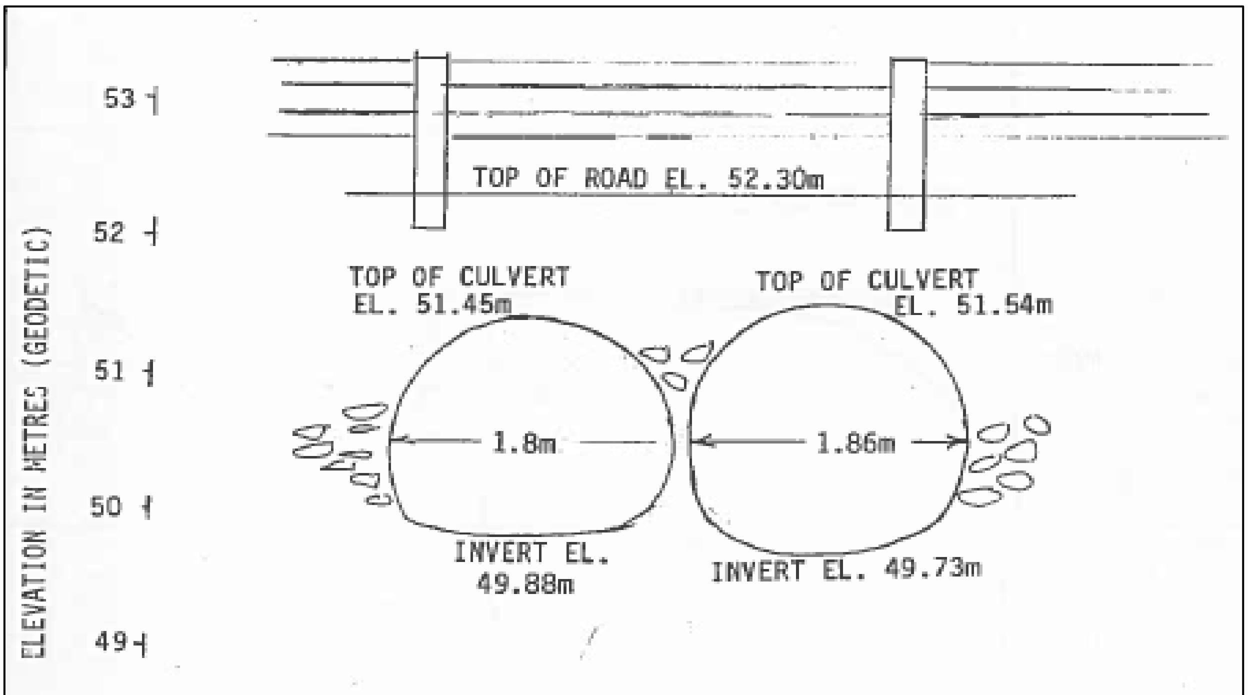
Bridge ID:	CRSS_ID 96
Structure Type:	Bridge
Overtopping Elevation (m):	52.08
Nominal Diameter (mm):	-
Width (m):	11.95 (est.)
Height (m):	2.4 (est.)
Material:	unknown
Length (m):	3
Slope:	0.3% (est.)
Inlet Invert (m):	50.93 (est.)
Outlet Invert (m):	50.92 (est.)

Photos/drawings/sketches:

Structure Name:	**Beaver Bank Cross Road Culvert**		**Structure #: 11**

Bridge ID:	61	
Structure Type:	Circular Culverts (x2)	
Overtopping Elevation (m):	52.956	
	Culvert 1	Culvert 2
Nominal Diameter (mm):	1800	1800
Width (m):	1.8	1.86
Height (m):	1.57	1.72
Material:	Concrete	Concrete
Length (m):	18	18
Slope:	0.5% (est.)	0.5% (est.)
Inlet Invert (m):	49.73	49.88
Outlet Invert (m):	49.63 (est.)	49.78 (est.)

Photos/drawings/sketches:

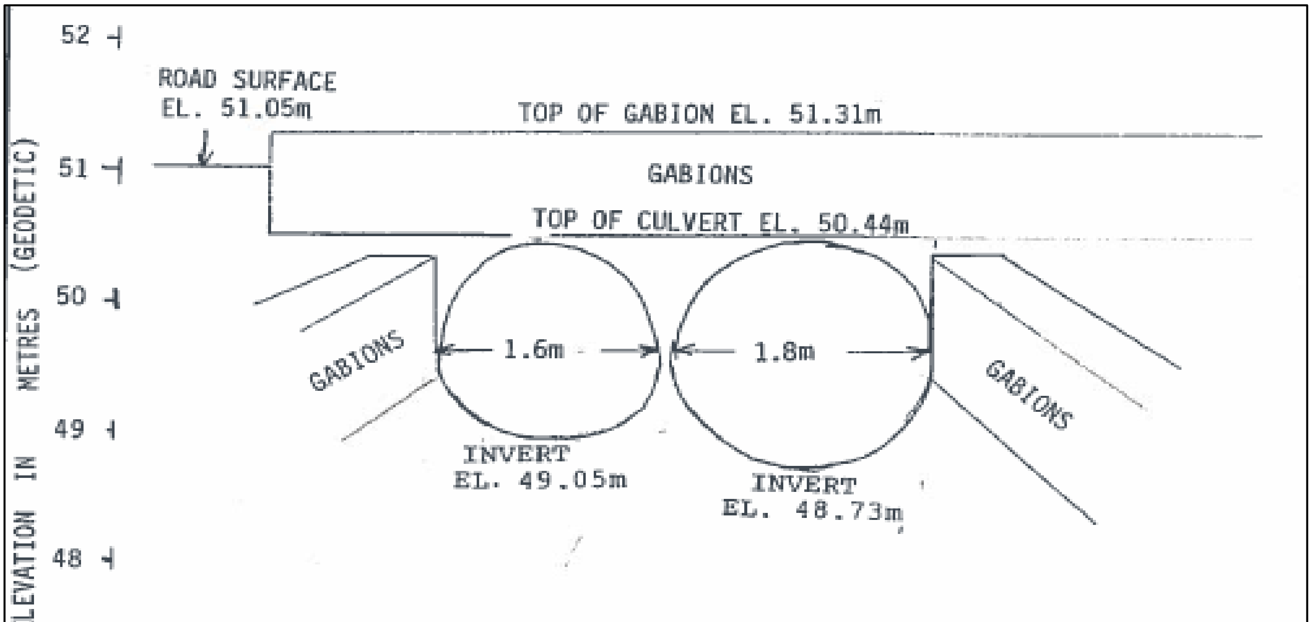


Source: Nolan Davis & Associated Limited. (1987). "Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	Gantry Road Culvert		Structure #: 12

Bridge ID:	-	
Structure Type:	Circular Culverts (x2)	
Overtopping Elevation (m):	50.59	
	Culvert 1	Culvert 2
Nominal Diameter (mm):	1500	1800
Width (m):	1.6	1.8
Height (m):	1.39	1.71
Material:	Concrete	Concrete
Length (m):	14	14
Slope:	0.7% (est.)	0.7% (est.)
Inlet Invert (m):	49.05	48.73
Outlet Invert (m):	48.95 (est.)	48.63 (est.)

Photos/drawings/sketches:

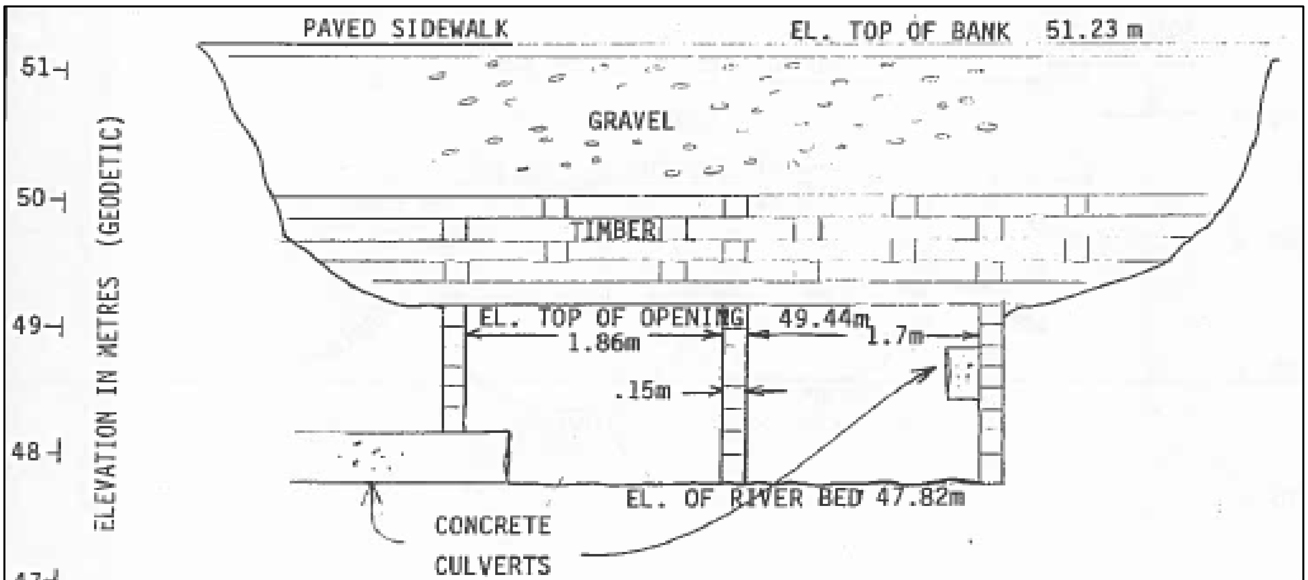


Source: Nolan Davis & Associated Limited. (1987). "Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	**Beaver Bank Road Culvert**		**Structure #: 13**
Bridge ID:	126		
Structure Type:	Box Culverts (x2)		
Overtopping Elevation (m):	50.71		

	Culvert 1	Culvert 2
Nominal Diameter (mm):	-	-
Width (m):	1.86	1.7
Height (m):	1.62	1.62
Material:	Timber	Timber
Length (m):	25	25
Slope:	0.4% (est.)	0.4% (est.)
Inlet Invert (m):	47.82	47.82
Outlet Invert (m):	47.72 (est.)	47.72 (est.)

Photos/drawings/sketches:



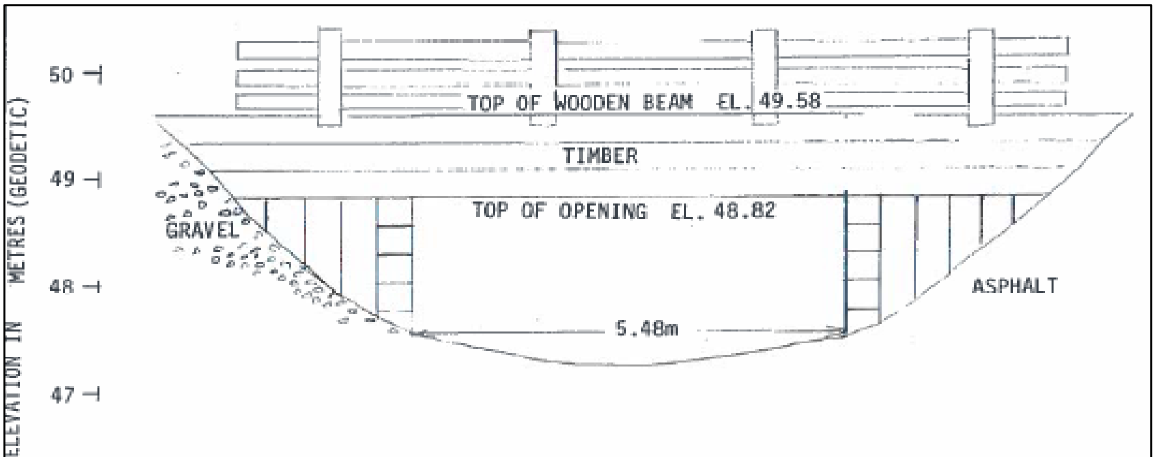
Source: Nolan Davis & Associated Limited. (1987). "Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	Old Beaver Bank Road Culvert	Structure #: 14

Bridge ID: 42
Structure Type: Bridge
Overtopping Elevation (m): 49.21

Nominal Diameter (mm): -
Width (m): 5.48
Height (m): 1.57 (est.)
Material: Timber
Length (m): 9
Slope: 0.5% (est.)
Inlet Invert (m): 49.25 (est.)
Outlet Invert (m): 49.20 (est.)

Photos/drawings/sketches:



Source: Nolan Davis & Associates Limited. (1987). "Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	Sackville Drive Culverts	Structure #: 15

Bridge ID: 118
Structure Type: Culverts (x2)
Overtopping Elevation (m): 40.11

	Culvert 1	Culvert 2
Nominal Diameter (mm):	(arch)	(arch)
Width (m):	3.3	3.4
Height (m):	3	2.04
Material:	CSP	CSP
Length (m):	40	40
Slope:	1.9%	1.4%
Inlet Invert (m):	36.76	37.06
Outlet Invert (m):	36.03	36.48

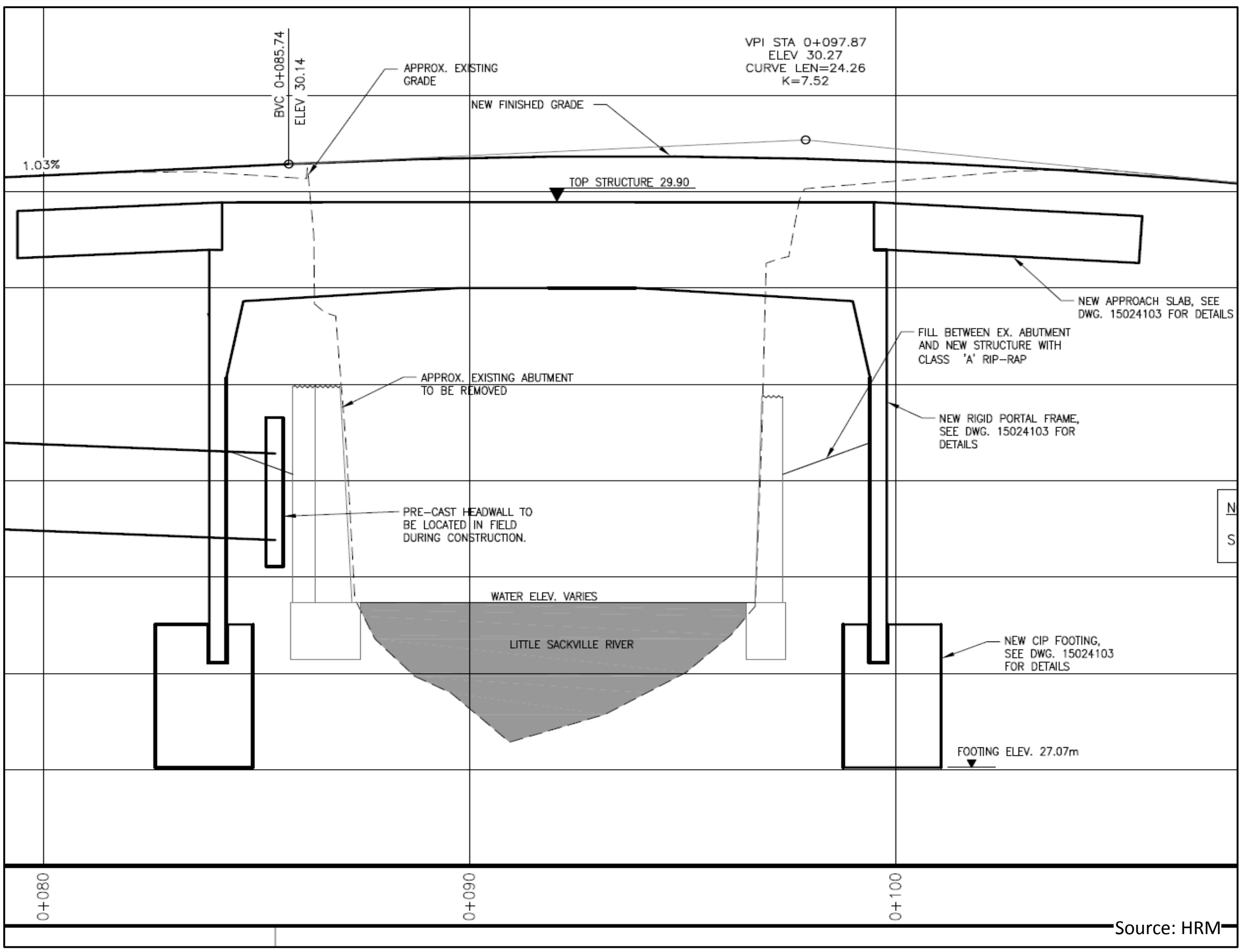
Photos/drawings/sketches:

Structure Name:	Sackville Cross Road Bridge	Structure #: 16

Bridge ID: 43
Structure Type: Bridge
Overtopping Elevation (m): 29.52

Nominal Diameter (mm): -
Width (m): 9.3
Height (m): 2.48
Material: Concrete
Length (m): 12
Slope: 0.8% (est.)
Inlet Invert (m): 26.94
Outlet Invert (m): 26.84 (est.)

Photos/drawings/sketches:

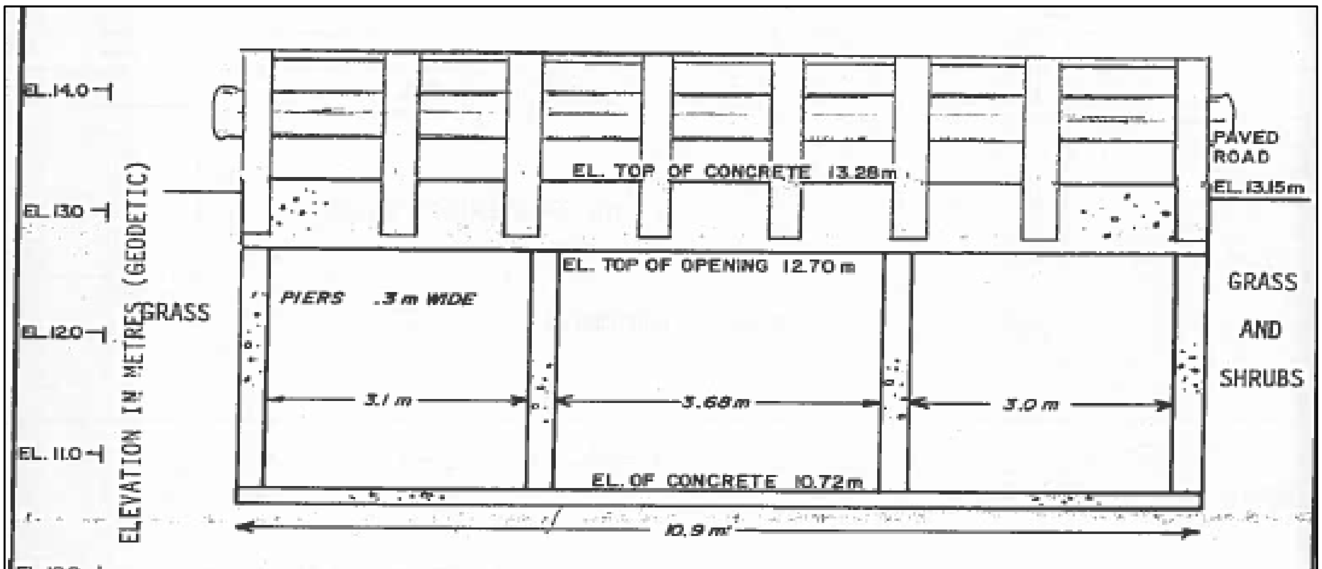


Source: HRM

Structure Name:	Highway 101 Bridge			Structure #: 17
Bridge ID:	40			
Structure Type:	Box Culverts (x3)			
Overtopping Elevation (m):	13.187			

	Culvert 1	Culvert 2	Culvert 3
Nominal Diameter (mm):	-	-	-
Width (m):	3.1	3.68	3
Height (m):	1.98	1.98	1.98
Material:	Concrete	Concrete	Concrete
Length (m):	46	46	46
Slope:	0.4% (est.)	0.4% (est.)	0.4% (est.)
Inlet Invert (m):	10.72	10.72	10.72
Outlet Invert (m):	10.52 (est.)	10.52 (est.)	10.52 (est.)

Photos/drawings/sketches:



Source: Nolan Davis & Associated Limited. (1987). "Hydrotechnical Study of the Little Sackville River Floodplain".

Structure Name:	Greenway Trail Bridge 1	Structure #: 18

Bridge ID:	-
Structure Type:	Bridge
Overtopping Elevation (m):	13.22
Nominal Diameter (mm):	-
Width (m):	15.5
Height (m):	1.54
Material:	unknown
Length (m):	3
Slope:	3.3% (est.)
Inlet Invert (m):	10.26
Outlet Invert (m):	10.16 (est.)

Photos/drawings/sketches:

Structure Name: **Lucasville Road Bridge** **Structure #: 19**

Bridge ID: 41
Structure Type: Bridge
Overtopping Elevation (m): 75.2

Nominal Diameter (mm): -
Width (m): 9.14
Height (m): 3.58
Material: Concrete
Length (m): 14
Slope: 2.7% (est.)
Inlet Invert (m): 69.70
Outlet Invert (m): 69.04 (est.)

Photos/drawings/sketches:

TABLE 2
SUMMARY OF STRUCTURE PROPERTIES

STRUCTURE NO.	DESCRIPTION	HEIGHT OR DIA. (m)	TOP WIDTH (m)	OPENING AREA (m^2)	INVERT (m)	TOP OF ROAD ELEV. (m)	LOW CHORD (m)	PIER WIDTH (m)	SIDE SLOPES
1	- Shore Drive - Wooden Bridge	3.96	16.15	** 57.32	-1.07	3.50	2.89	--	Vert.
2	- Railway Bridge - near Shore Drive	10.50	63.25	** 490.00	-1.40	10.10	9.10	2 @ 3.2	1:1
3	- Highway #1 - Steel Girder Bridge	3.66	11.89	43.48	1.72	6.90	5.38	--	Vert.
4	- At Plaza near Towers - Prestressed Beam & Slab	2.64	24.38	64.29	3.56	7.60	6.20	--	Vert.
5	- At Plaza near Consumers - Prestressed Beam & Slab	4.11	23.77	97.83	2.98	9.10	7.09	--	Vert.
6	- Hwy #1 (Bicentennial Dr.) - Concrete Beam & Slab	6.80	66.00	** 330.00	4.18	13.00	10.98	4.57	3:1
7	- Dept. of National Defence - Wooden Bridge	2.10	18.30	** 40.00	--	--	--	--	--
8	- Private Gravel Road - Wooden Bridge - Washed Out	--	--	--	--	--	--	--	--
9	- Lucasville Road - Wooden Beam - Concrete Abutments	3.58	9.14	32.75	69.70	74.10	73.28	--	Vert.
10	- Boulder Bridge - 10 csp - 24" Culverts	0.70	4.79	2.92	73.60	74.50	74.30	--	--

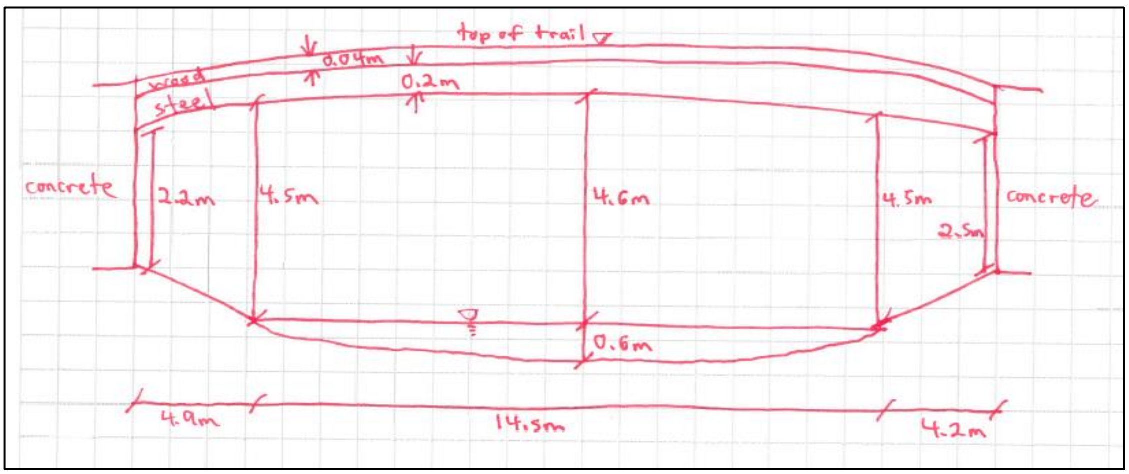
Source: Interprovincial Engineering Liimited. (1981). "Hydrotechnical Study of the Sackville River".

Structure Name:	Greenway Trail Bridge 2	Structure #: 20

Bridge ID: -
Structure Type: Bridge
Overtopping Elevation (m): 12.31

Nominal Diameter (mm): -
Width (m): 23.6
Height (m): 5.2
Material: Concrete
Length (m): 3
Slope: 0.5% (est.)
Inlet Invert (m): 7.18
Outlet Invert (m): 7.17 (est.)

Photos/drawings/sketches:

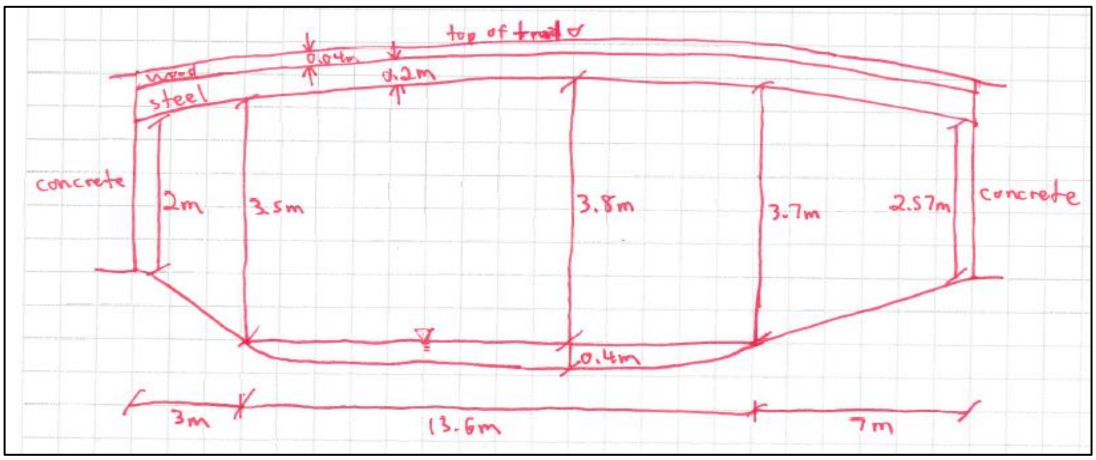


Structure Name:	Greenway Trail Bridge 3	Structure #: 21

Bridge ID: -
Structure Type: Bridge
Overtopping Elevation (m): 7.37

Nominal Diameter (mm): -
Width (m): 23.6
Height (m): 4.2
Material: Concrete
Length (m): 3
Slope: 0.3% (est.)
Inlet Invert (m): 5.74
Outlet Invert (m): 5.73 (est.)

Photos/drawings/sketches:

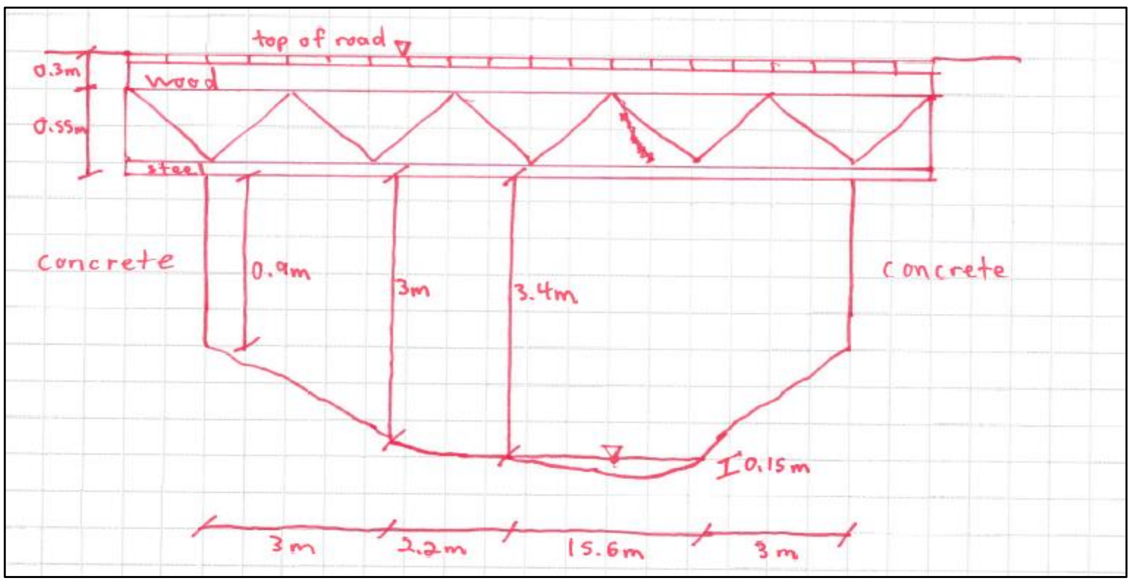


Structure Name:	Bedford Rifle Range Bridge	Structure #: 22

Bridge ID: -
Structure Type: Bridge
Overtopping Elevation (m): 6.82

Nominal Diameter (mm): -
Width (m): 23.8
Height (m): 3.55
Material: Concrete
Length (m): 4
Slope: 0.3% (est.)
Inlet Invert (m): 5.01
Outlet Invert (m): 5.00 (est.)

Photos/drawings/sketches:



Structure Name:	Highway 102 Bridge	Structure #: 23

Bridge ID:	-
Structure Type:	Bridge
Overtopping Elevation (m):	10.33
Nominal Diameter (mm):	-
Width (m):	37.7
Height (m):	7.13
Material:	Concrete
Length (m):	42
Slope:	0.2% (est.)
Inlet Invert (m):	3.75
Outlet Invert (m):	3.65 (est.)

Photos/drawings/sketches:

Source: GHD. (2016). "Sackville Rivers Floodplain Study: Phase I".

Structure Name:	Greenway Trail Bridge 4	Structure #: 24

Bridge ID: -
Structure Type: Bridge
Overtopping Elevation (m): 5.524

Nominal Diameter (mm): -
Width (m): 7.2
Height (m): 4.93
Material: unknown
Length (m): 2
Slope: 0.5% (est.)
Inlet Invert (m): 2.27 (est.)
Outlet Invert (m): 2.26

Photos/drawings/sketches:

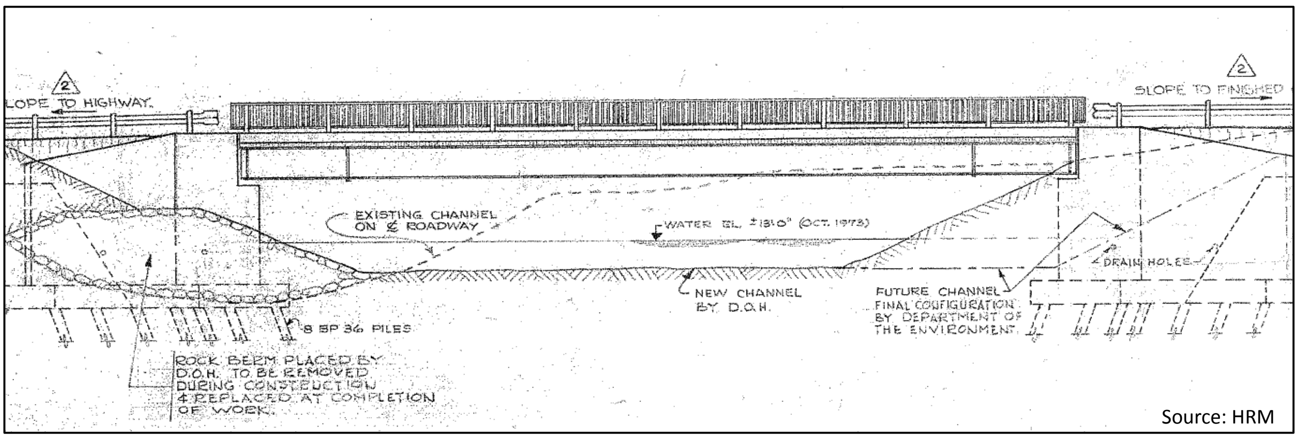

Source: GHD. (2016). "Sackville Rivers Floodplain Study: Phase I".

Structure Name:	River Lane Bridge ("Westgate Park Bridge")	Structure #: 25

Bridge ID: 144
Structure Type: Bridge
Overtopping Elevation (m): 7.00

Nominal Diameter (mm): -
Width (m): 23.7
Height (m): 4.88
Material: Concrete
Length (m): 9
Slope: 0.5% (est.)
Inlet Invert (m): 3.57
Outlet Invert (m): 3.52 (est.)

Photos/drawings/sketches:



Source: HRM


Source: GHD. (2016). "Sackville Rivers Floodplain Study: Phase I".

Structure Name:	**Bedford Place Mall Bridge 1**	**Structure #: 26**

Bridge ID: 148
Structure Type: Bridge
Overtopping Elevation (m): 7.23

Nominal Diameter (mm): -
Width (m): 24.5
Height (m): 3.34
Material: Concrete
Length (m): 9.5
Slope: 0.5% (est.)
Inlet Invert (m): 2.87
Outlet Invert (m): 2.82 (est.)

Photos/drawings/sketches:


Source: GHD. (2016). "Sackville Rivers Floodplain Study: Phase I".

Structure Name:	Bedford Place Mall Bridge 2	Structure #: 27

Bridge ID: 147
Structure Type: Bridge
Overtopping Elevation (m): 7.00

Nominal Diameter (mm): -
Width (m): 21.2
Height (m): 4.03
Material: Concrete
Length (m): 14
Slope: 0.7% (est.)
Inlet Invert (m): 2.96
Outlet Invert (m): 2.85 (est.)

Photos/drawings/sketches:

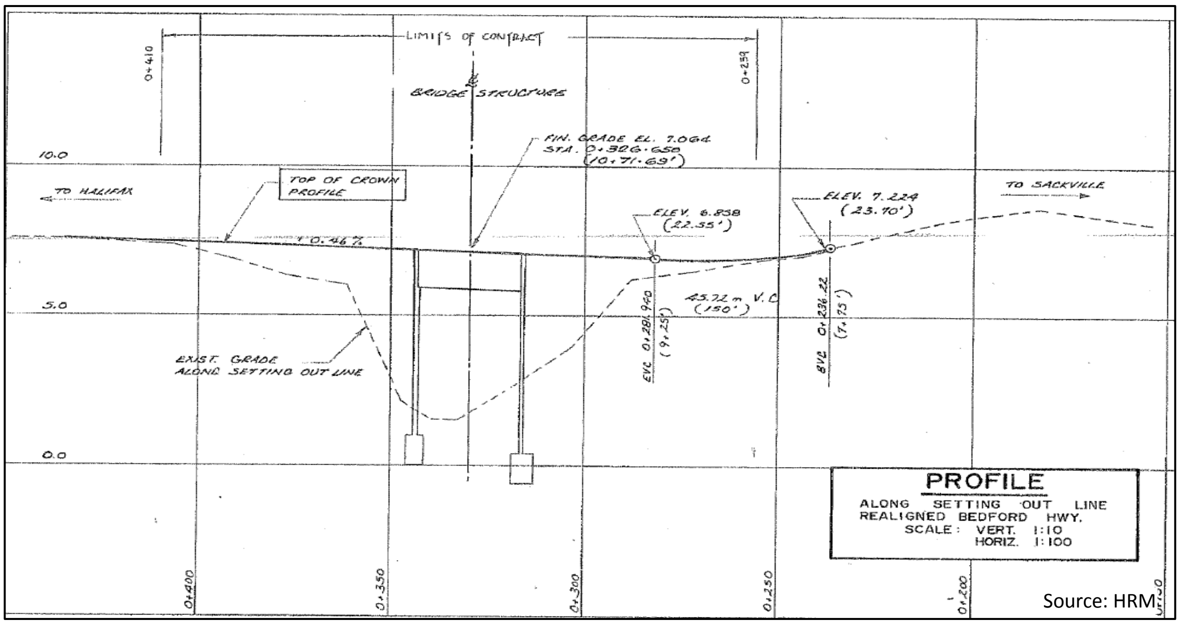

Source: GHD. (2016). "Sackville Rivers Floodplain Study: Phase I".

Structure Name:	Bedford Highway Bridge	Structure #: 28

Bridge ID: 15
Structure Type: Bridge
Overtopping Elevation (m): 7.00

Nominal Diameter (mm): -
Width (m): 14.3
Height (m): 4.64
Material: Concrete
Length (m): 32
Slope: 0.7%
Inlet Invert (m): 0.91 (est.)
Outlet Invert (m): 0.70

Photos/drawings/sketches:



Source: HRM


Source: GHD. (2016). "Sackville Rivers Floodplain Study: Phase I".

Structure Name:	Railway Bridge	Structure #: 29

Bridge ID: -
Structure Type: Bridge
Overtopping Elevation (m): 13.10

Nominal Diameter (mm): -
Width (m): 71.6
Height (m): 11.2
Material: Concrete
Length (m): 5
Slope: 7.0%
Inlet Invert (m): -0.80
Outlet Invert (m): -1.15

Photos/drawings/sketches:

Source: GHD. (2016). "Sackville Rivers Floodplain Study: Phase I".

Structure Name:	Shore Drive Bridge	Structure #: 30

Bridge ID:	-
Structure Type:	Bridge
Overtopping Elevation (m):	2.97
Nominal Diameter (mm):	-
Width (m):	17.3
Height (m):	4.28
Material:	Concrete
Length (m):	9.5
Slope:	7.4%
Inlet Invert (m):	-1.21
Outlet Invert (m):	-1.91

Photos/drawings/sketches:


Source: GHD. (2016). "Sackville Rivers Floodplain Study: Phase I".

APPENDIX B

Watershed Characteristics Tables

Table B-1: Estimated Watershed Characteristics for the Sackville River (Pre-Calibration) (Existing Development Conditions)

Sub-Watershed Name	Area (ha)	Slope (%)	Maximum Overland Flow Length (m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)	Percent of Area Routed to Pervious Area (%)
SR-01	6,921	0.4	16,700	18.7	0.028	0.334	106.4	1.8	65
SR-02	357	1.5	3,341	23.9	0.027	0.300	102.0	1.9	57
SR-03	387	2.0	2,552	34.2	0.030	0.296	87.0	1.6	56
SR-04	1,465	1.0	7,810	26.2	0.024	0.207	93.9	1.8	41
SR-05	205	2.3	2,219	29.3	0.025	0.221	78.5	1.4	42
SR-06	498	2.3	3,502	22.4	0.025	0.262	141.8	1.4	49
SR-07	57	3.4	1,395	38.0	0.024	0.249	220.0	0.8	41
SR-08	179	3.5	1,862	23.6	0.028	0.334	145.4	1.4	61
SR-09	60	4.6	903	26.6	0.027	0.324	219.6	0.8	60
SR-10	2,401	0.8	9,333	21.5	0.027	0.276	100.0	1.9	53
SR-11	109	2.1	2,202	19.0	0.029	0.377	130.2	1.9	72
SR-12	60	2.3	1,739	24.5	0.028	0.334	219.6	0.8	63
SR-13	41	1.5	1,693	20.6	0.029	0.286	214.6	0.8	53
SR-14	84	3.6	1,701	23.0	0.028	0.352	184.2	1.2	64
SR-15	195	2.7	2,681	53.2	0.022	0.195	113.8	1.6	25
SR-16	4.0	4.5	222	19.4	0.029	0.353	171.3	1.4	65
SR-17	1.8	4.2	142	46.1	0.021	0.178	110.0	2.2	26
SR-18	12	2.5	726	15.3	0.030	0.306	208.9	0.9	59
SR-19	3.3	3.0	339	47.3	0.021	0.155	110.0	2.2	22
SR-20	135	4.4	1,908	16.4	0.030	0.381	144.0	1.7	71
SR-21	17	2.7	600	36.8	0.024	0.199	115.3	2.0	33
SR-22	9.8	6.9	524	50.2	0.020	0.151	99.5	1.9	20
SR-23	83	3.7	1,663	42.6	0.023	0.222	145.7	1.7	35
SR-24	0.4	10.8	58	68.3	0.020	0.150	88.9	1.7	11
SR-25	0.9	10.8	277	54.7	0.020	0.150	164.0	1.1	18
SR-26	2.7	4.7	172	69.9	0.020	0.150	89.7	1.7	10
SR-27	36	4.7	1,057	50.5	0.020	0.150	171.6	1.3	20
SR-28	2.1	10.1	111	70.0	0.020	0.150	95.0	1.8	10
SR-29	5.1	10.1	345	60.7	0.020	0.150	143.9	1.5	15
SR-30	66	5.3	2,390	53.0	0.020	0.150	109.0	2.2	18
SR-31	36	2.5	1,302	57.7	0.020	0.150	125.8	1.9	17
SR-32	13	7.1	740	50.2	0.020	0.150	110.0	2.2	20
SR-33	1.7	5.9	185	69.8	0.020	0.150	110.0	2.2	10

Table B-2: Estimated Watershed Characteristics for the Little Sackville River (Pre-Calibration) (Existing Development Conditions)

Sub-Watershed Name	Area (ha)	Slope (%)	Maximum Overland Flow Length (m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)	Percent of Area Routed to Pervious Area (%)
LSR-01	158	1.4	2,045	17.6	0.030	0.381	172.6	1.0	72
LSR-02	88	2.8	1,188	25.8	0.030	0.348	124.9	1.4	64
LSR-03	3.6	4.8	231	40.0	0.023	0.222	110.0	2.2	36
LSR-04	44	3.3	815	39.9	0.023	0.222	187.1	1.2	36
LSR-05	7.0	5.7	274	42.9	0.022	0.201	110.0	2.2	31
LSR-06	6.7	3.3	602	51.1	0.020	0.147	133.3	1.9	20
LSR-07	3.6	5.8	360	54.8	0.021	0.139	110.0	2.2	18
LSR-08	2.9	2.8	400	60.9	0.022	0.124	110.0	2.2	16
LSR-09	0.0	3.5	17	50.0	0.020	0.150	110.0	2.2	20
LSR-10	0.6	6.2	224	50.0	0.020	0.150	110.0	2.2	20
LSR-11	0.8	1.9	160	50.0	0.020	0.150	110.0	2.2	20
LSR-12	2.8	6.2	211	50.0	0.020	0.150	110.0	2.2	20
LSR-13	41	0.8	819	50.0	0.020	0.150	148.4	1.7	20
LSR-14	47	3.3	1,282	37.5	0.024	0.239	123.5	1.8	41
LSR-15	11	3.6	564	39.0	0.023	0.229	111.3	2.2	44
LSR-16	251	2.9	3,022	25.4	0.025	0.263	188.5	1.0	48
LSR-17	15	2.8	677	49.0	0.020	0.157	110.0	2.2	22
LSR-18	59	4.0	1,694	44.3	0.021	0.163	182.8	0.9	25
LSR-19	4.2	5.1	138	41.8	0.022	0.208	104.2	2.1	38
LSR-20	131	4.9	1,706	44.3	0.021	0.180	195.3	0.7	28
LSR-21	13	6.2	565	47.8	0.021	0.166	95.3	1.9	25
LSR-22	4.0	5.8	344	50.0	0.020	0.150	186.0	1.2	20
LSR-23	1.4	3.2	124	50.0	0.020	0.150	97.8	1.9	20
LSR-24	23	5.6	402	50.0	0.020	0.150	197.3	1.1	20
LSR-25	41	6.2	1,084	50.0	0.020	0.150	179.7	1.3	20
LSR-26	6.4	3.7	229	50.0	0.020	0.150	172.2	1.4	20
LSR-27	1.8	3.1	385	50.0	0.020	0.150	110.0	2.2	20
LSR-28	24	3.9	367	54.3	0.020	0.150	172.7	1.4	18
LSR-29	95	4.9	1,304	50.2	0.020	0.150	199.3	1.0	20
LSR-30	146	2.4	1,685	57.2	0.020	0.150	220.0	0.8	16
LSR-31	9.0	3.0	233	51.5	0.020	0.150	216.8	0.8	19
LSR-32	111	2.4	2,500	49.3	0.020	0.155	208.1	0.9	21
LSR-33	192	4.1	1,420	54.7	0.020	0.150	181.6	1.3	18

Table B-3: Estimated Watershed Characteristics for the Sackville River (Pre-Calibration) (Future Development Conditions)

Sub-Watershed Name	Area (ha)	Slope (%)	Maximum Overland Flow Length (m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)	Percent of Area Routed to Pervious Area (%)
SR-01	6,921	0.4	16,700	21.9	0.028	0.316	106.4	1.8	61
SR-02	357	1.5	3,341	40.2	0.023	0.190	102.0	1.9	32
SR-03	387	2.0	2,552	61.7	0.022	0.122	87.0	1.6	15
SR-04	1,465	1.0	7,810	39.1	0.022	0.153	93.9	1.8	26
SR-05	205	2.3	2,219	31.0	0.024	0.209	78.5	1.4	39
SR-06	498	2.3	3,502	31.6	0.023	0.217	141.8	1.4	39
SR-07	57	3.4	1,395	41.5	0.023	0.229	220.0	0.8	36
SR-08	179	3.5	1,862	30.9	0.025	0.256	145.4	1.4	45
SR-09	60	4.6	903	29.7	0.026	0.285	219.6	0.8	52
SR-10	2,401	0.8	9,333	25.5	0.026	0.275	100.0	1.9	51
SR-11	109	2.1	2,202	15.8	0.030	0.381	130.2	1.9	74
SR-12	60	2.3	1,739	16.1	0.030	0.360	219.6	0.8	70
SR-13	41	1.5	1,693	24.6	0.027	0.320	214.6	0.8	58
SR-14	84	3.6	1,701	15.4	0.030	0.389	184.2	1.2	73
SR-15	195	2.7	2,681	52.9	0.022	0.195	113.8	1.6	25
SR-16	4.0	4.5	222	19.4	0.029	0.353	171.3	1.4	65
SR-17	1.8	4.2	142	46.1	0.021	0.178	110.0	2.2	26
SR-18	12	2.5	726	15.3	0.030	0.306	208.9	0.9	59
SR-19	3.3	3.0	339	47.3	0.021	0.154	110.0	2.2	22
SR-20	135	4.4	1,908	16.4	0.030	0.381	144.0	1.7	71
SR-21	17	2.7	600	36.8	0.024	0.199	115.3	2.0	33
SR-22	9.8	6.9	524	50.2	0.020	0.151	99.5	1.9	20
SR-23	83	3.7	1,663	42.6	0.023	0.222	145.7	1.7	35
SR-24	0.4	10.8	58	68.3	0.020	0.150	88.9	1.7	11
SR-25	0.9	10.8	277	54.7	0.020	0.150	164.0	1.1	18
SR-26	2.7	4.7	172	69.9	0.020	0.150	89.7	1.7	10
SR-27	36	4.7	1,057	50.5	0.020	0.150	171.6	1.3	20
SR-28	2.1	10.1	111	70.0	0.020	0.150	95.0	1.8	10
SR-29	5.1	10.1	345	60.7	0.020	0.150	143.9	1.5	15
SR-30	66	5.3	2,390	53.0	0.020	0.150	109.0	2.2	18
SR-31	36	2.5	1,302	57.7	0.020	0.150	125.8	1.9	17
SR-32	13	7.1	740	50.2	0.020	0.150	110.0	2.2	20
SR-33	1.7	5.9	185	69.6	0.020	0.150	110.0	2.2	10

Table B-4: Estimated Watershed Characteristics for the Little Sackville River (Pre-Calibration) (Future Development Conditions)

Sub-Watershed Name	Area (ha)	Slope (%)	Maximum Overland Flow Length (m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)	Percent of Area Routed to Pervious Area (%)
LSR-01	158	1.4	2,045	17.8	0.029	0.377	172.6	1.0	71
LSR-02	88	2.8	1,188	26.2	0.030	0.349	124.9	1.4	65
LSR-03	3.6	4.8	231	40.0	0.023	0.222	110.0	2.2	36
LSR-04	44	3.3	815	39.9	0.023	0.222	187.1	1.2	36
LSR-05	7.0	5.7	274	42.9	0.022	0.201	110.0	2.2	31
LSR-06	6.7	3.3	602	50.8	0.020	0.148	133.3	1.9	20
LSR-07	3.6	5.8	360	54.8	0.021	0.139	110.0	2.2	18
LSR-08	2.9	2.8	400	60.9	0.022	0.124	110.0	2.2	16
LSR-09	0.0	3.5	17	50.0	0.020	0.150	110.0	2.2	20
LSR-10	0.6	6.2	224	50.0	0.020	0.150	110.0	2.2	20
LSR-11	0.8	1.9	160	50.0	0.020	0.150	110.0	2.2	20
LSR-12	2.8	6.2	211	50.0	0.020	0.150	110.0	2.2	20
LSR-13	41	0.8	819	50.0	0.020	0.150	148.4	1.7	20
LSR-14	47	3.3	1,282	37.5	0.024	0.239	123.5	1.8	41
LSR-15	11	3.6	564	39.0	0.023	0.229	111.3	2.2	44
LSR-16	251	2.9	3,022	26.1	0.024	0.261	188.5	1.0	47
LSR-17	15	2.8	677	49.0	0.020	0.157	110.0	2.2	22
LSR-18	59	4.0	1,694	44.6	0.021	0.163	182.8	0.9	25
LSR-19	4.2	5.1	138	41.8	0.022	0.208	104.2	2.1	38
LSR-20	131	4.9	1,706	44.6	0.021	0.178	195.3	0.7	28
LSR-21	13	6.2	565	47.8	0.021	0.166	95.3	1.9	25
LSR-22	4.0	5.8	344	50.0	0.020	0.150	186.0	1.2	20
LSR-23	1.4	3.2	124	50.0	0.020	0.150	97.8	1.9	20
LSR-24	23	5.6	402	50.0	0.020	0.150	197.3	1.1	20
LSR-25	41	6.2	1,084	50.0	0.020	0.150	179.7	1.3	20
LSR-26	6.4	3.7	229	50.0	0.020	0.150	172.2	1.4	20
LSR-27	1.8	3.1	385	50.0	0.020	0.150	110.0	2.2	20
LSR-28	24	3.9	367	54.3	0.020	0.150	172.7	1.4	18
LSR-29	95	4.9	1,304	50.2	0.020	0.150	199.3	1.0	20
LSR-30	146	2.4	1,685	57.2	0.020	0.150	220.0	0.8	16
LSR-31	9.0	3.0	233	51.5	0.020	0.150	216.8	0.8	19
LSR-32	111	2.4	2,500	49.3	0.020	0.155	208.1	0.9	21
LSR-33	192	4.1	1,420	54.6	0.020	0.150	181.6	1.3	18

Table B-5: Estimated Watershed Characteristics for the Sackville River (Post-Calibration) (Winter Conditions) (Existing Development Conditions)

Sub-Watershed Name	Area (ha)	Slope (%)	Maximum Overland Flow Length (m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)	Percent of Area Routed to Pervious Area (%)
SR-01	6,921	0.4	16,700	91.9	0.015	0.334	106.4	1.8	65
SR-02	357	1.5	3,341	92.4	0.015	0.300	102.0	1.9	57
SR-03	387	2.0	2,552	93.4	0.015	0.296	87.0	1.6	56
SR-04	1,465	1.0	7,810	92.6	0.010	0.207	93.9	1.8	41
SR-05	205	2.3	2,219	92.9	0.010	0.221	78.5	1.4	42
SR-06	498	2.3	3,502	92.2	0.015	0.262	141.8	1.4	49
SR-07	57	3.4	1,395	93.8	0.010	0.249	220.0	0.8	41
SR-08	179	3.5	1,862	92.4	0.015	0.334	145.4	1.4	61
SR-09	60	4.6	903	92.7	0.015	0.324	219.6	0.8	60
SR-10	2,401	0.8	9,333	92.2	0.015	0.276	100.0	1.9	53
SR-11	109	2.1	2,202	91.9	0.015	0.377	130.2	1.9	72
SR-12	60	2.3	1,739	92.5	0.015	0.334	219.6	0.8	63
SR-13	41	1.5	1,693	92.1	0.015	0.286	214.6	0.8	53
SR-14	84	3.6	1,701	92.3	0.015	0.352	184.2	1.2	64
SR-15	195	2.7	2,681	95.3	0.010	0.195	113.8	1.6	25
SR-16	4.0	4.5	222	91.9	0.015	0.353	171.3	1.4	65
SR-17	1.8	4.2	142	94.6	0.010	0.178	110.0	2.2	26
SR-18	12	2.5	726	91.5	0.015	0.306	208.9	0.9	59
SR-19	3.3	3.0	339	94.7	0.010	0.155	110.0	2.2	22
SR-20	135	4.4	1,908	91.6	0.015	0.381	144.0	1.7	71
SR-21	17	2.7	600	93.7	0.010	0.199	115.3	2.0	33
SR-22	9.8	6.9	524	95.0	0.010	0.151	99.5	1.9	20
SR-23	83	3.7	1,663	94.3	0.010	0.222	145.7	1.7	35
SR-24	0.4	10.8	58	96.8	0.010	0.150	88.9	1.7	11
SR-25	0.9	10.8	277	95.5	0.010	0.150	164.0	1.1	18
SR-26	2.7	4.7	172	97.0	0.010	0.150	89.7	1.7	10
SR-27	36	4.7	1,057	95.1	0.010	0.150	171.6	1.3	20
SR-28	2.1	10.1	111	97.0	0.010	0.150	95.0	1.8	10
SR-29	5.1	10.1	345	96.1	0.010	0.150	143.9	1.5	15
SR-30	66	5.3	2,390	95.3	0.010	0.150	109.0	2.2	18
SR-31	36	2.5	1,302	95.8	0.010	0.150	125.8	1.9	17
SR-32	13	7.1	740	95.0	0.010	0.150	110.0	2.2	20
SR-33	1.7	5.9	185	97.0	0.010	0.150	110.0	2.2	10

Table B-6: **Estimated Watershed Characteristics for the Little Sackville River (Post-Calibration) (Fall Conditions) (Existing Development Conditions)**

Sub-Watershed Name	Area (ha)	Slope (%)	Maximum Overland Flow Length (m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)	Percent of Area Routed to Pervious Area (%)
LSR-01	158	1.4	2,045	91.8	0.051	0.381	172.6	1.0	72
LSR-02	88	2.8	1,188	92.6	0.051	0.348	124.9	1.4	64
LSR-03	3.6	4.8	231	94.0	0.034	0.222	110.0	2.2	36
LSR-04	44	3.3	815	94.0	0.034	0.222	187.1	1.2	36
LSR-05	7.0	5.7	274	94.3	0.034	0.201	110.0	2.2	31
LSR-06	6.7	3.3	602	95.1	0.034	0.147	133.3	1.9	20
LSR-07	3.6	5.8	360	95.5	0.034	0.139	110.0	2.2	18
LSR-08	2.9	2.8	400	96.1	0.034	0.124	110.0	2.2	16
LSR-09	0.0	3.5	17	95.0	0.034	0.150	110.0	2.2	20
LSR-10	0.6	6.2	224	95.0	0.034	0.150	110.0	2.2	20
LSR-11	0.8	1.9	160	95.0	0.034	0.150	110.0	2.2	20
LSR-12	2.8	6.2	211	95.0	0.034	0.150	110.0	2.2	20
LSR-13	41	0.8	819	95.0	0.034	0.150	148.4	1.7	20
LSR-14	47	3.3	1,282	93.8	0.034	0.239	123.5	1.8	41
LSR-15	11	3.6	564	93.9	0.034	0.229	111.3	2.2	44
LSR-16	251	2.9	3,022	92.5	0.034	0.263	188.5	1.0	48
LSR-17	15	2.8	677	94.9	0.034	0.157	110.0	2.2	22
LSR-18	59	4.0	1,694	94.4	0.034	0.163	182.8	0.9	25
LSR-19	4.2	5.1	138	94.2	0.034	0.208	104.2	2.1	38
LSR-20	131	4.9	1,706	94.4	0.034	0.180	195.3	0.7	28
LSR-21	13	6.2	565	94.8	0.034	0.166	95.3	1.9	25
LSR-22	4.0	5.8	344	95.0	0.034	0.150	186.0	1.2	20
LSR-23	1.4	3.2	124	95.0	0.034	0.150	97.8	1.9	20
LSR-24	23	5.6	402	95.0	0.034	0.150	197.3	1.1	20
LSR-25	41	6.2	1,084	95.0	0.034	0.150	179.7	1.3	20
LSR-26	6.4	3.7	229	95.0	0.034	0.150	172.2	1.4	20
LSR-27	1.8	3.1	385	95.0	0.034	0.150	110.0	2.2	20
LSR-28	24	3.9	367	95.4	0.034	0.150	172.7	1.4	18
LSR-29	95	4.9	1,304	95.0	0.034	0.150	199.3	1.0	20
LSR-30	146	2.4	1,685	95.7	0.034	0.150	220.0	0.8	16
LSR-31	9.0	3.0	233	95.2	0.034	0.150	216.8	0.8	19
LSR-32	111	2.4	2,500	94.9	0.034	0.155	208.1	0.9	21
LSR-33	192	4.1	1,420	95.5	0.034	0.150	181.6	1.3	18

Table B-7: Estimated Watershed Characteristics for the Sackville River (Post-Calibration) (Winter Conditions) (Future Development Conditions)

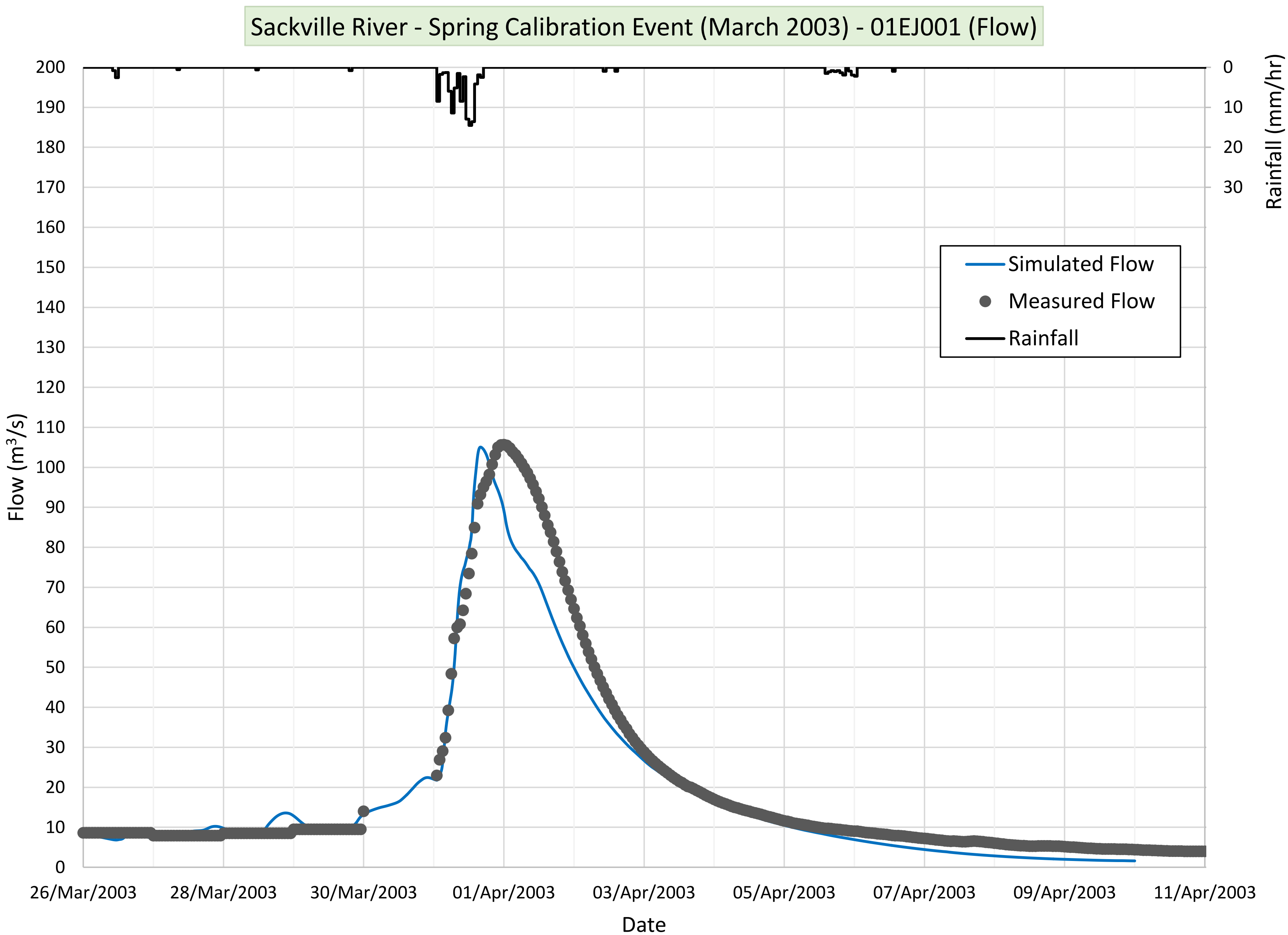
Sub-Watershed Name	Area (ha)	Slope (%)	Maximum Overland Flow Length (m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)	Percent of Area Routed to Pervious Area (%)
SR-01	6,921	0.4	16,700	92.2	0.010	0.316	106.4	1.8	61
SR-02	357	1.5	3,341	94.0	0.010	0.190	102.0	1.9	32
SR-03	387	2.0	2,552	96.2	0.010	0.122	87.0	1.6	15
SR-04	1,465	1.0	7,810	93.9	0.010	0.153	93.9	1.8	26
SR-05	205	2.3	2,219	93.1	0.010	0.209	78.5	1.4	39
SR-06	498	2.3	3,502	93.2	0.010	0.217	141.8	1.4	39
SR-07	57	3.4	1,395	94.2	0.010	0.229	220.0	0.8	36
SR-08	179	3.5	1,862	93.1	0.010	0.256	145.4	1.4	45
SR-09	60	4.6	903	93.0	0.010	0.285	219.6	0.8	52
SR-10	2,401	0.8	9,333	92.5	0.010	0.275	100.0	1.9	51
SR-11	109	2.1	2,202	91.6	0.020	0.381	130.2	1.9	74
SR-12	60	2.3	1,739	91.6	0.010	0.360	219.6	0.8	70
SR-13	41	1.5	1,693	92.5	0.010	0.320	214.6	0.8	58
SR-14	84	3.6	1,701	91.5	0.010	0.389	184.2	1.2	73
SR-15	195	2.7	2,681	95.3	0.010	0.195	113.8	1.6	25
SR-16	4.0	4.5	222	91.9	0.010	0.353	171.3	1.4	65
SR-17	1.8	4.2	142	94.6	0.010	0.178	110.0	2.2	26
SR-18	12	2.5	726	91.5	0.010	0.306	208.9	0.9	59
SR-19	3.3	3.0	339	94.7	0.010	0.154	110.0	2.2	22
SR-20	135	4.4	1,908	91.6	0.010	0.381	144.0	1.7	71
SR-21	17	2.7	600	93.7	0.010	0.199	115.3	2.0	33
SR-22	9.8	6.9	524	95.0	0.010	0.151	99.5	1.9	20
SR-23	83	3.7	1,663	94.3	0.010	0.222	145.7	1.7	35
SR-24	0.4	10.8	58	96.8	0.010	0.150	88.9	1.7	11
SR-25	0.9	10.8	277	95.5	0.010	0.150	164.0	1.1	18
SR-26	2.7	4.7	172	97.0	0.010	0.150	89.7	1.7	10
SR-27	36	4.7	1,057	95.0	0.010	0.150	171.6	1.3	20
SR-28	2.1	10.1	111	97.0	0.010	0.150	95.0	1.8	10
SR-29	5.1	10.1	345	96.1	0.010	0.150	143.9	1.5	15
SR-30	66	5.3	2,390	95.3	0.010	0.150	109.0	2.2	18
SR-31	36	2.5	1,302	95.8	0.010	0.150	125.8	1.9	17
SR-32	13	7.1	740	95.0	0.010	0.150	110.0	2.2	20
SR-33	1.7	5.9	185	97.0	0.010	0.150	110.0	2.2	10

Table B-8: Estimated Watershed Characteristics for the Little Sackville River (Post-Calibration) (Fall Conditions) (Future Development Conditions)

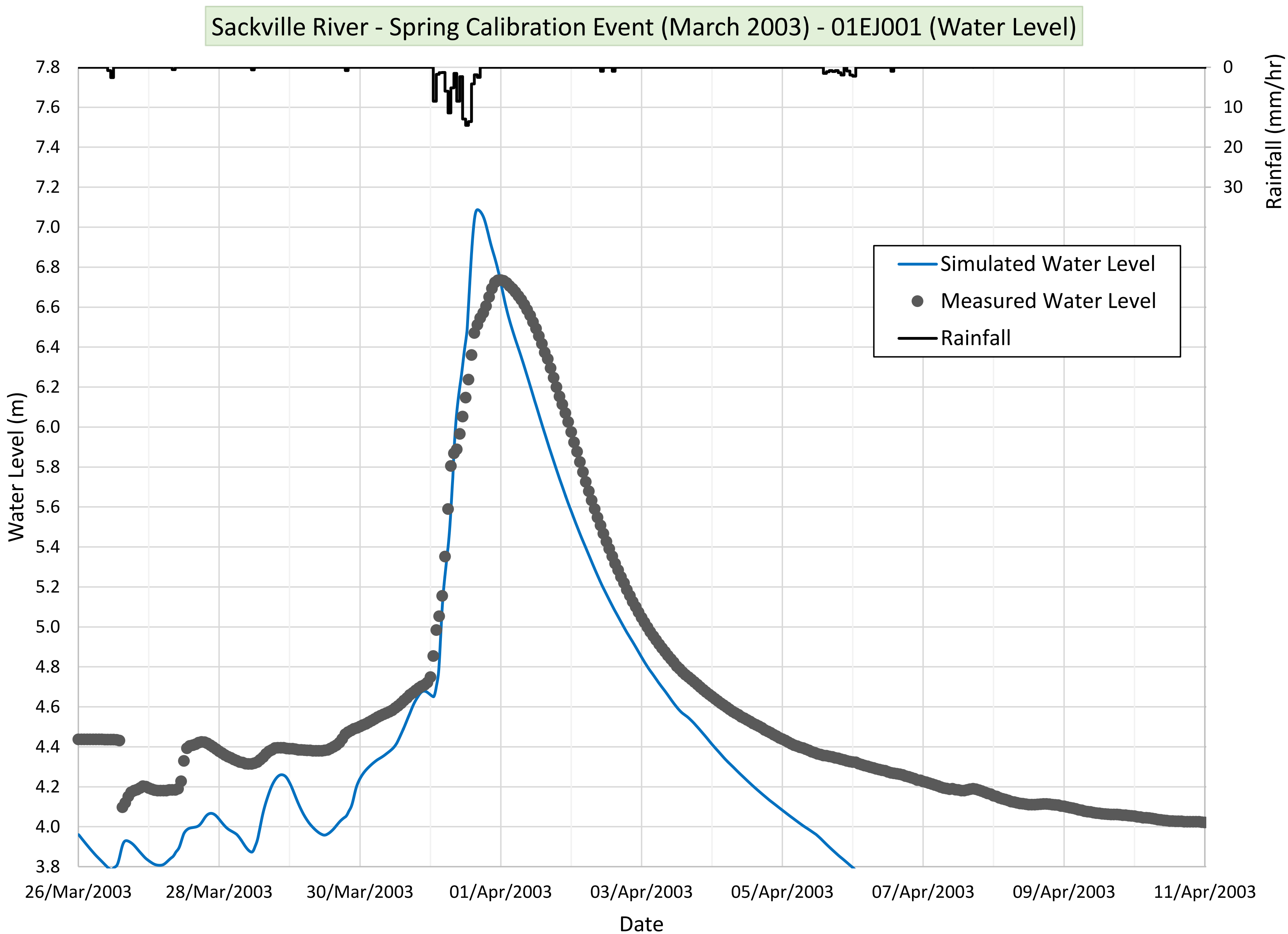
Sub-Watershed Name	Area (ha)	Slope (%)	Maximum Overland Flow Length (m)	Imperv. (%)	Impervious Area Roughness	Pervious Area Roughness	Capillary Suction Head (mm)	Saturated Hydraulic Conductivity (mm/hr)	Percent of Area Routed to Pervious Area (%)
LSR-01	158	1.4	2,045	91.8	0.050	0.377	172.6	1.0	71
LSR-02	88	2.8	1,188	92.6	0.050	0.349	124.9	1.4	65
LSR-03	3.6	4.8	231	94.0	0.040	0.222	110.0	2.2	36
LSR-04	44	3.3	815	94.0	0.040	0.222	187.1	1.2	36
LSR-05	7.0	5.7	274	94.3	0.040	0.201	110.0	2.2	31
LSR-06	6.7	3.3	602	95.1	0.030	0.148	133.3	1.9	20
LSR-07	3.6	5.8	360	95.5	0.040	0.139	110.0	2.2	18
LSR-08	2.9	2.8	400	96.1	0.040	0.124	110.0	2.2	16
LSR-09	0.0	3.5	17	95.0	0.030	0.150	110.0	2.2	20
LSR-10	0.6	6.2	224	95.0	0.030	0.150	110.0	2.2	20
LSR-11	0.8	1.9	160	95.0	0.030	0.150	110.0	2.2	20
LSR-12	2.8	6.2	211	95.0	0.030	0.150	110.0	2.2	20
LSR-13	41	0.8	819	95.0	0.030	0.150	148.4	1.7	20
LSR-14	47	3.3	1,282	93.8	0.040	0.239	123.5	1.8	41
LSR-15	11	3.6	564	93.9	0.040	0.229	111.3	2.2	44
LSR-16	251	2.9	3,022	92.6	0.040	0.261	188.5	1.0	47
LSR-17	15	2.8	677	94.9	0.030	0.157	110.0	2.2	22
LSR-18	59	4.0	1,694	94.5	0.030	0.163	182.8	0.9	25
LSR-19	4.2	5.1	138	94.2	0.040	0.208	104.2	2.1	38
LSR-20	131	4.9	1,706	94.5	0.040	0.178	195.3	0.7	28
LSR-21	13	6.2	565	94.8	0.040	0.166	95.3	1.9	25
LSR-22	4.0	5.8	344	95.0	0.030	0.150	186.0	1.2	20
LSR-23	1.4	3.2	124	95.0	0.030	0.150	97.8	1.9	20
LSR-24	23	5.6	402	95.0	0.030	0.150	197.3	1.1	20
LSR-25	41	6.2	1,084	95.0	0.030	0.150	179.7	1.3	20
LSR-26	6.4	3.7	229	95.0	0.030	0.150	172.2	1.4	20
LSR-27	1.8	3.1	385	95.0	0.030	0.150	110.0	2.2	20
LSR-28	24	3.9	367	95.4	0.030	0.150	172.7	1.4	18
LSR-29	95	4.9	1,304	95.0	0.030	0.150	199.3	1.0	20
LSR-30	146	2.4	1,685	95.7	0.030	0.150	220.0	0.8	16
LSR-31	9.0	3.0	233	95.1	0.030	0.150	216.8	0.8	19
LSR-32	111	2.4	2,500	94.9	0.030	0.155	208.1	0.9	21
LSR-33	192	4.1	1,420	95.5	0.030	0.150	181.6	1.3	18

APPENDIX C

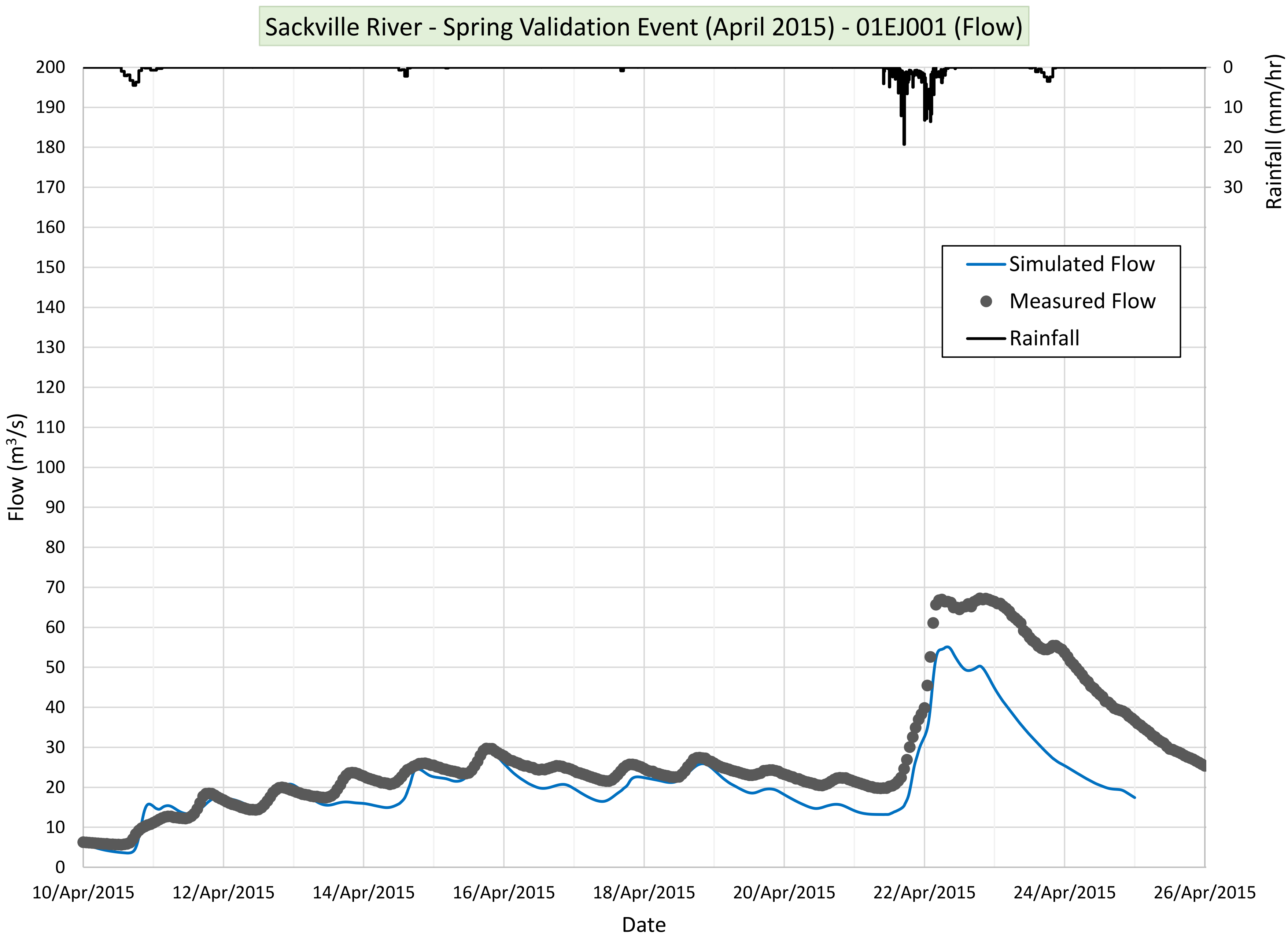
Model Calibration Results



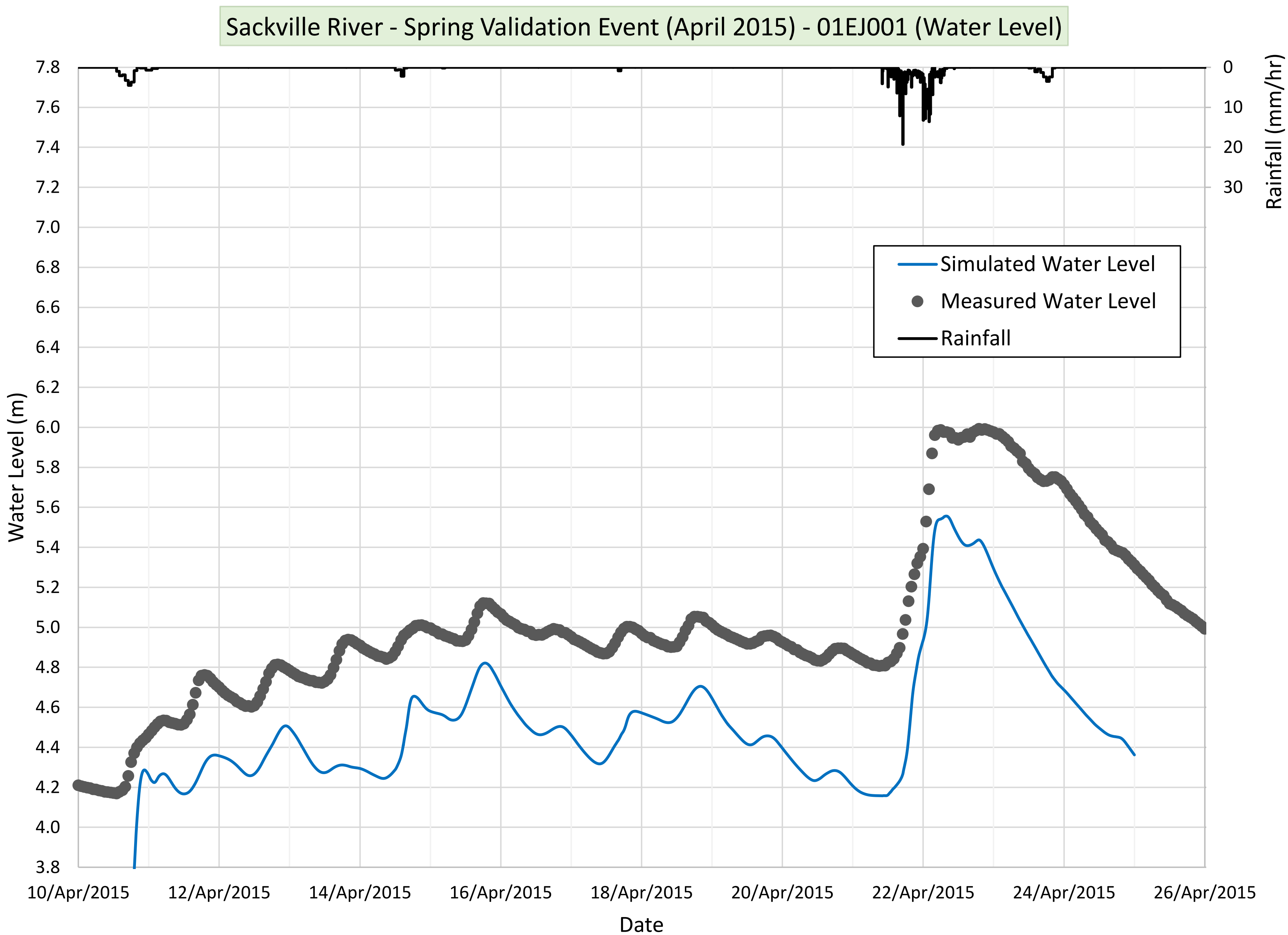
Model Calibration Results Graph #1



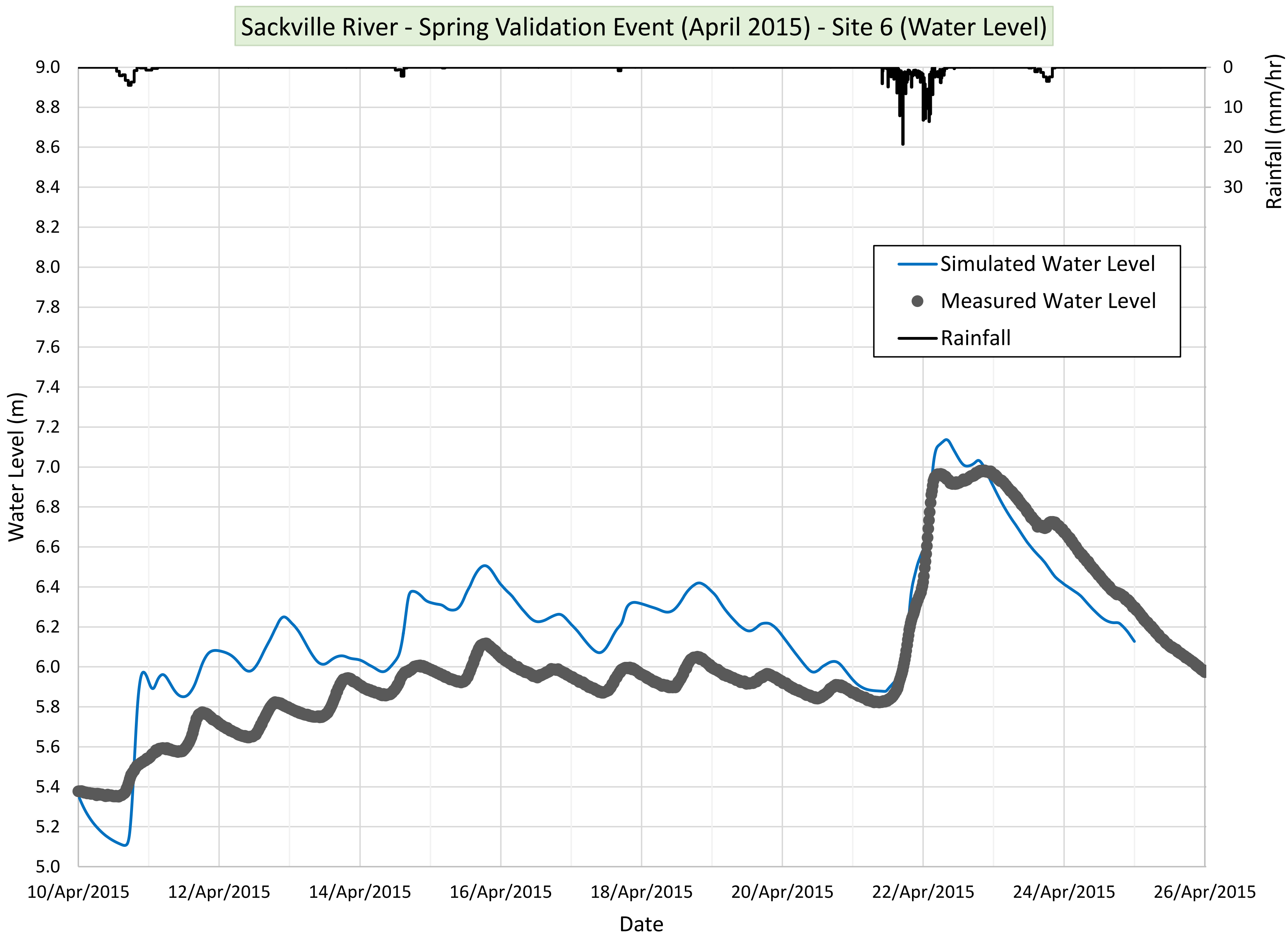
Model Calibration Results Graph #2



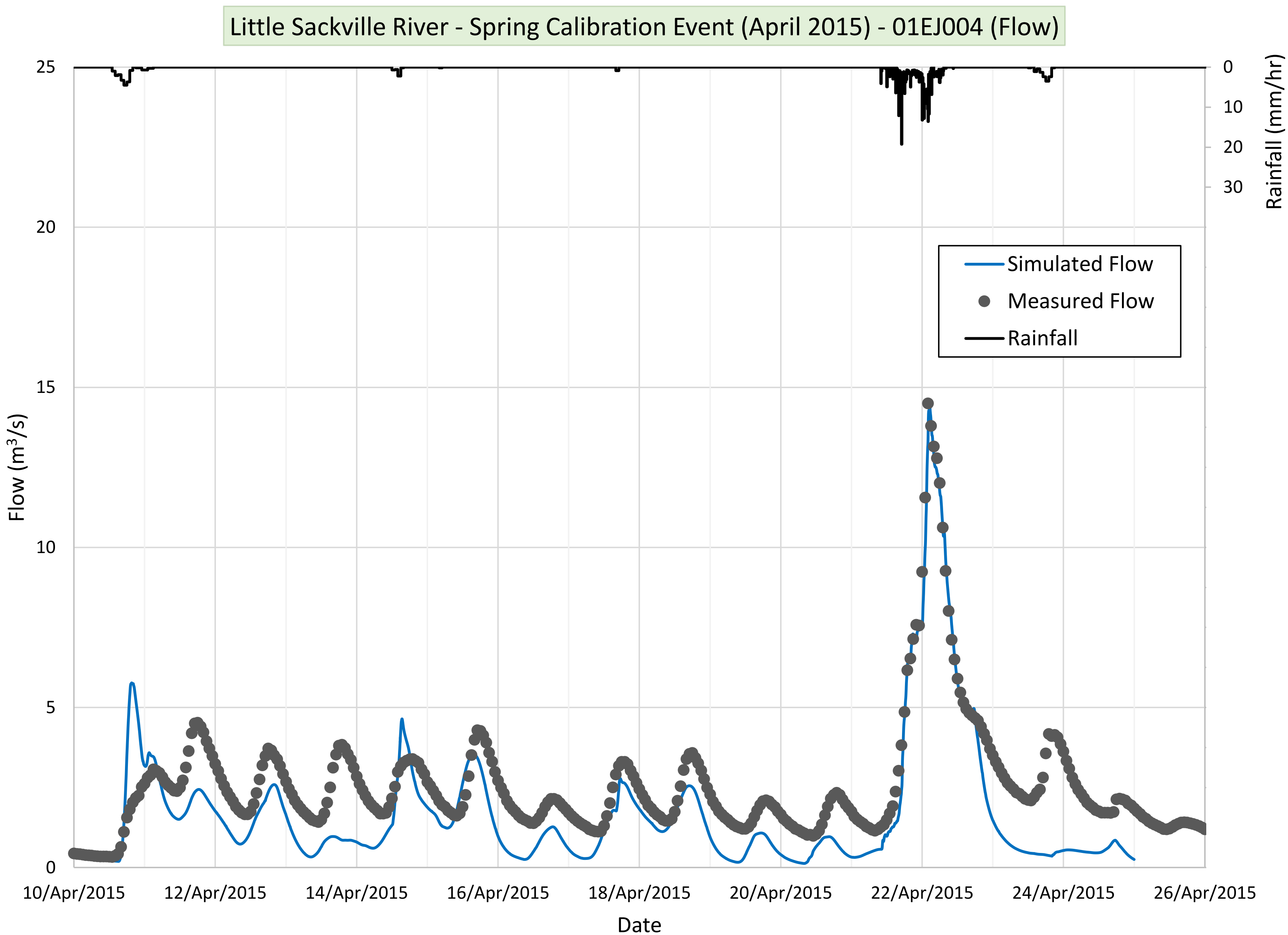
Model Calibration Results Graph #3



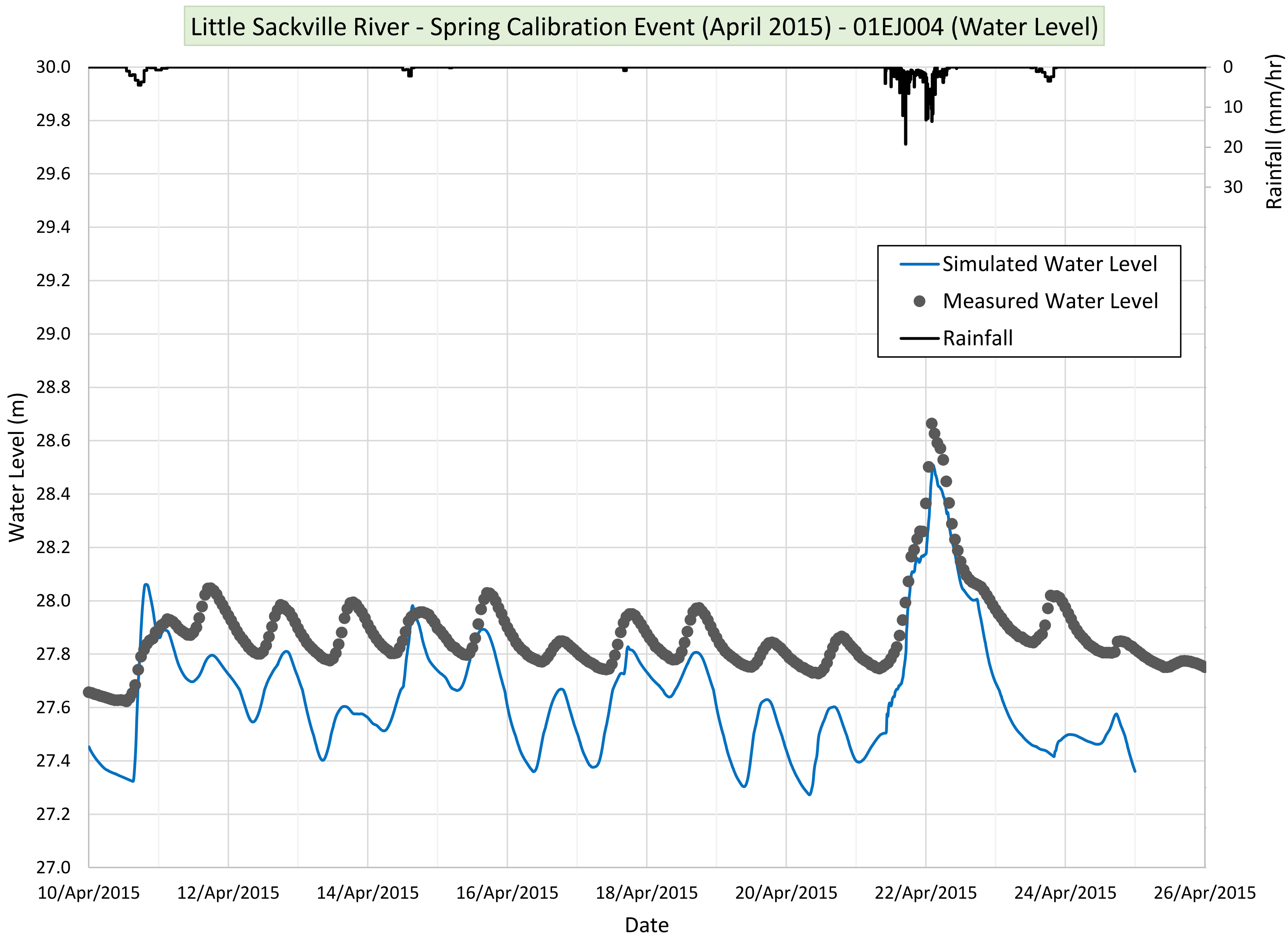
Model Calibration Results Graph #4



Model Calibration Results Graph #5



Model Calibration Results Graph #6



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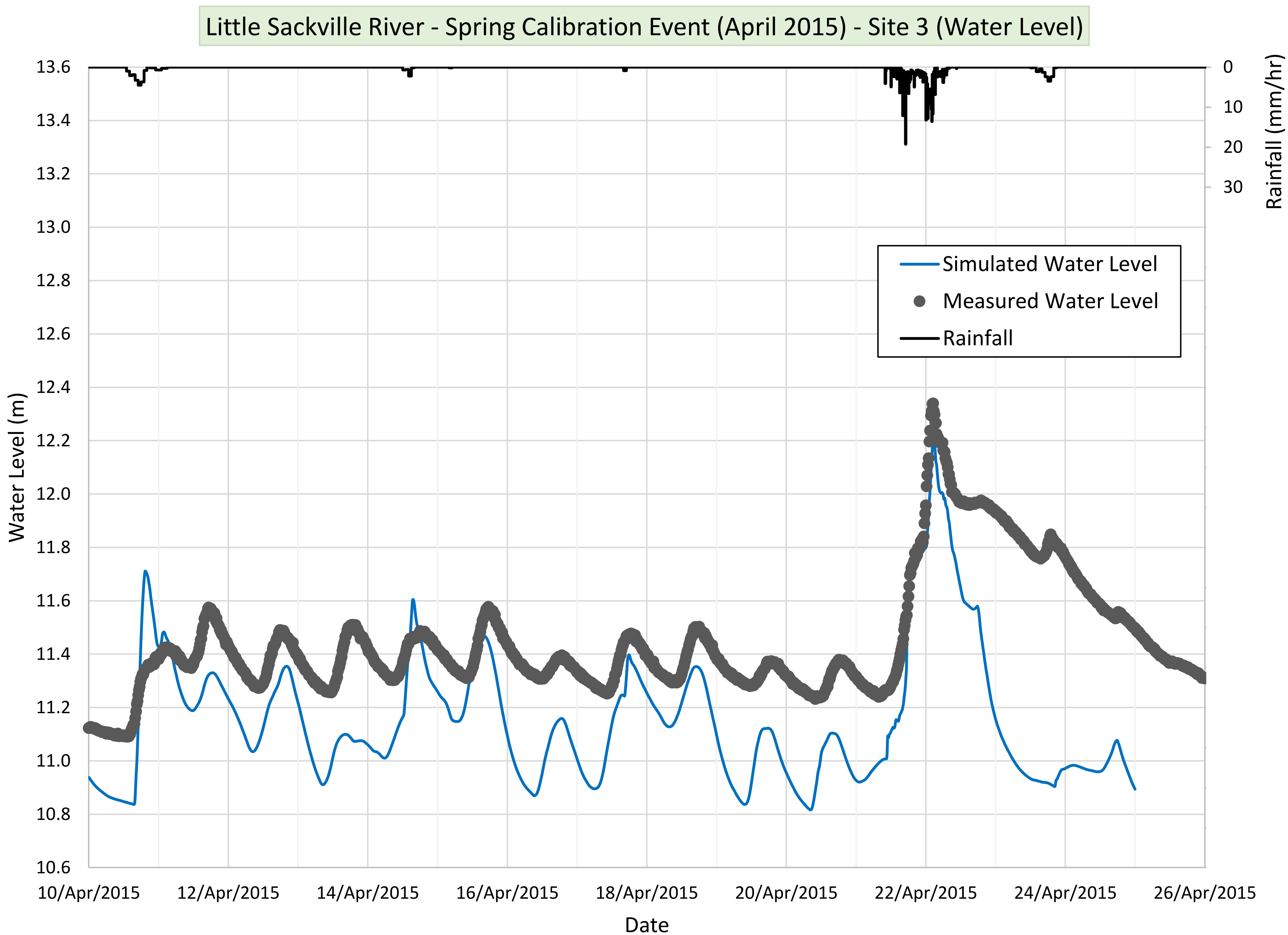
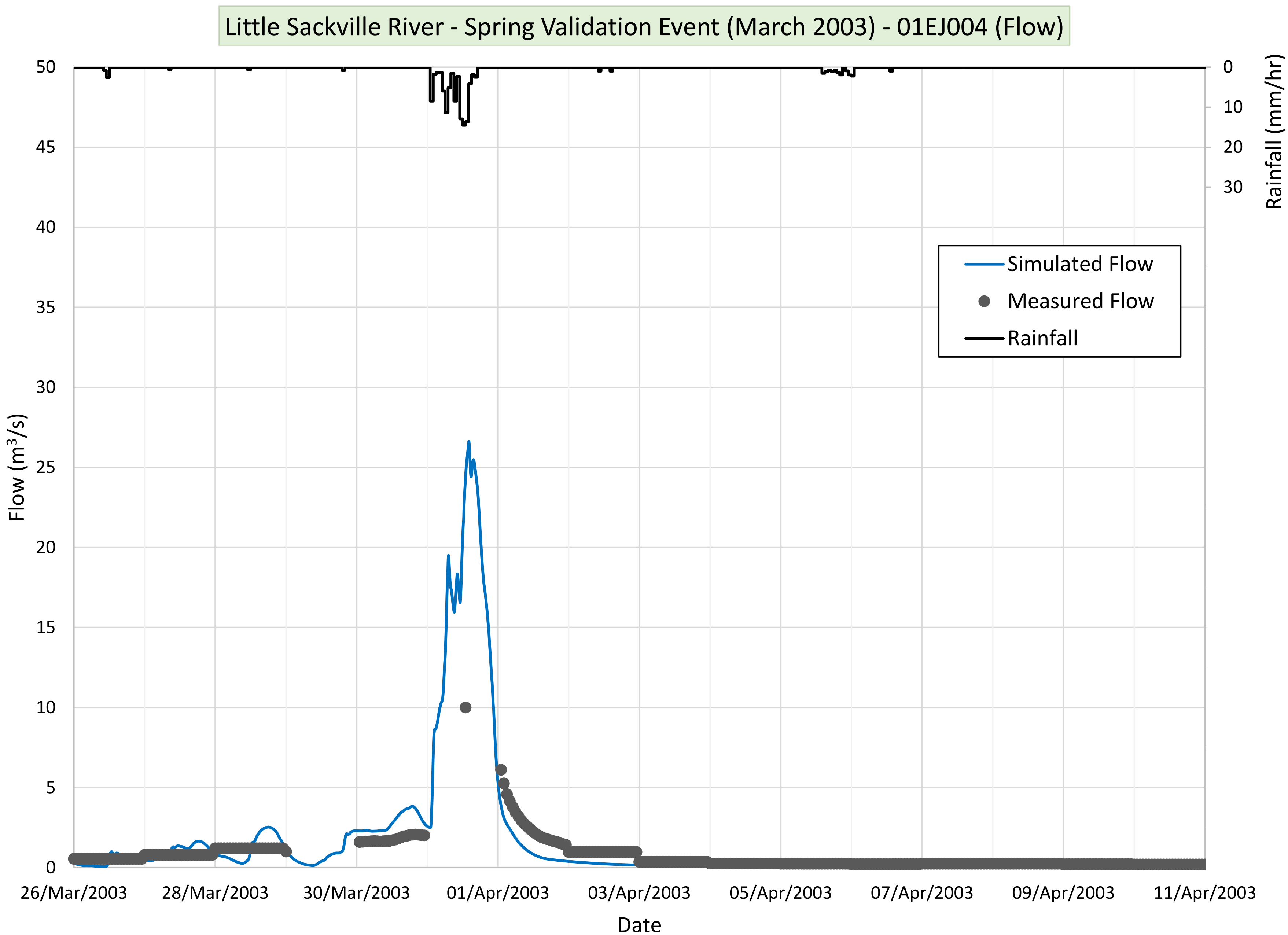
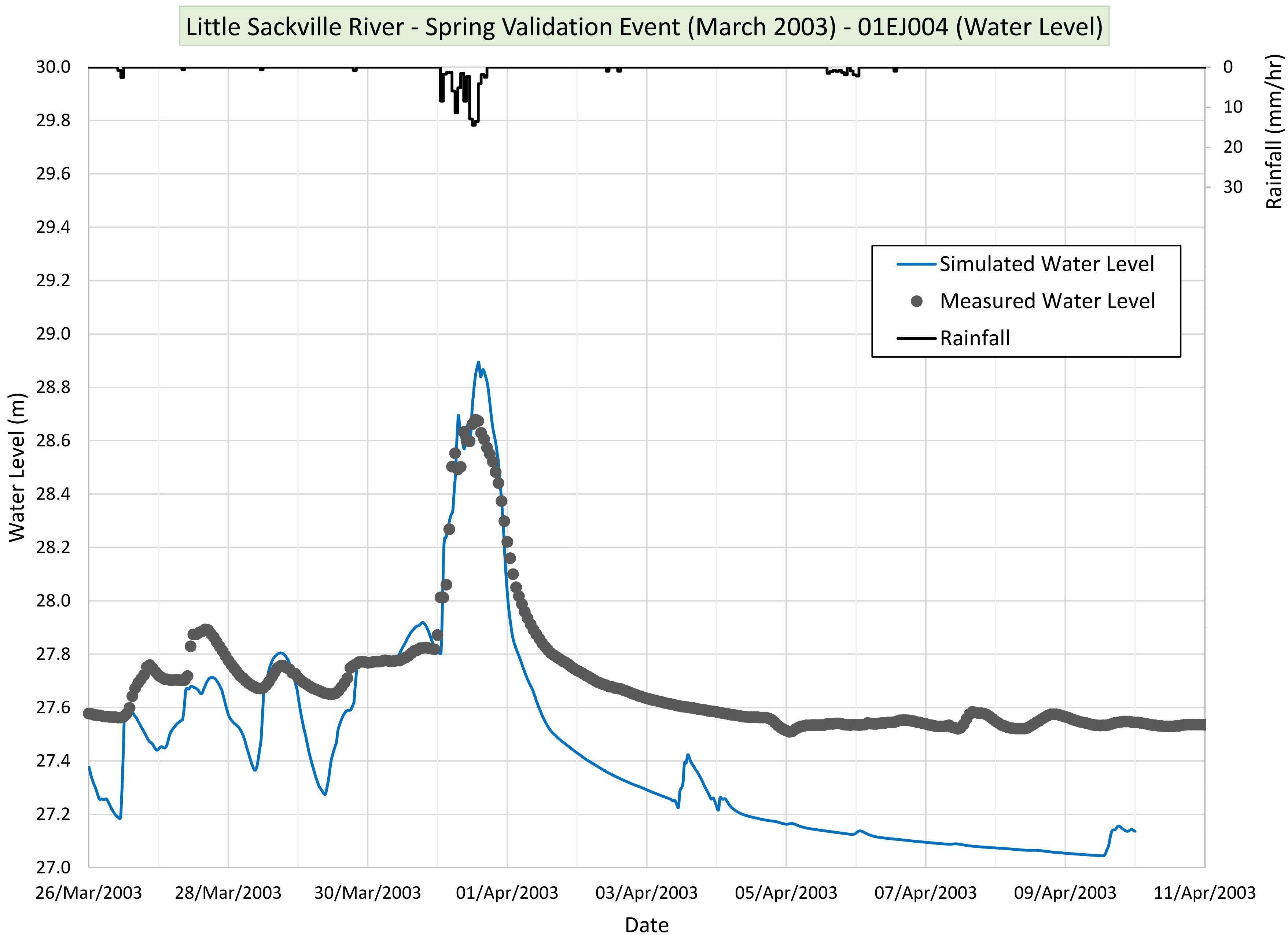


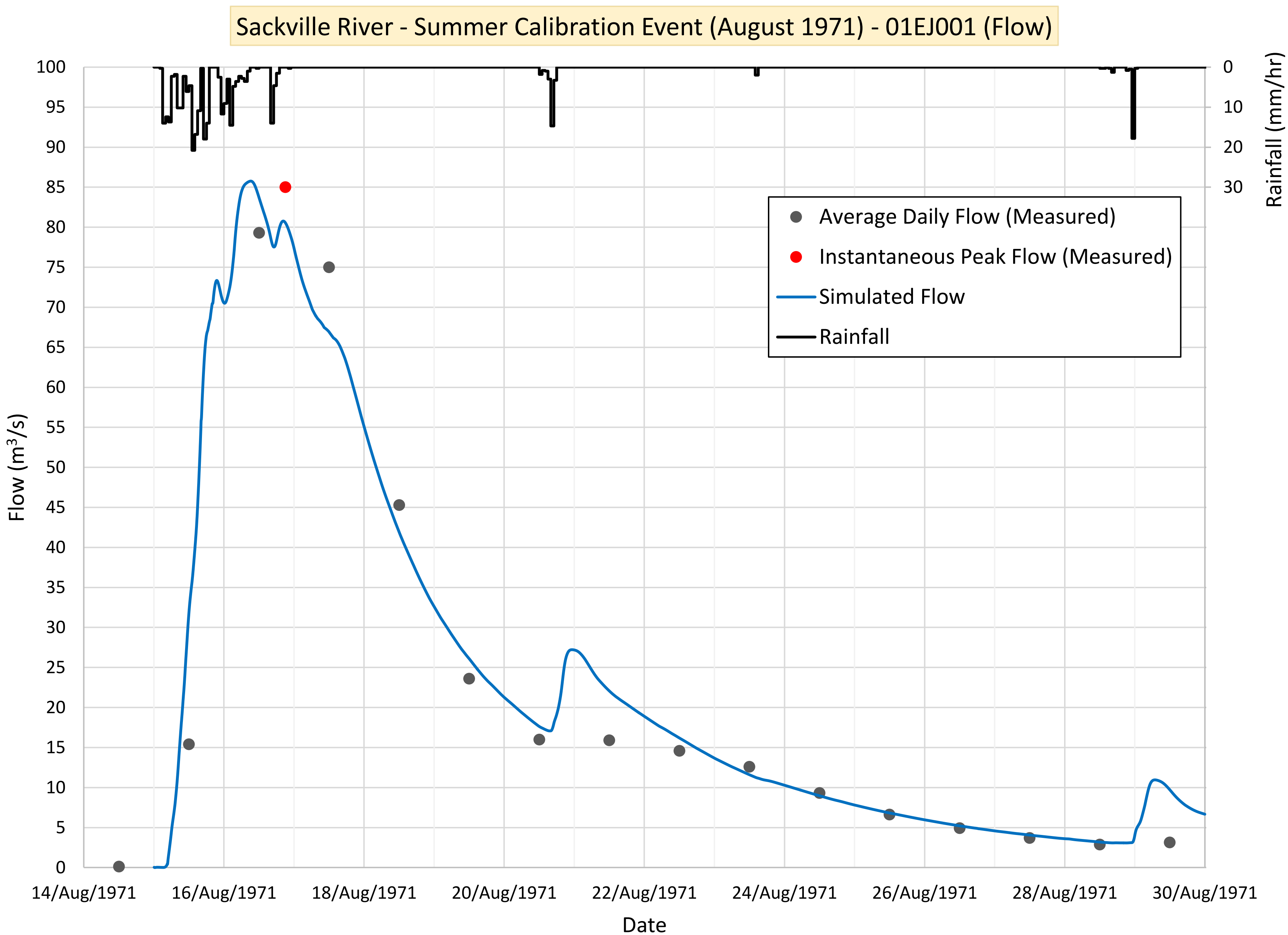
Model Calibration Results Graph #8



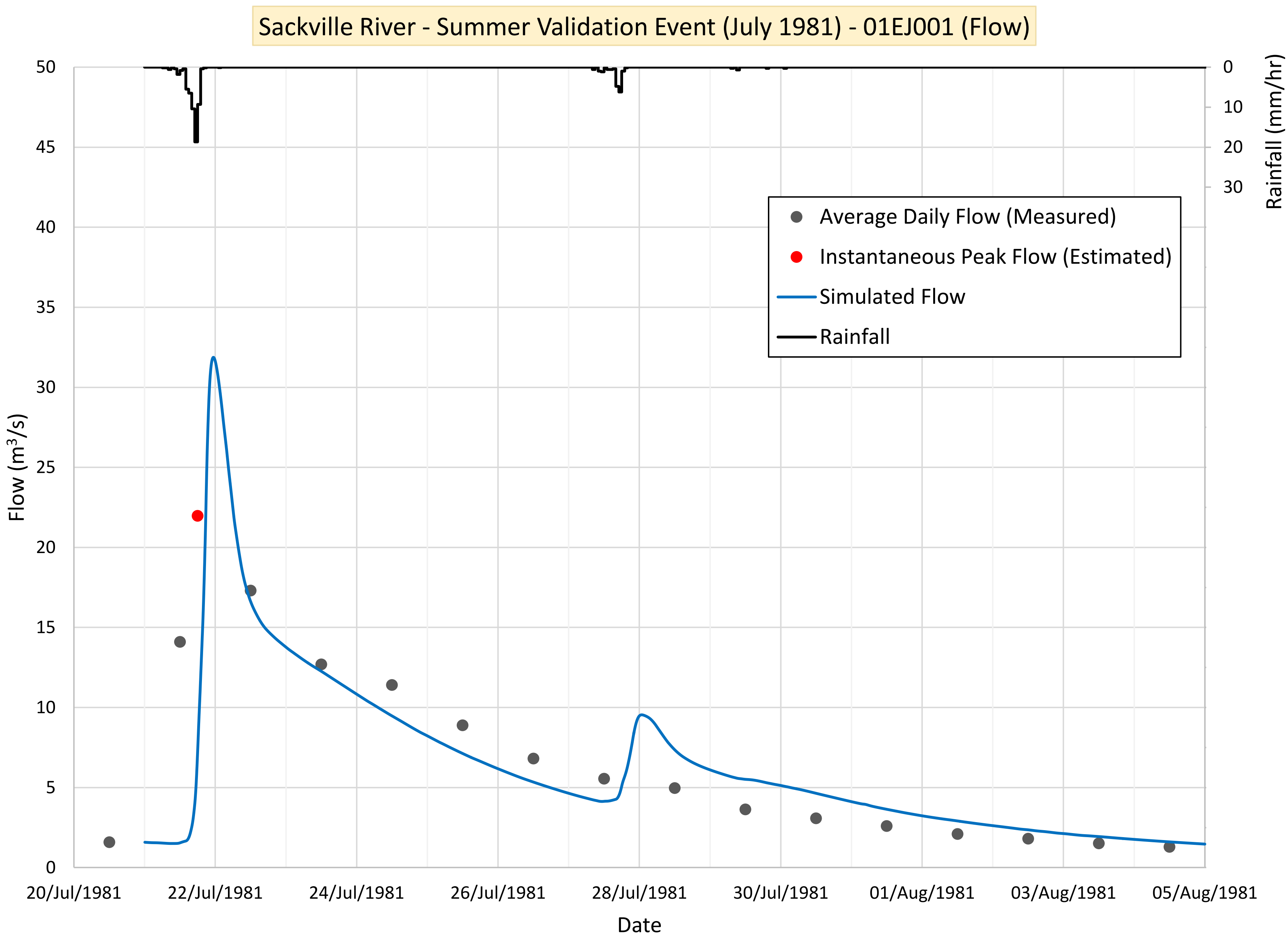
Model Calibration Results Graph #9



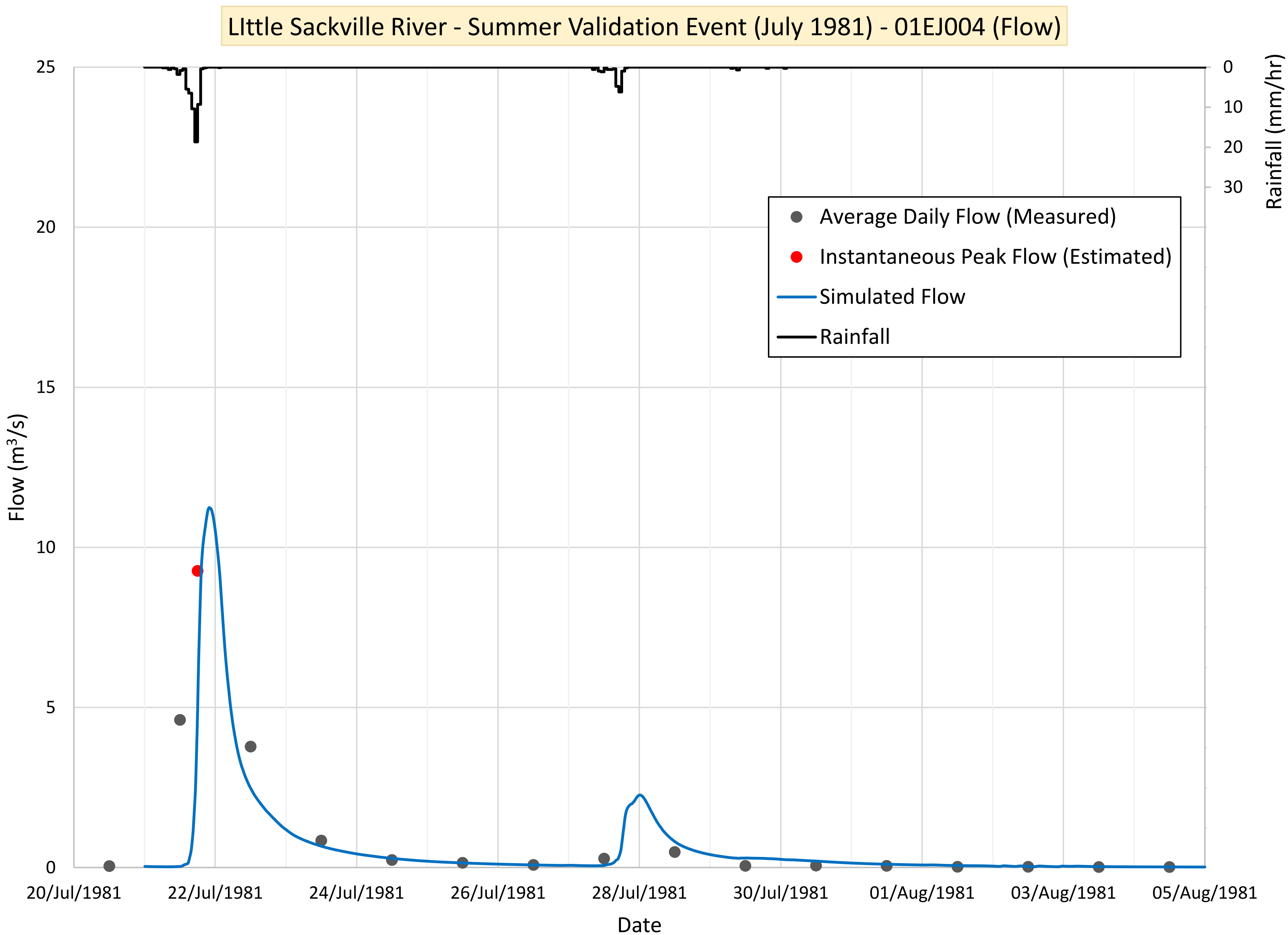
Model Calibration Results Graph #10



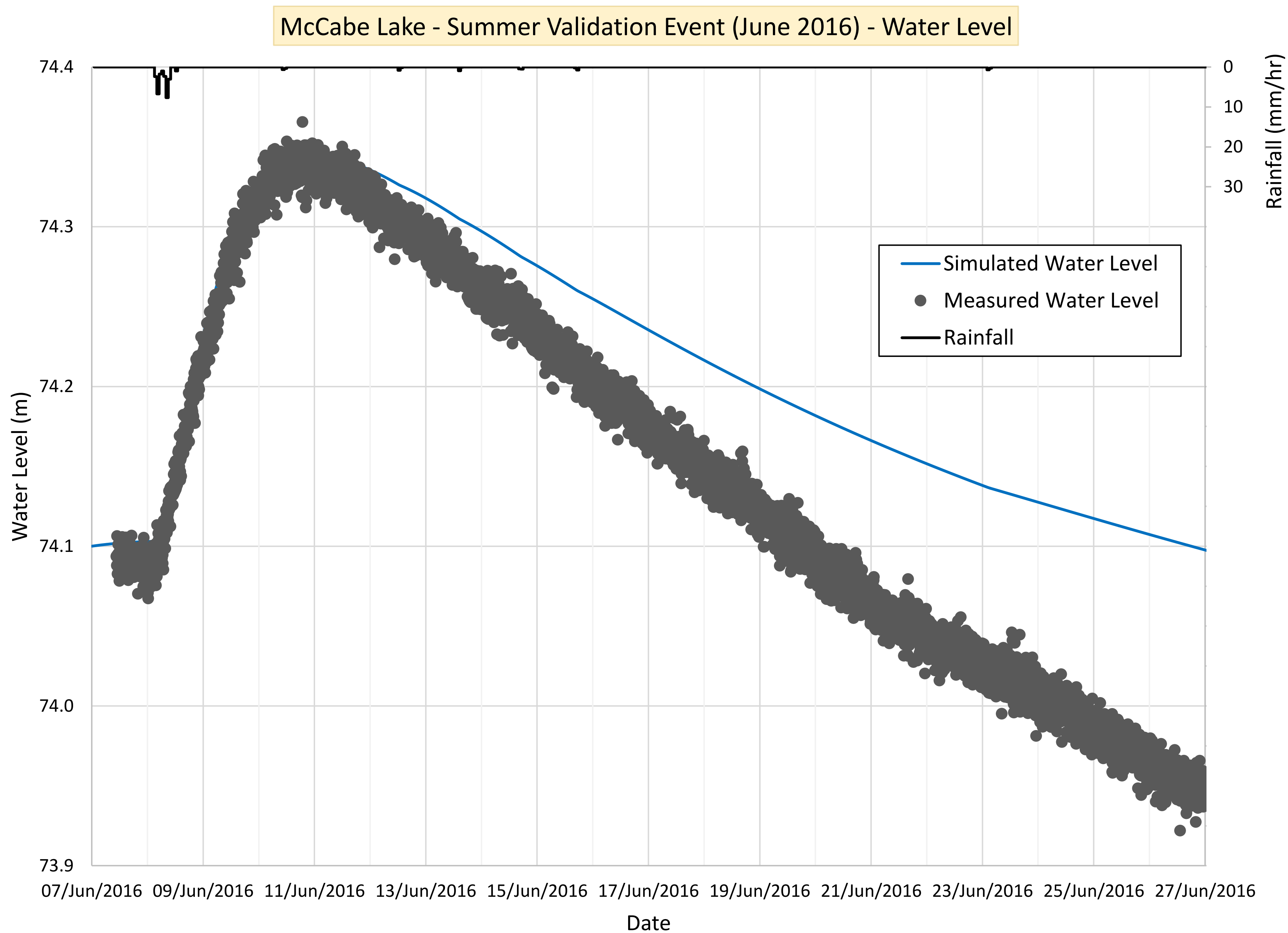
Model Calibration Results Graph #11



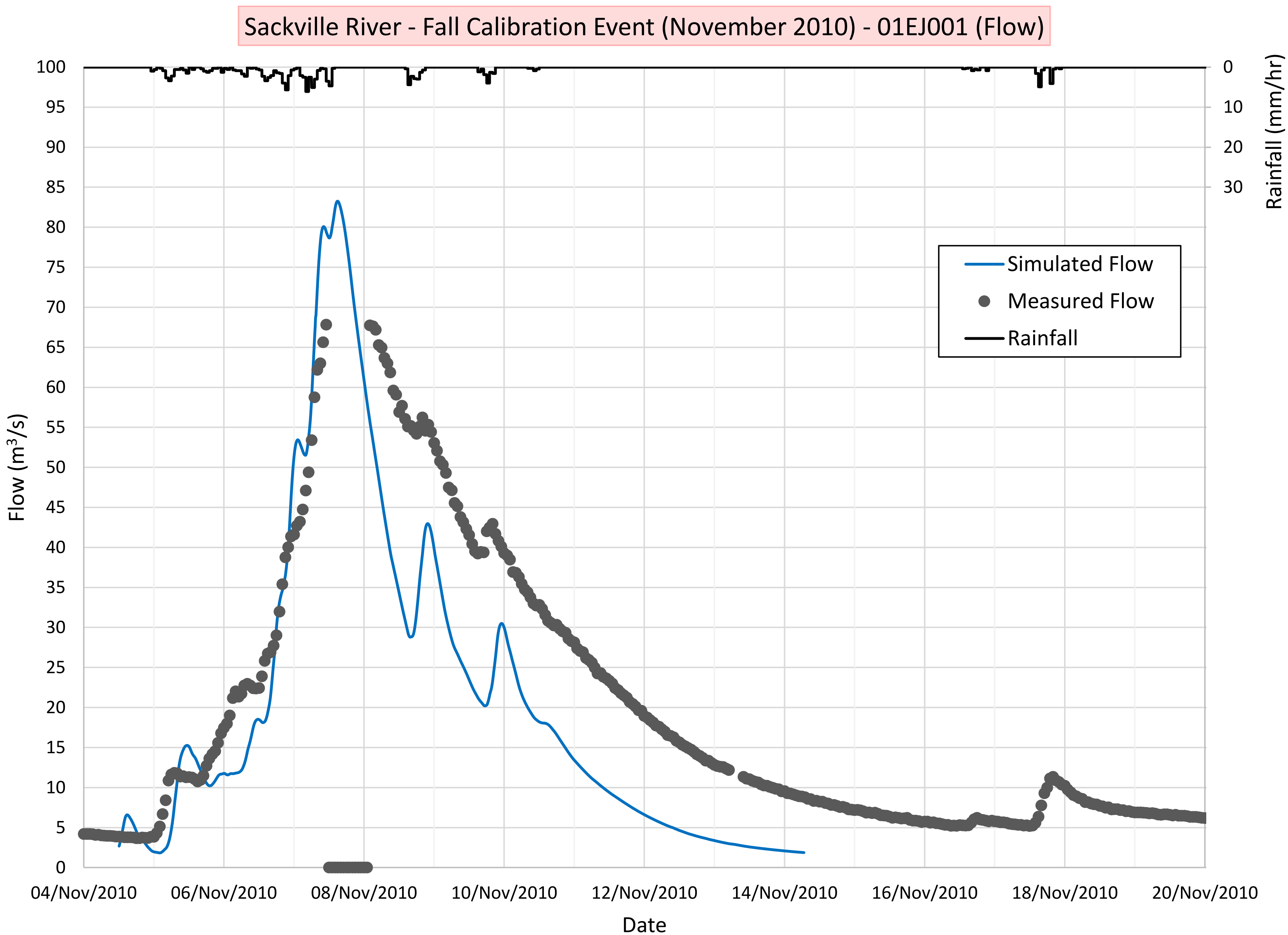
Model Calibration Results Graph #12



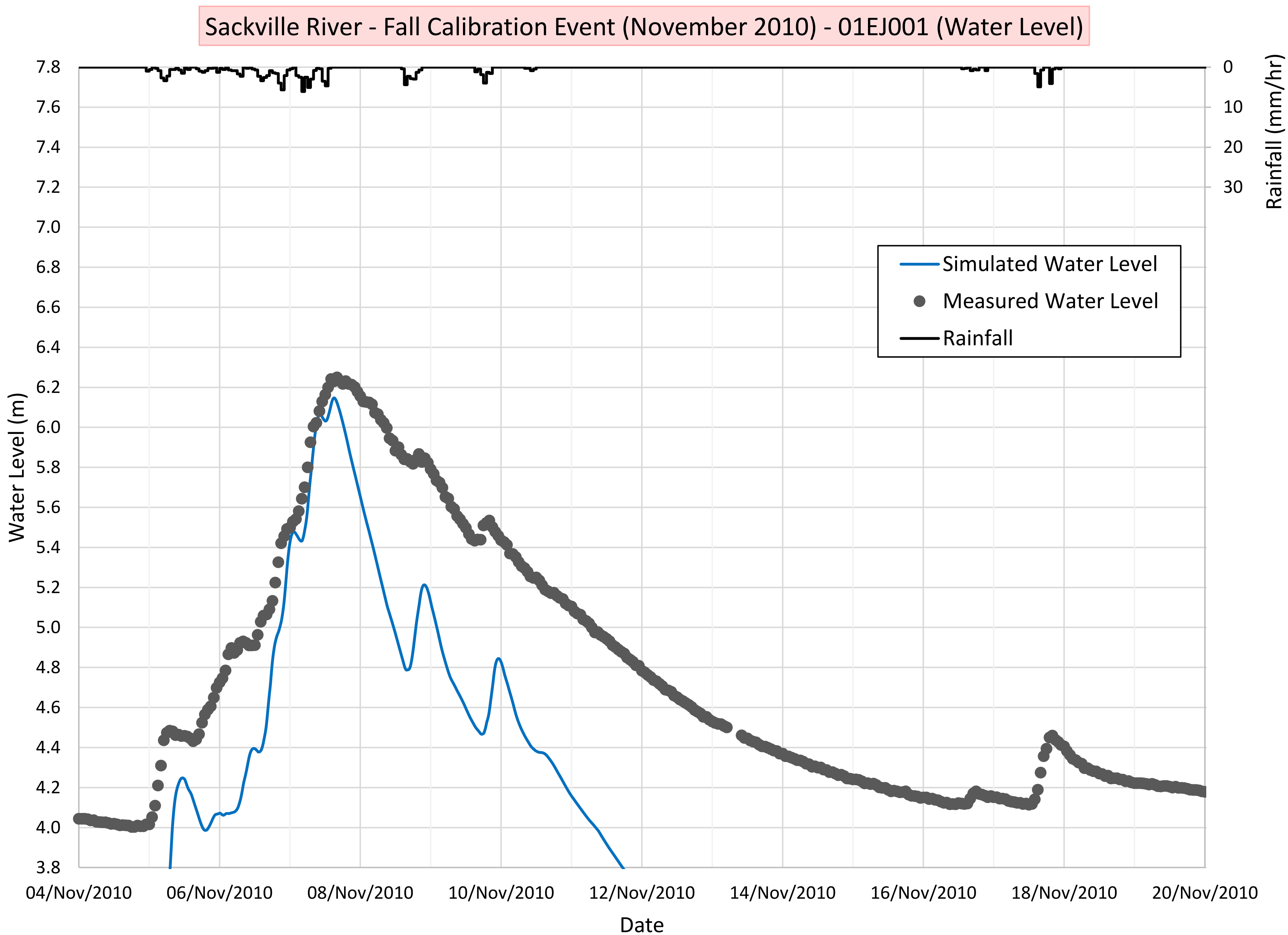
Model Calibration Results Graph #13



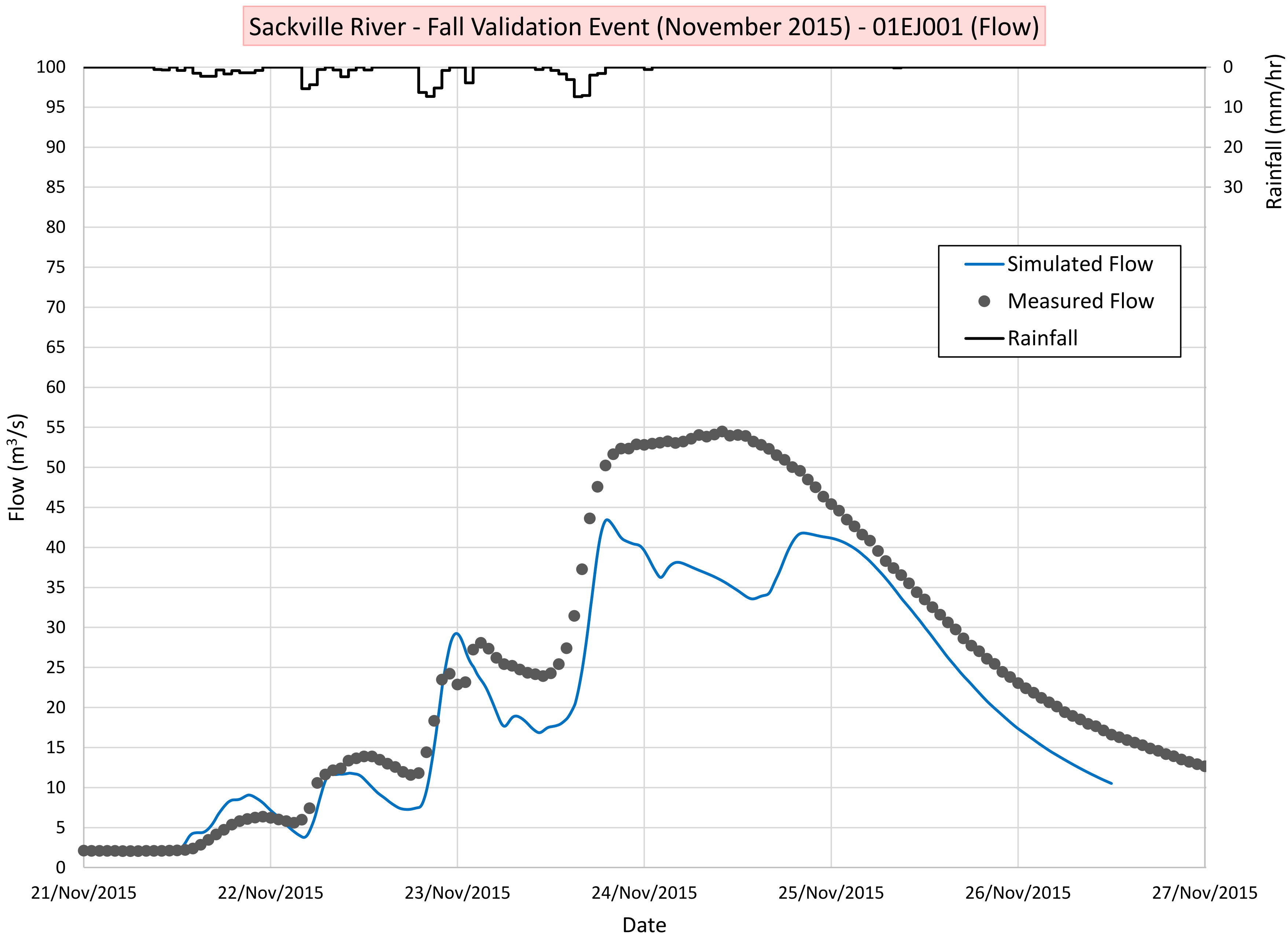
Model Calibration Results Graph #14



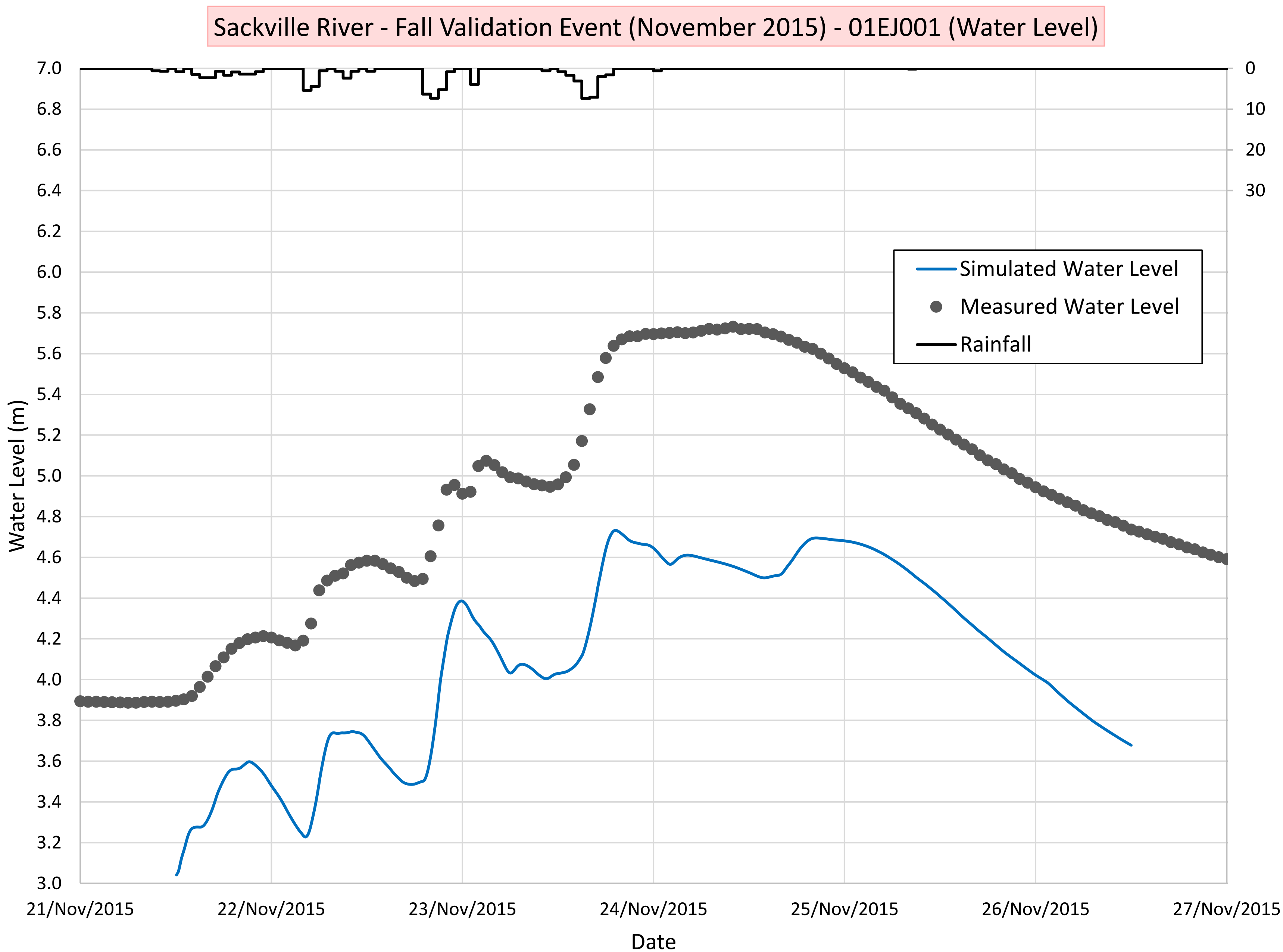
Model Calibration Results Graph #15



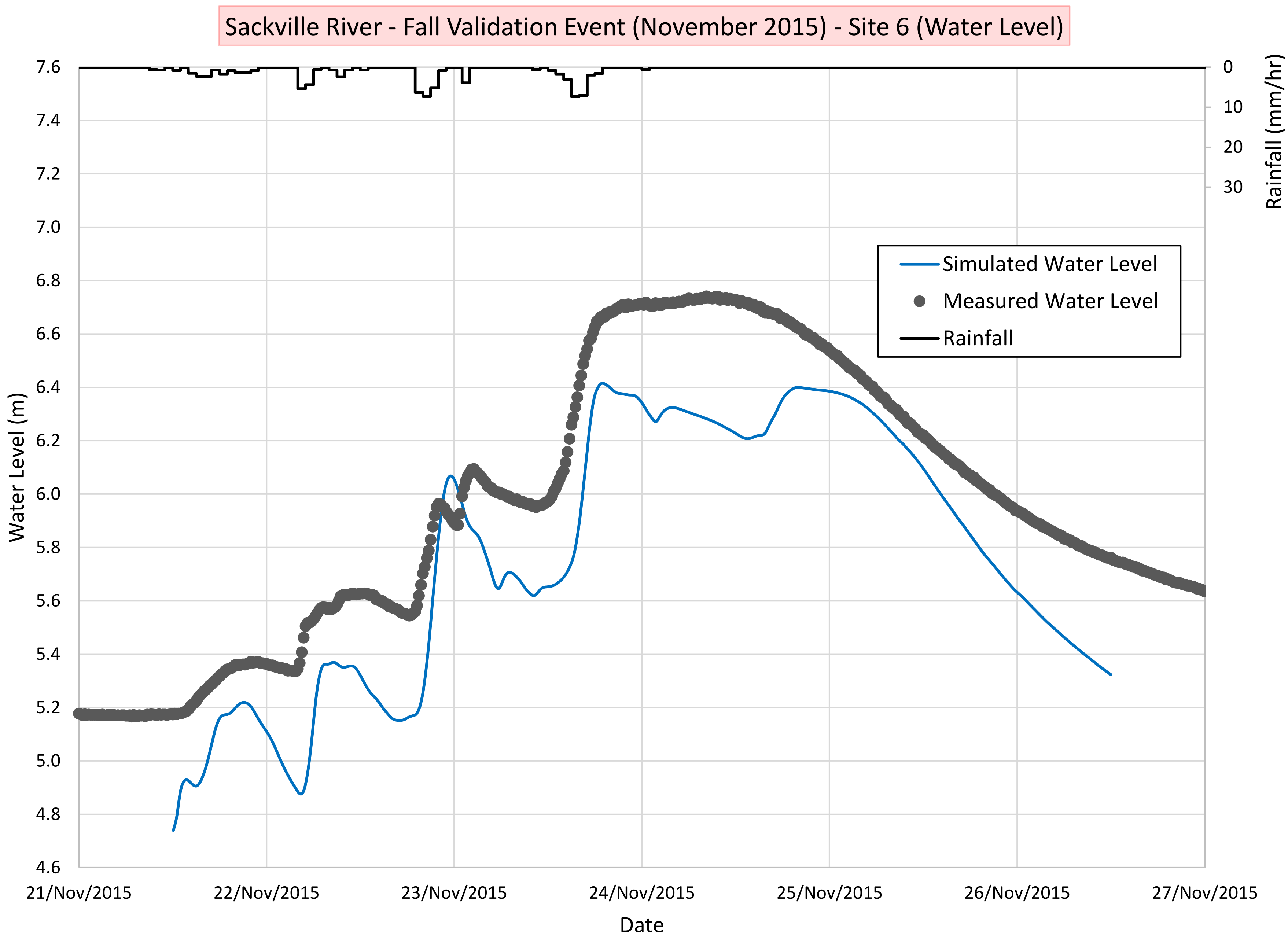
Model Calibration Results Graph #16



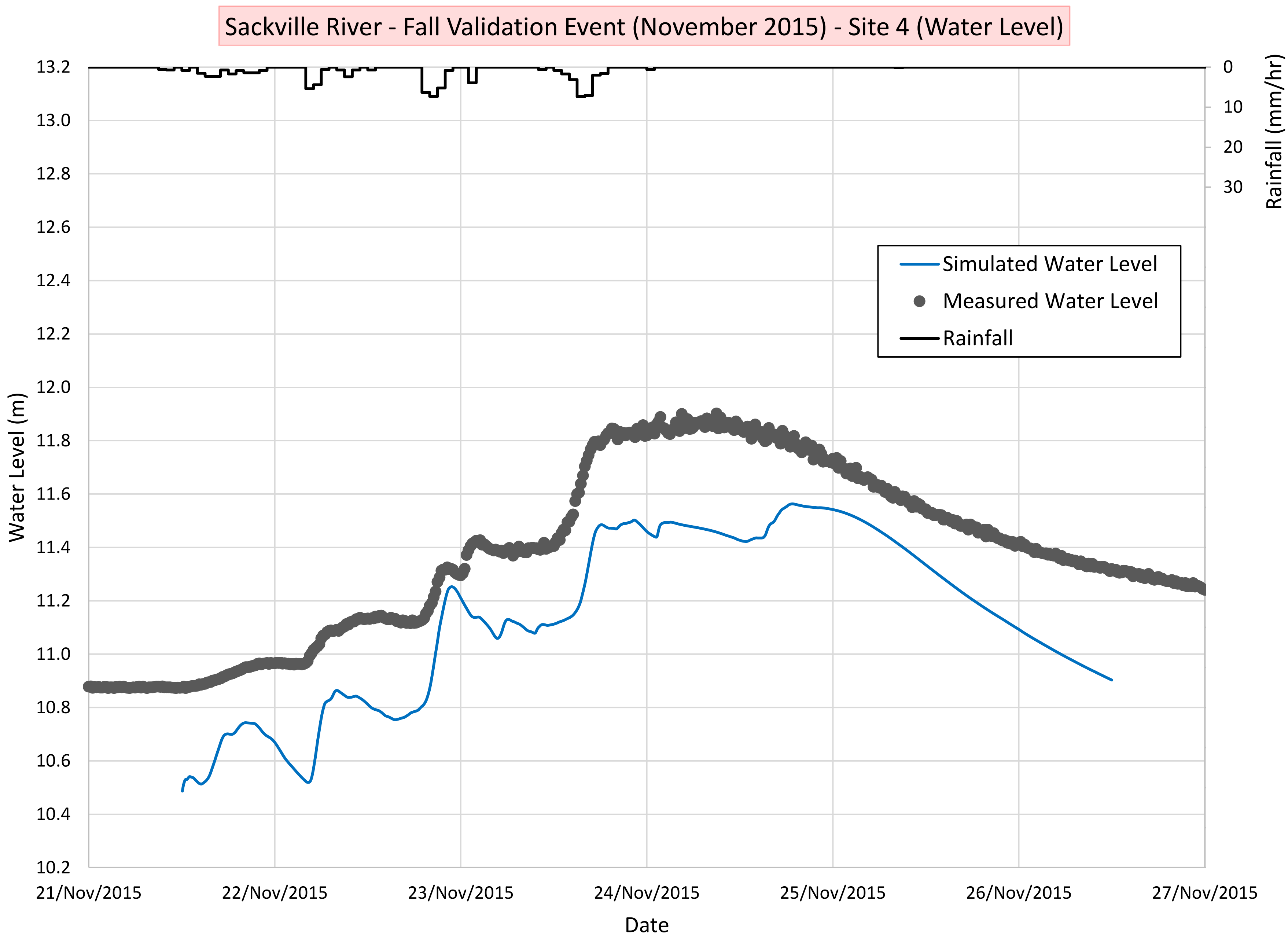
Model Calibration Results Graph #17



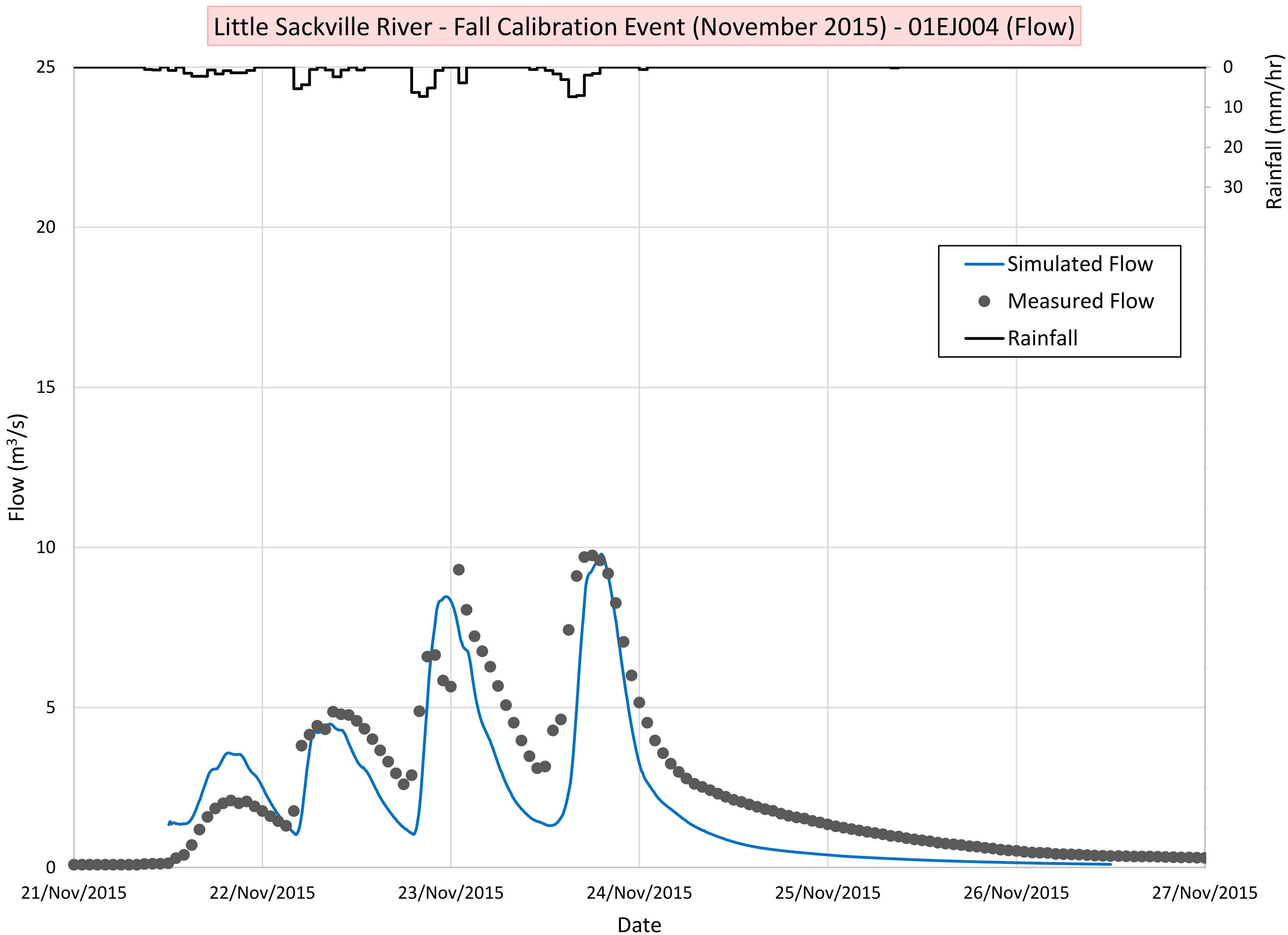
Model Calibration Results Graph #18



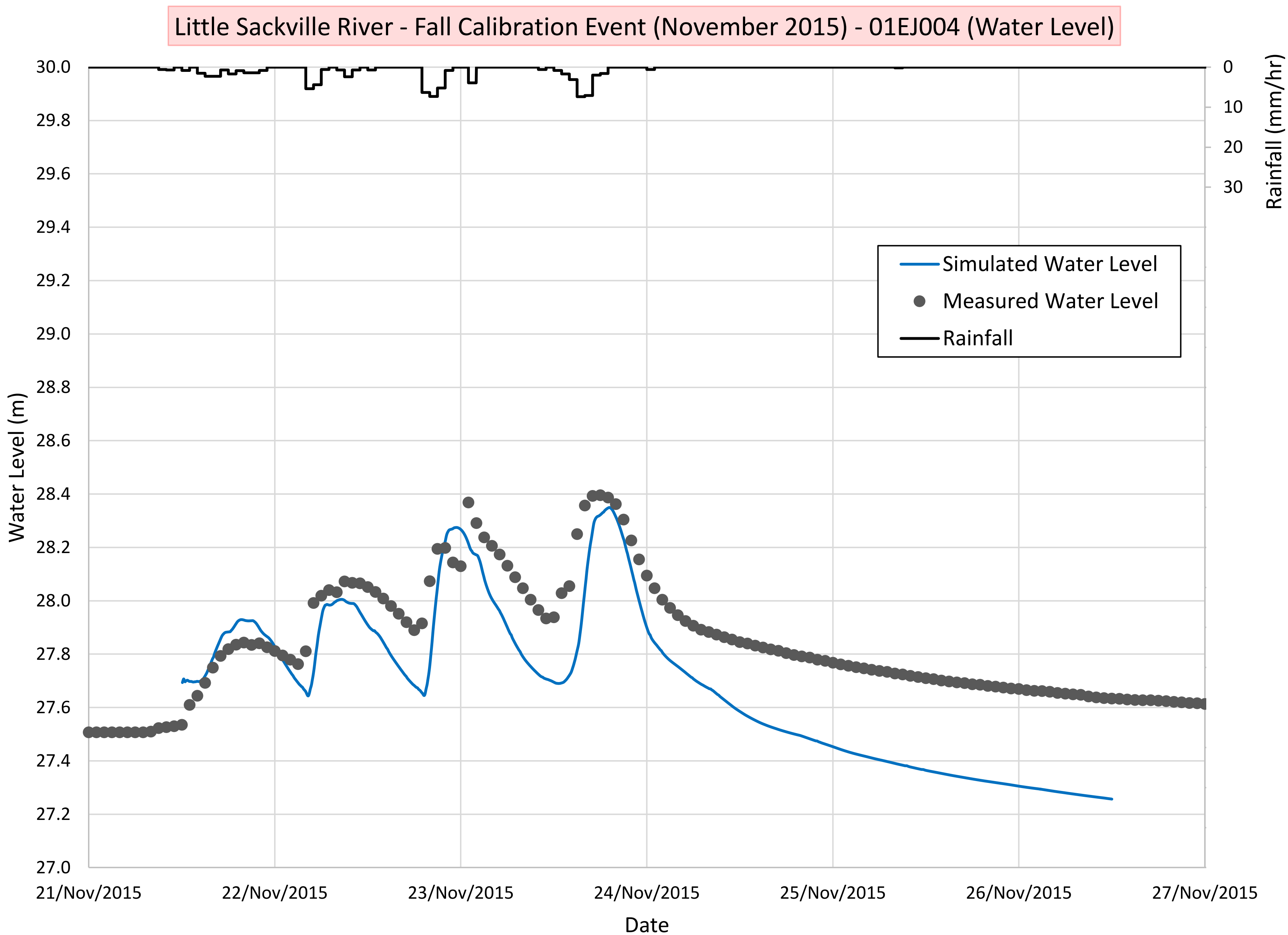
Model Calibration Results Graph #19



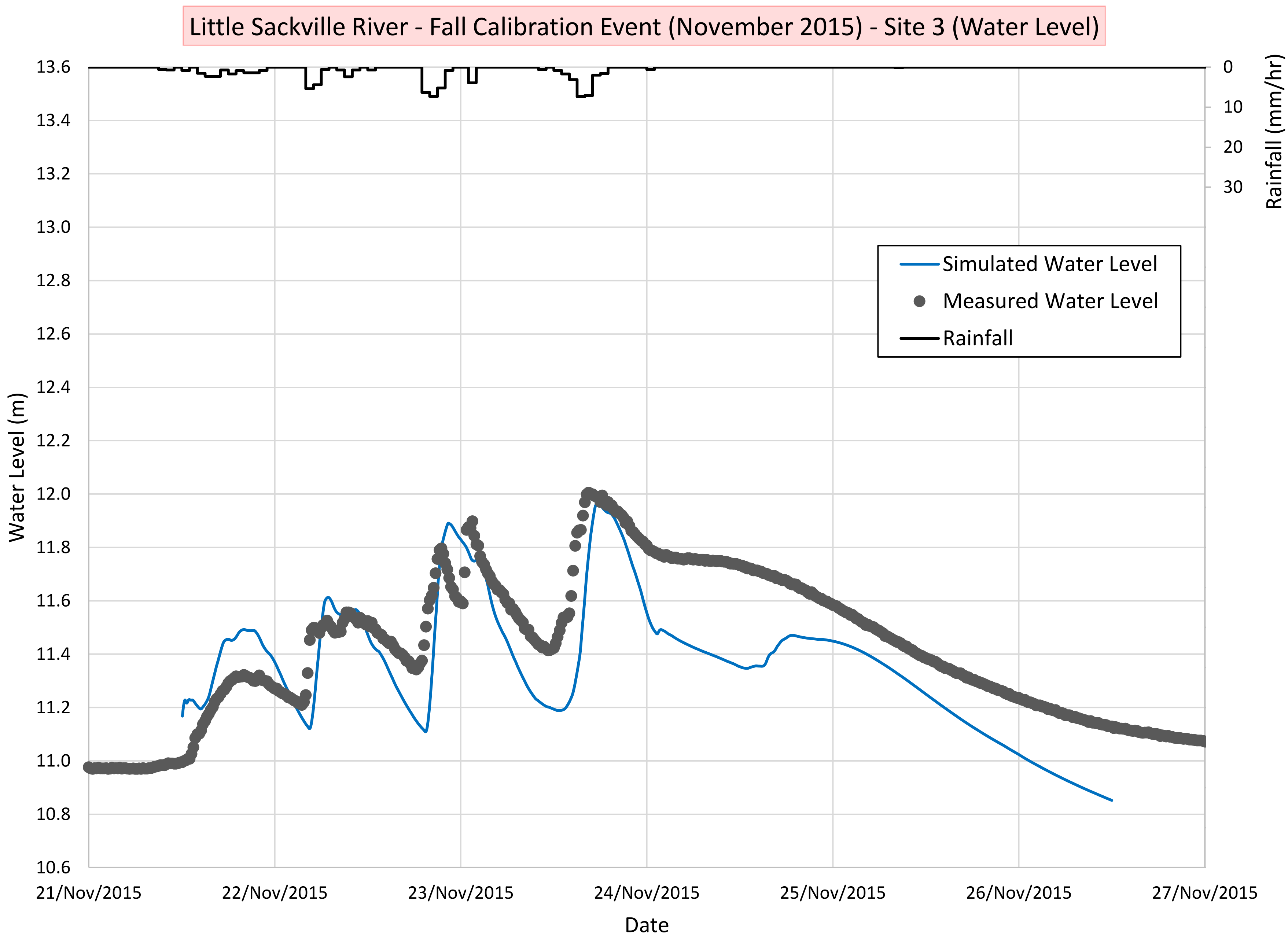
Model Calibration Results Graph #20



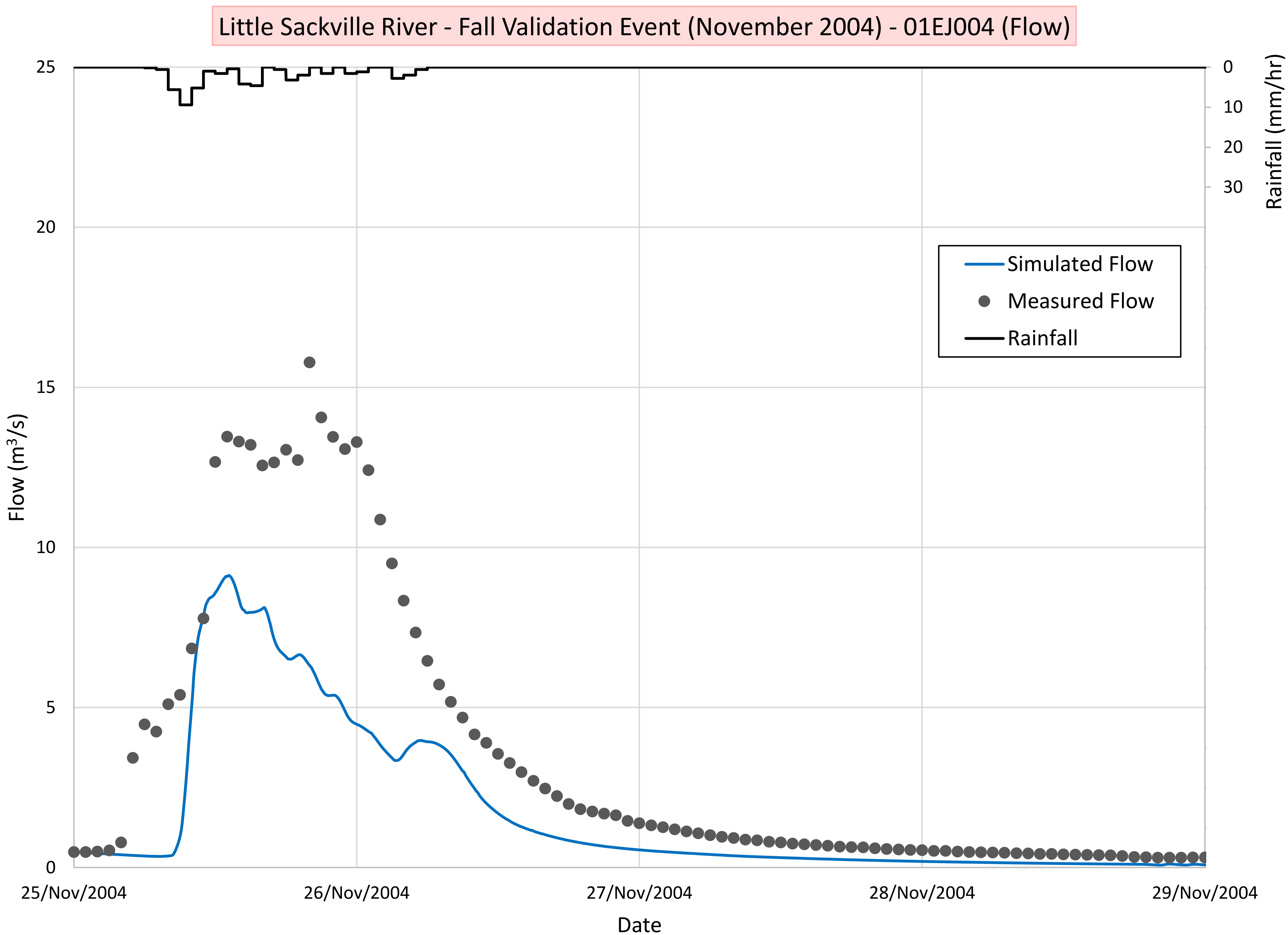
Model Calibration Results Graph #21



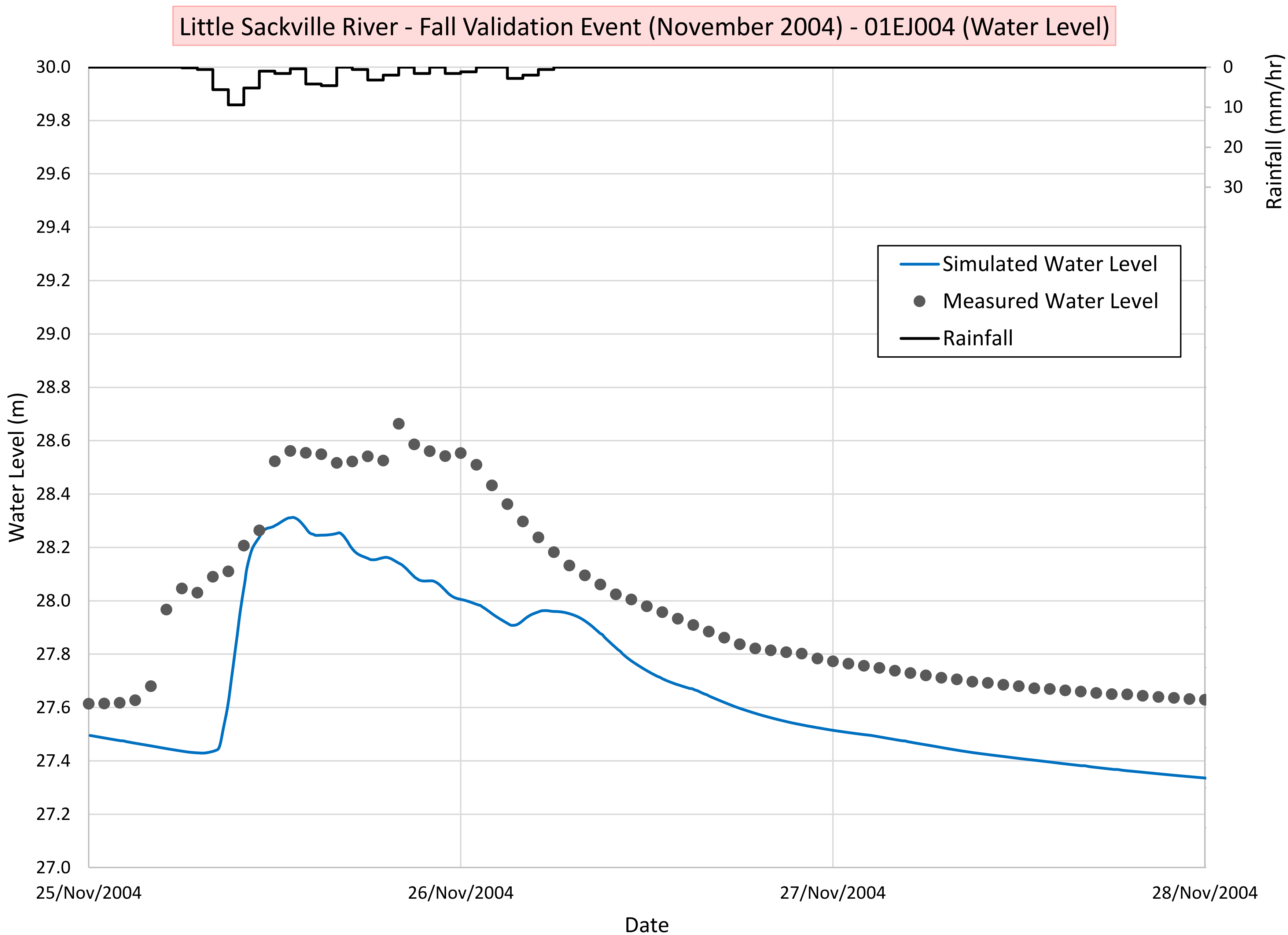
Model Calibration Results Graph #22



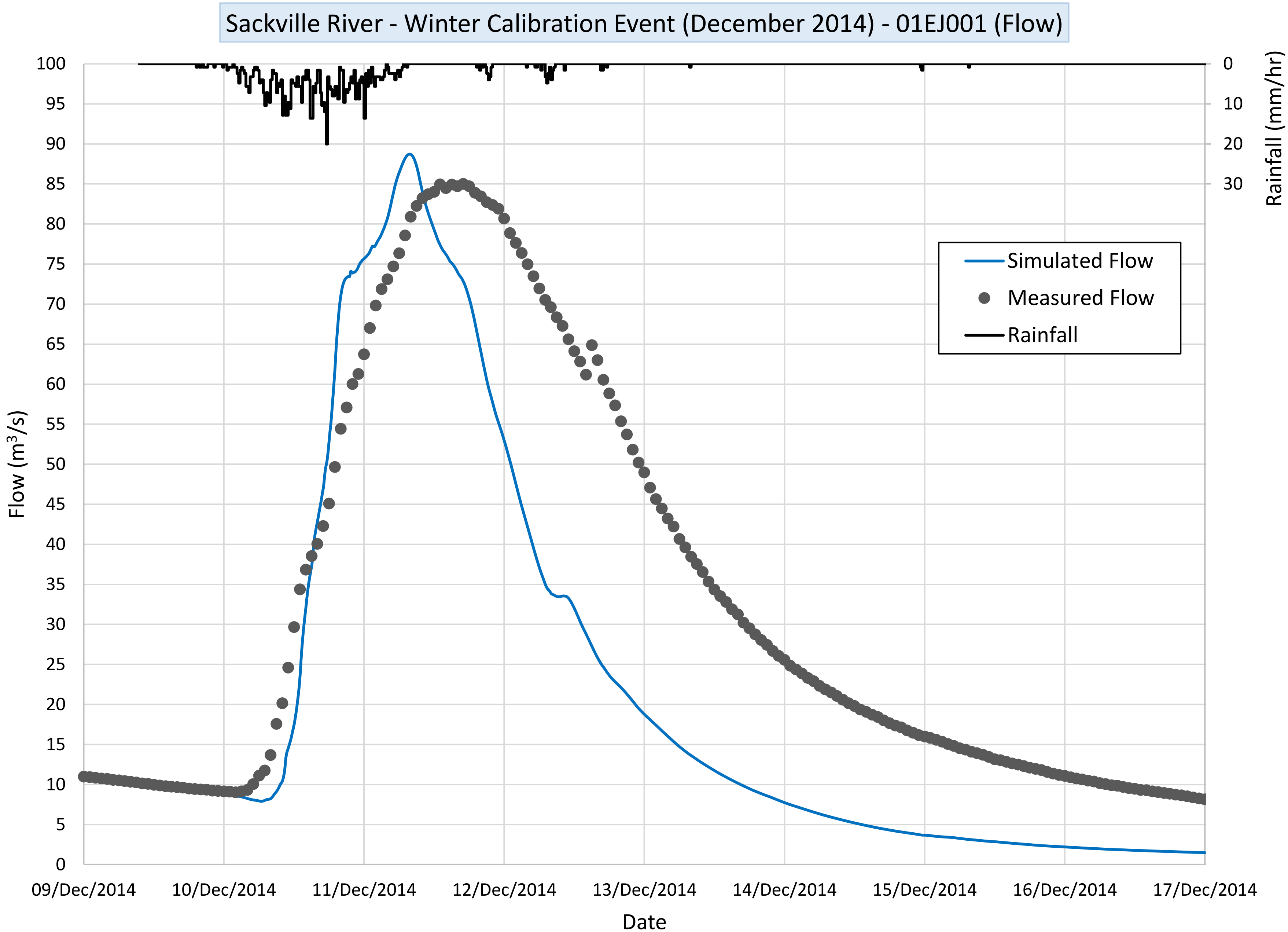
Model Calibration Results Graph #23



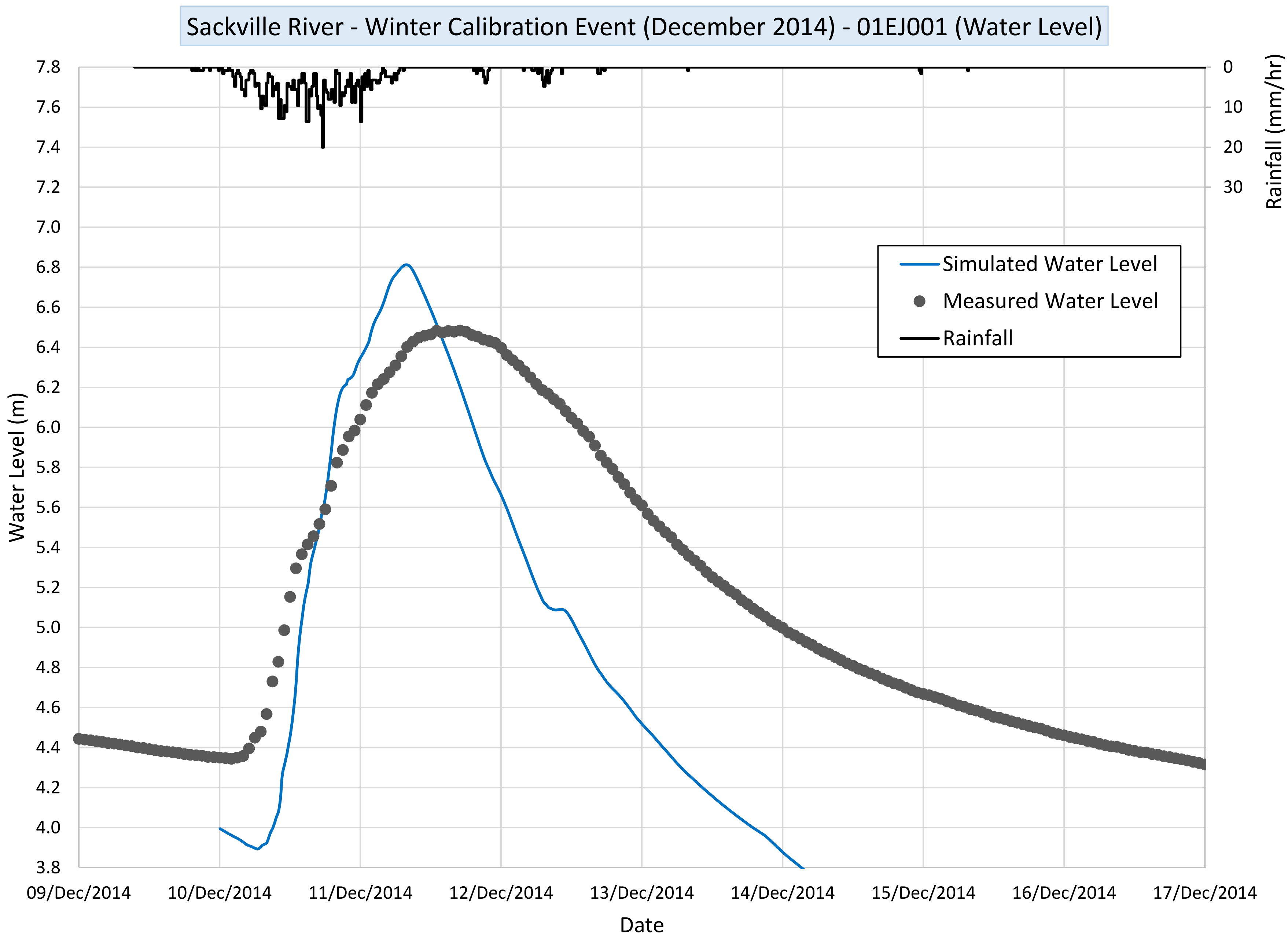
Model Calibration Results Graph #24



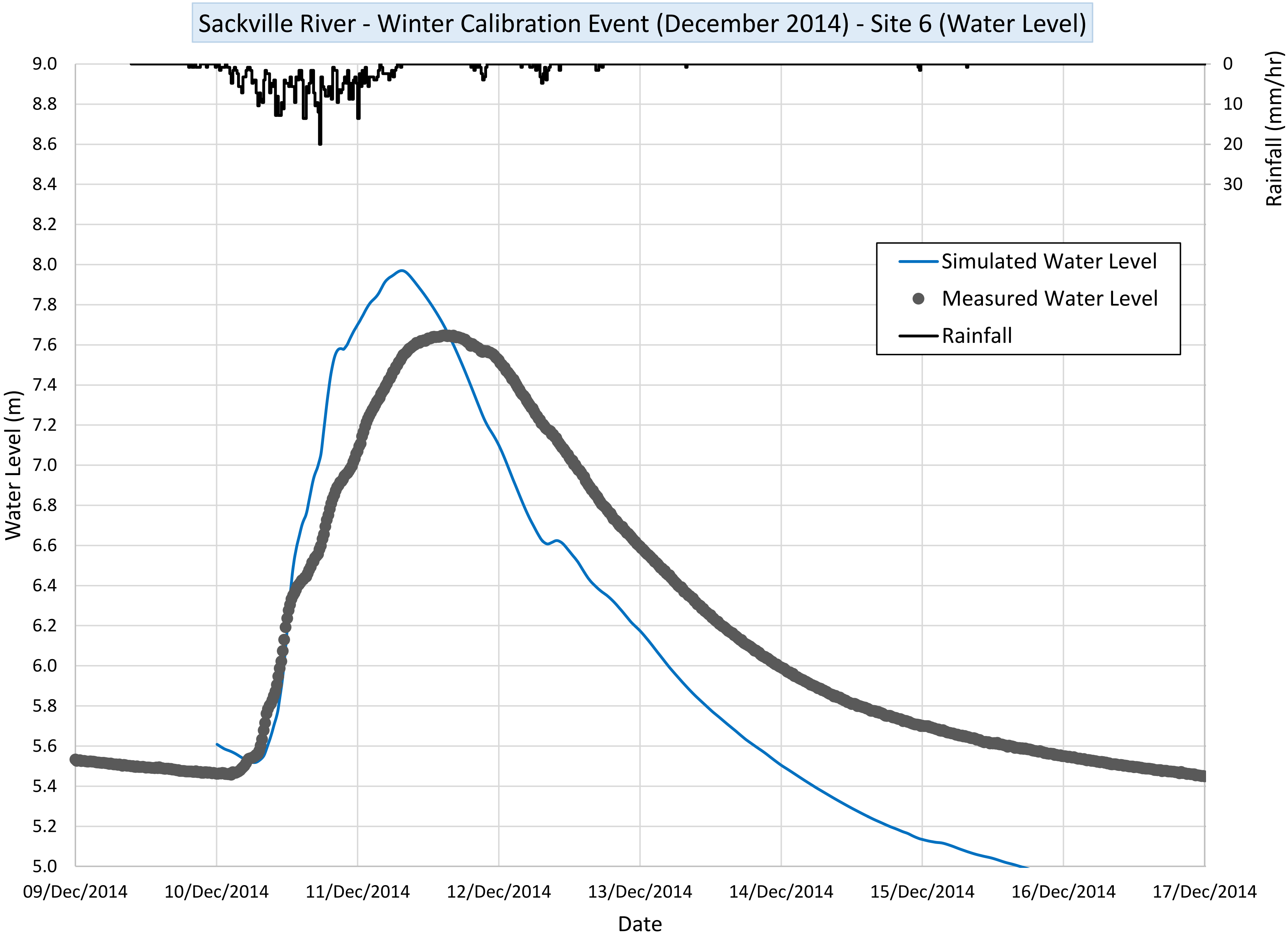
Model Calibration Results Graph #25



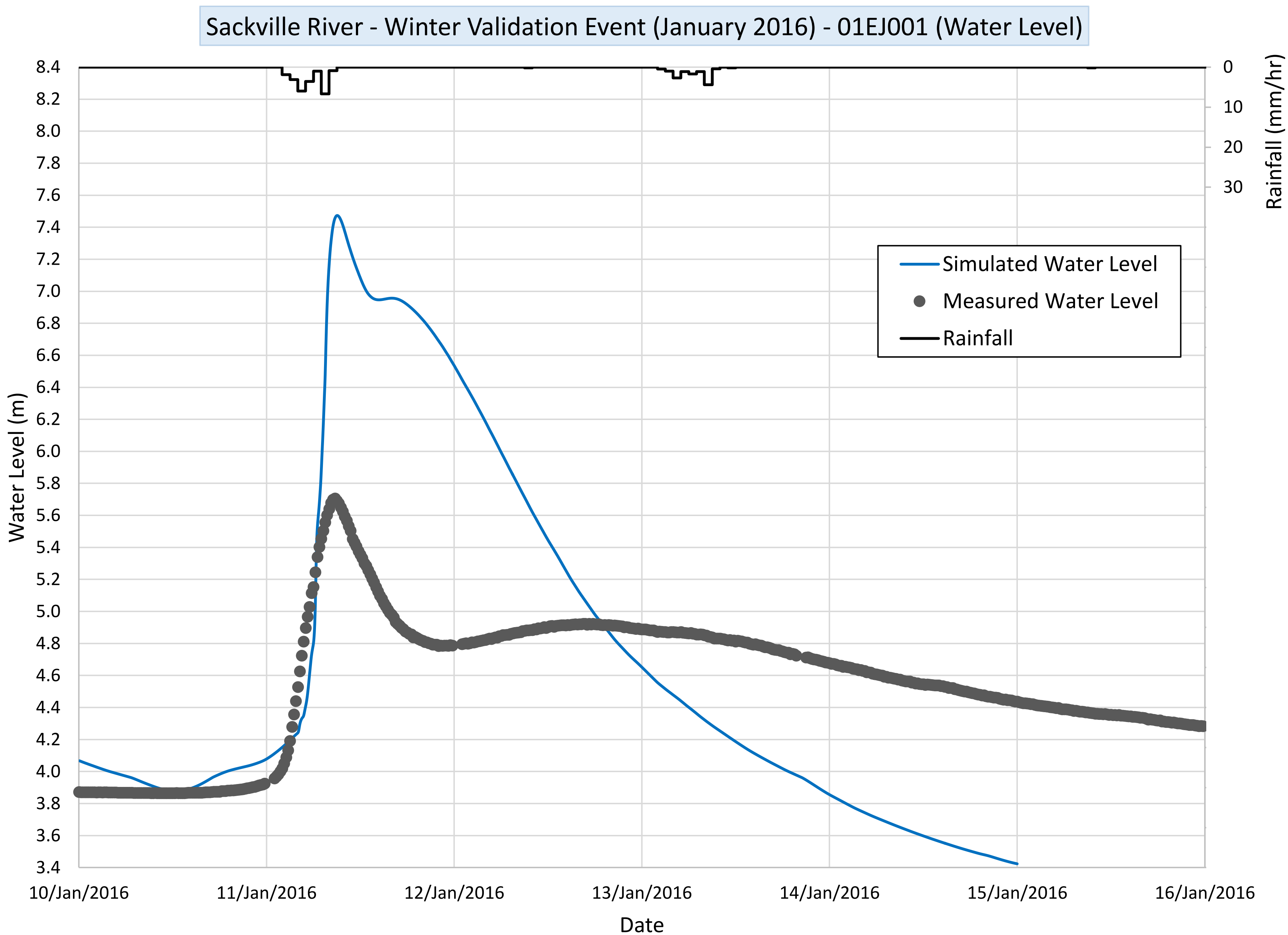
Model Calibration Results Graph #26



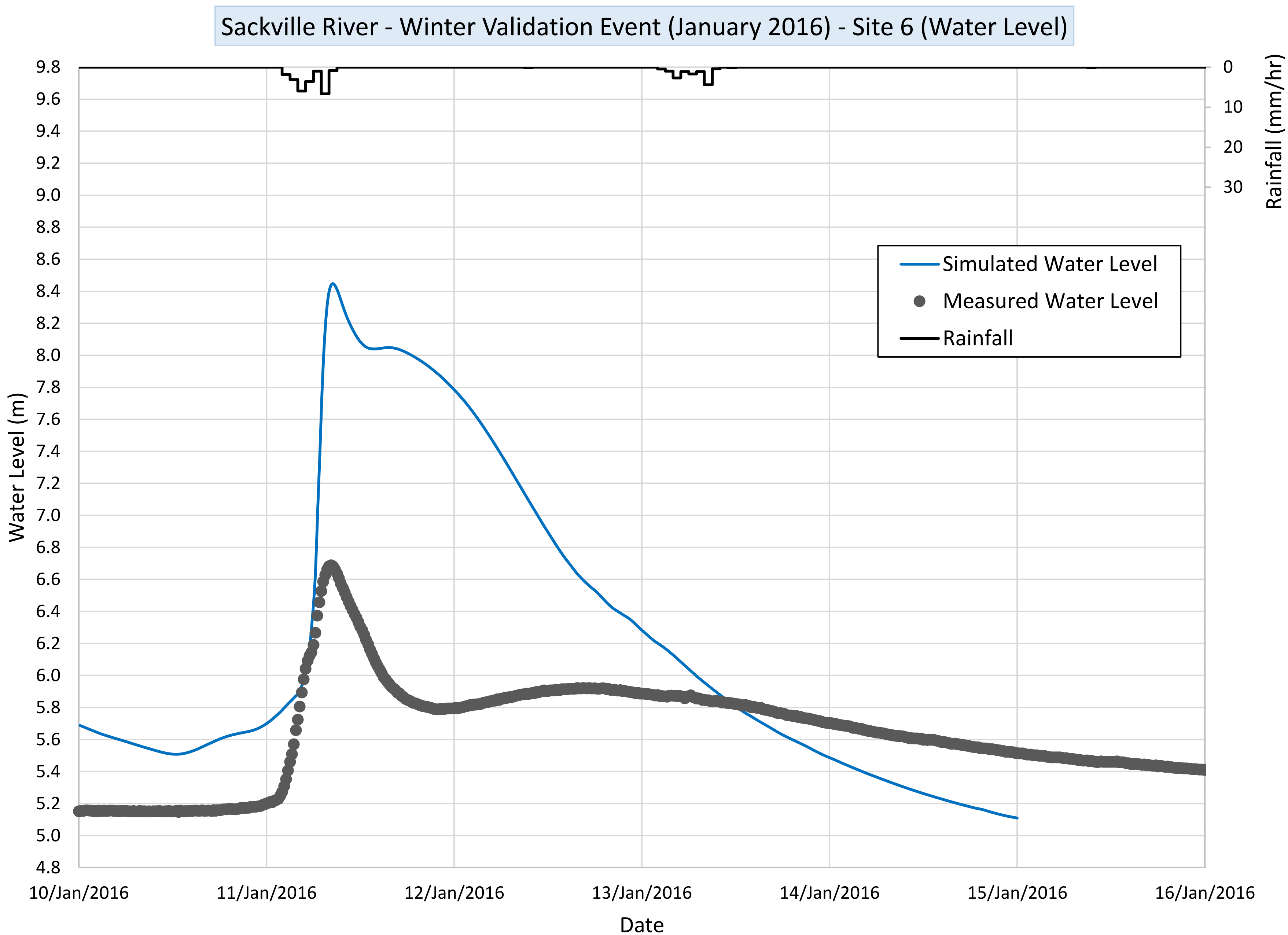
Model Calibration Results Graph #27



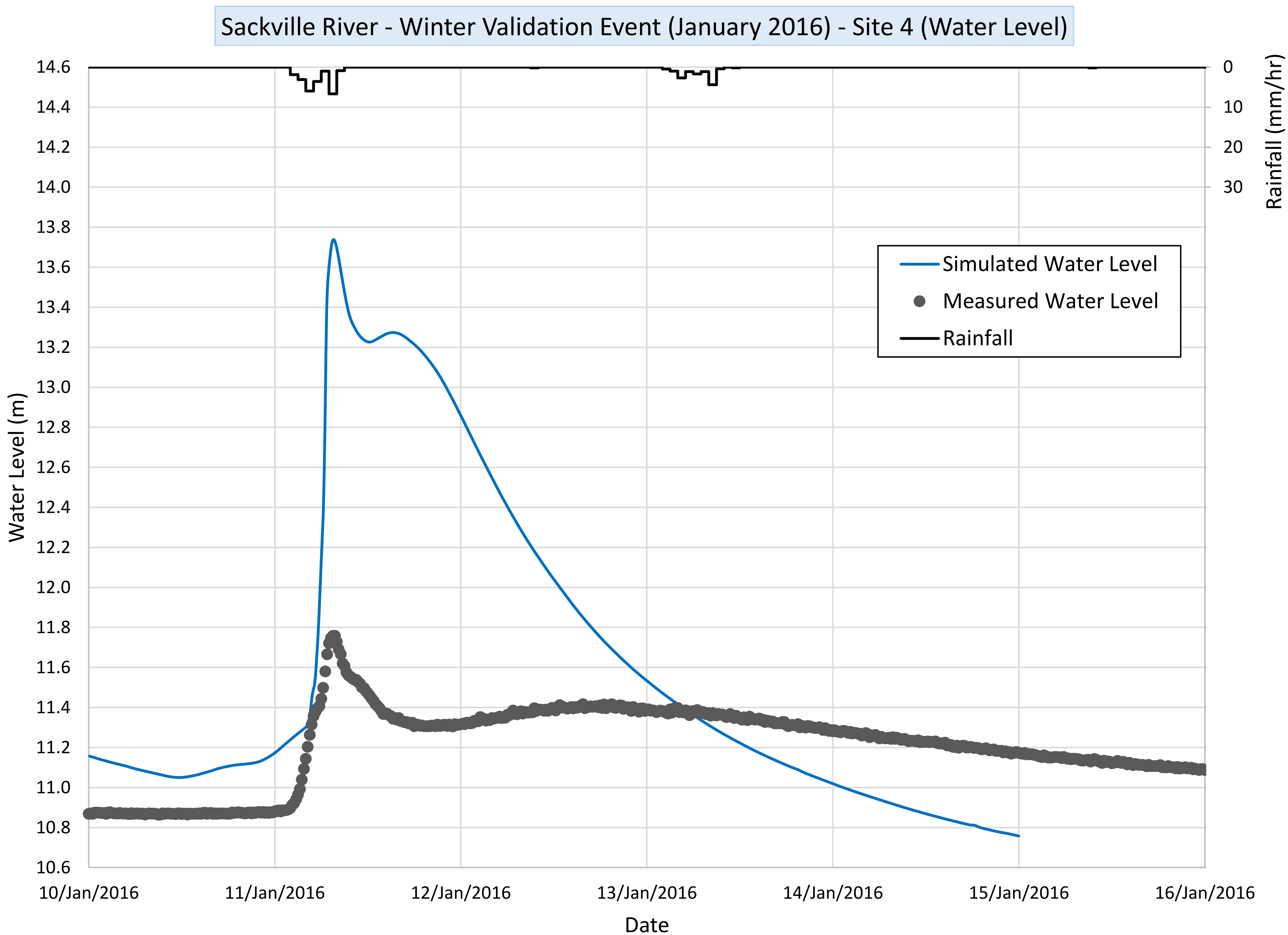
Model Calibration Results Graph #28



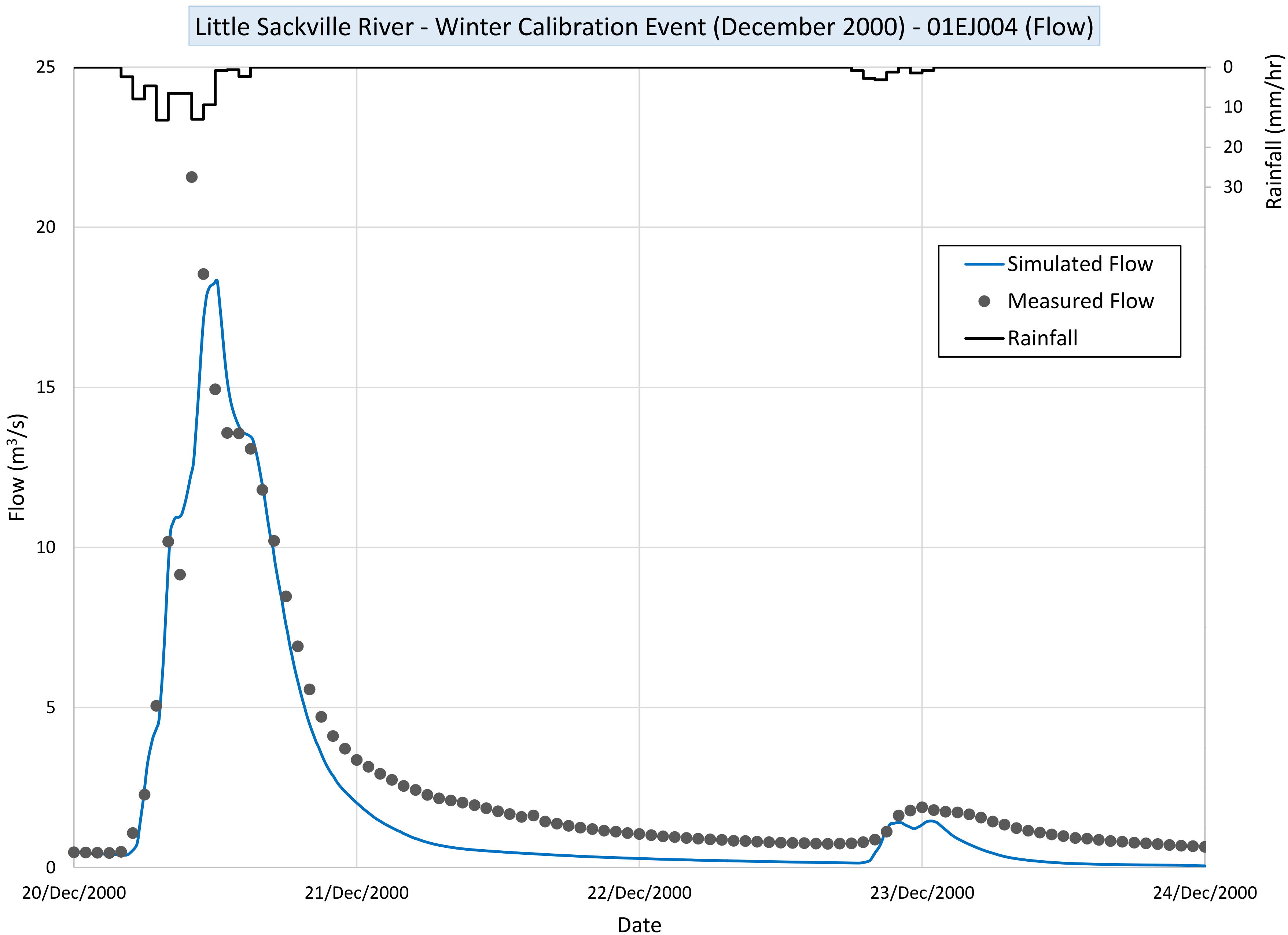
Model Calibration Results Graph #29



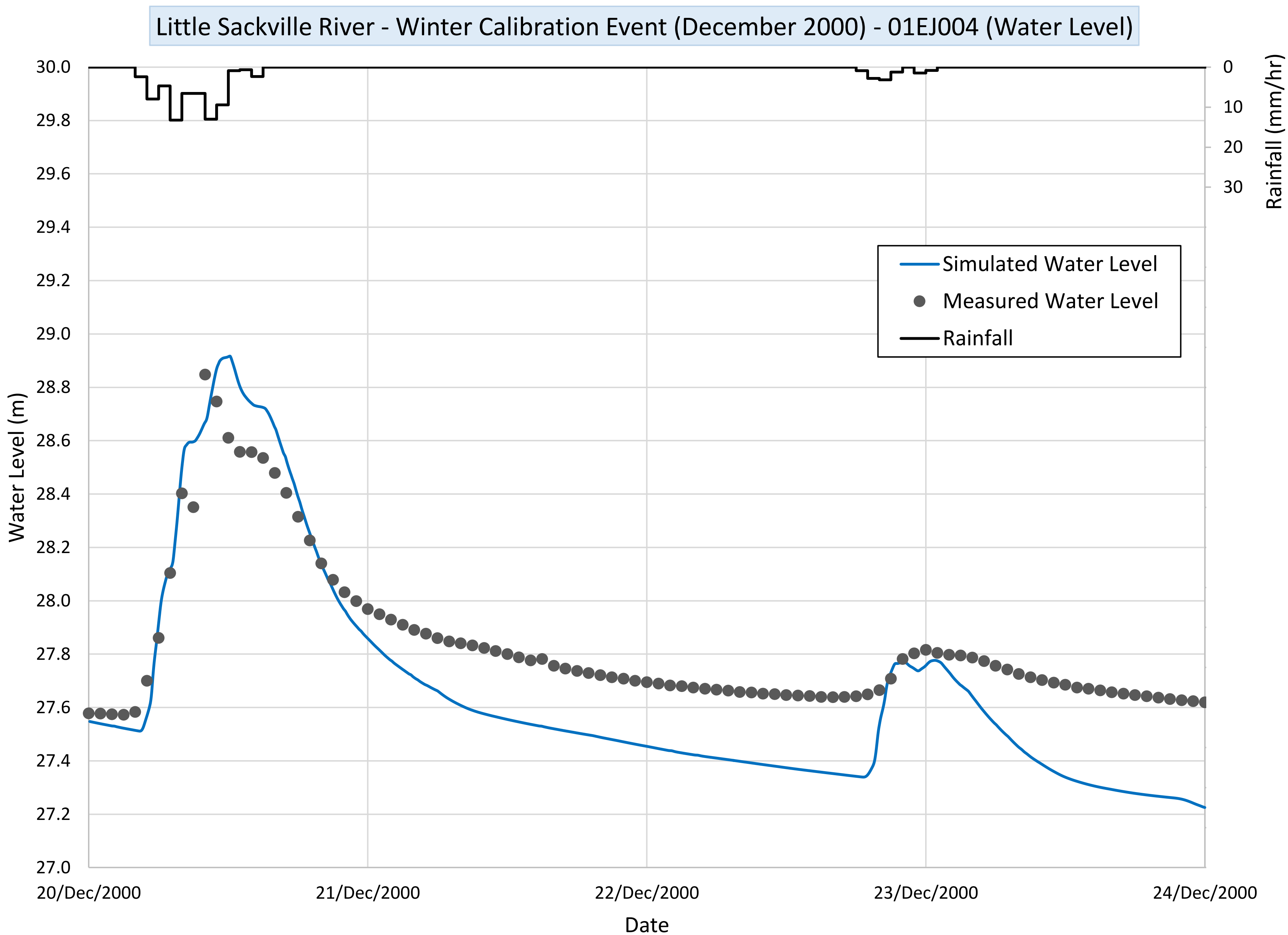
Model Calibration Results Graph #30



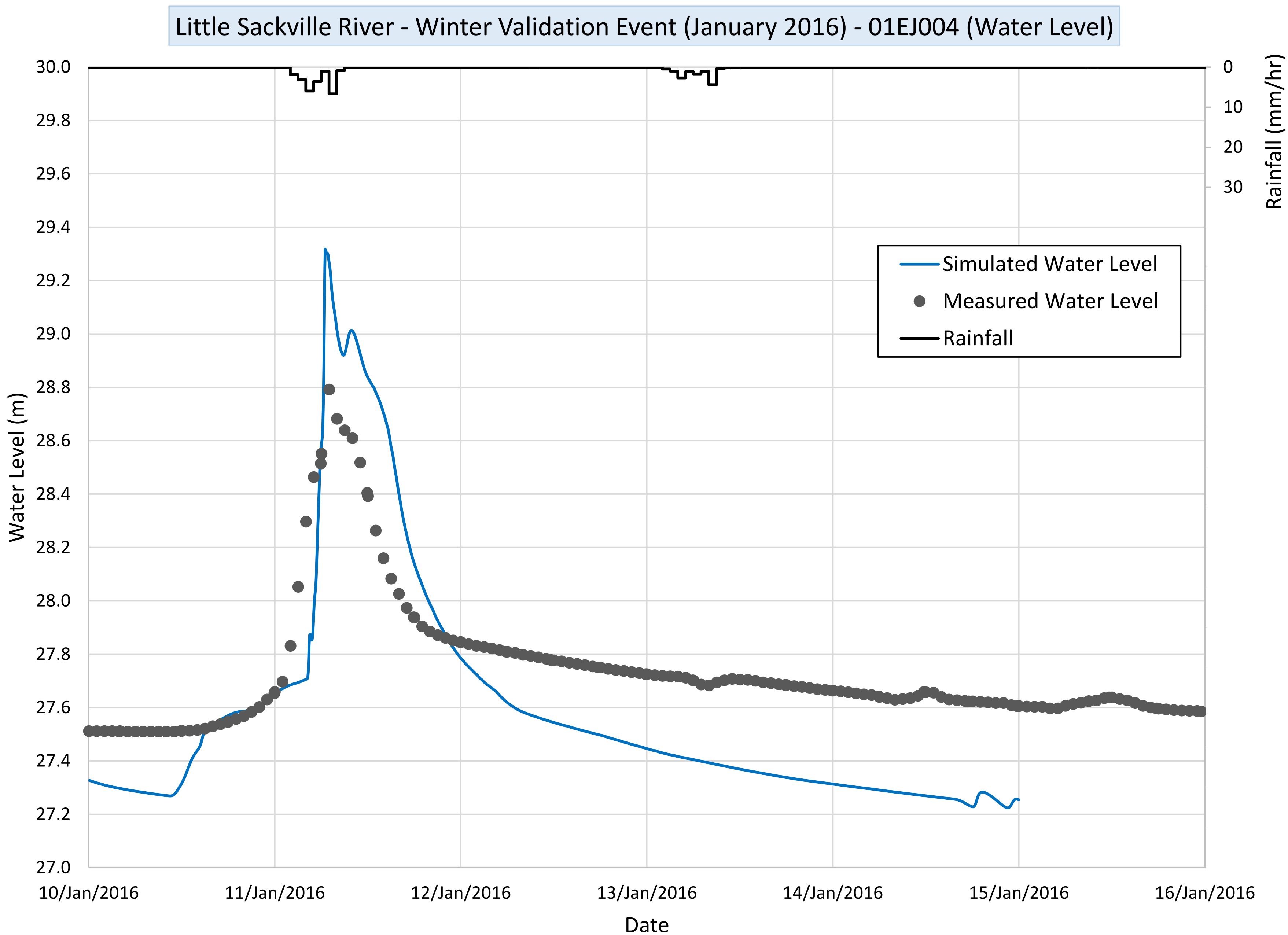
Model Calibration Results Graph #31



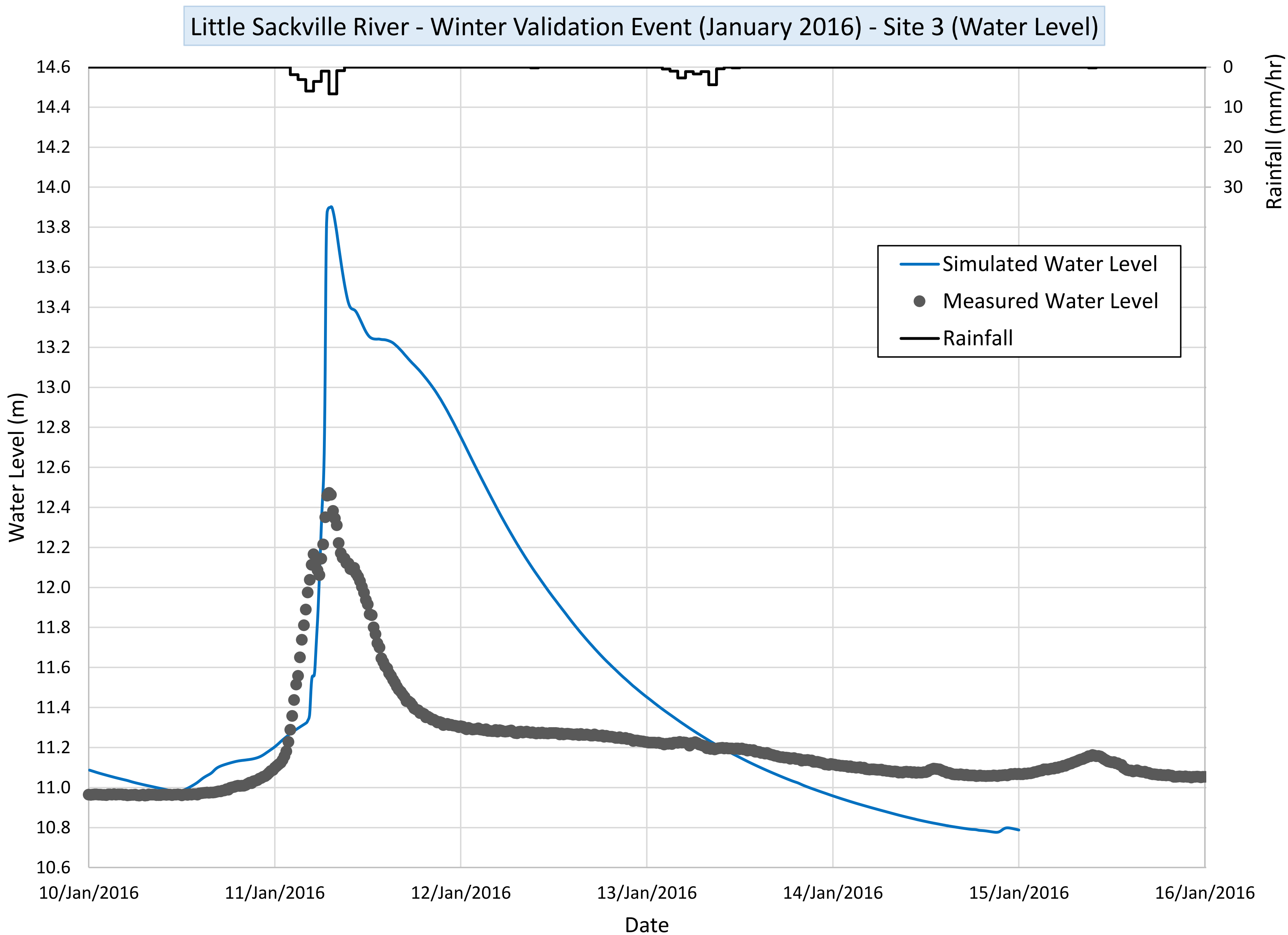
Model Calibration Results Graph #32



Model Calibration Results Graph #33



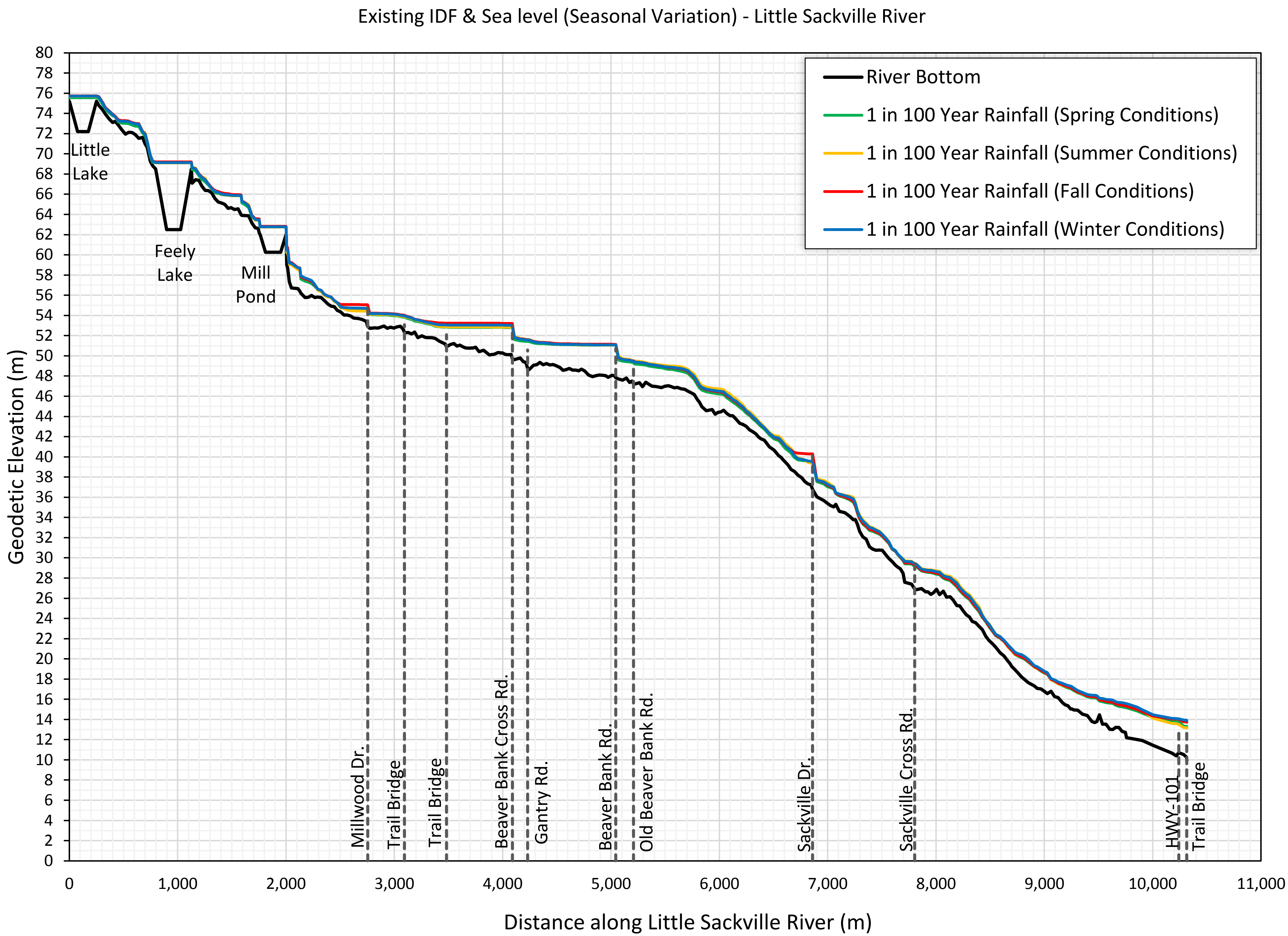
Model Calibration Results Graph #34



Model Calibration Results Graph #35

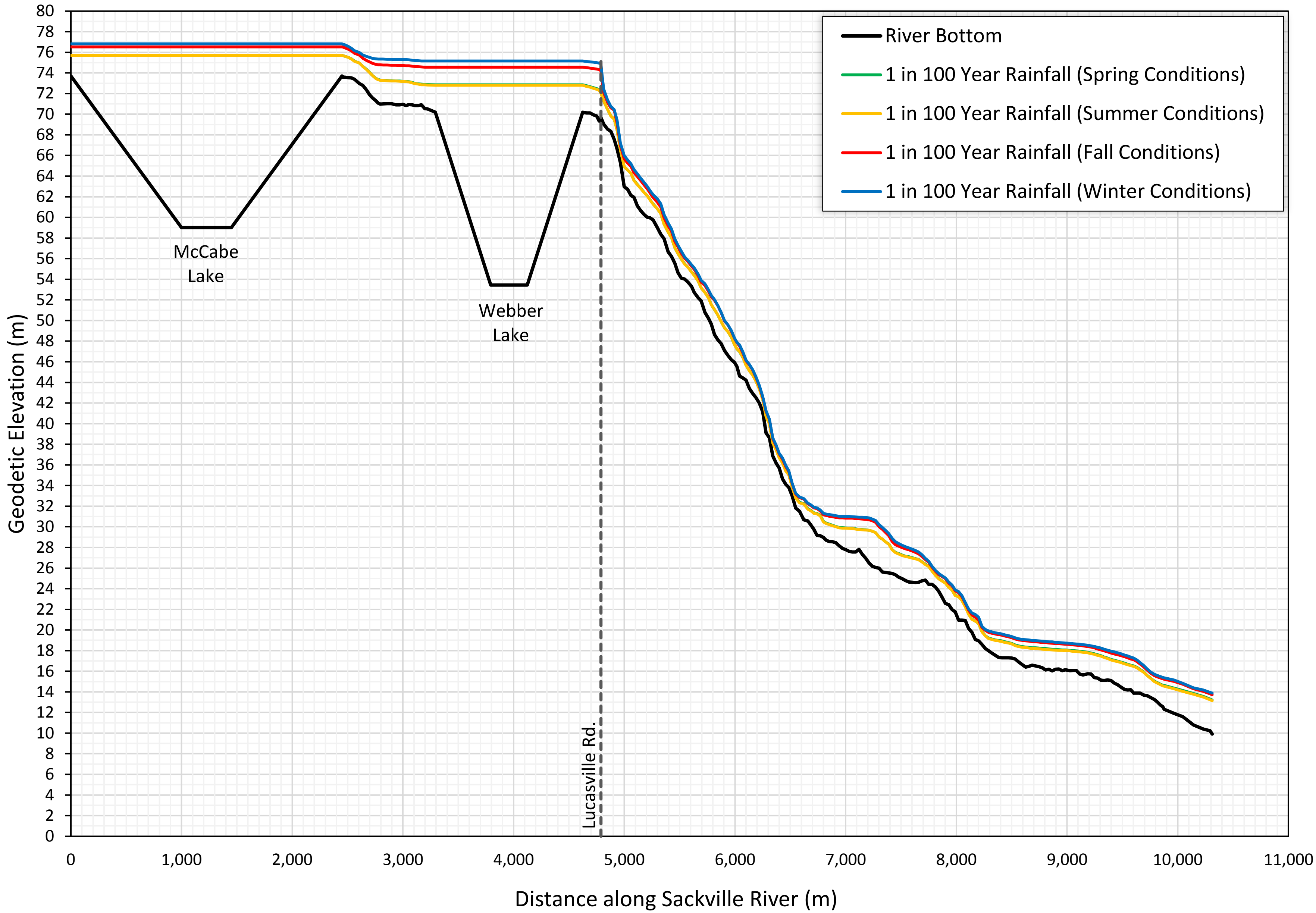
APPENDIX D

Water Elevation Profiles

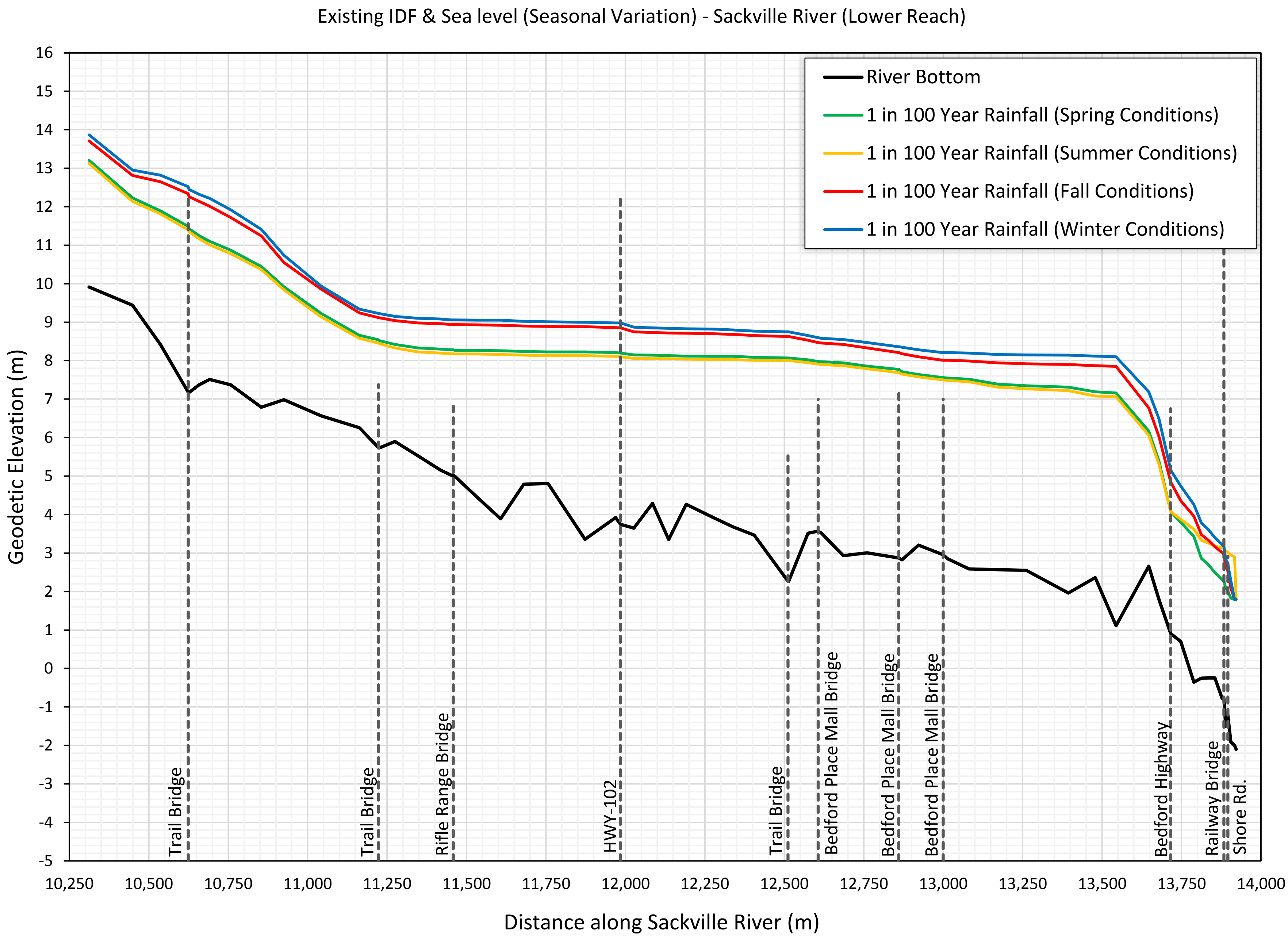


Existing IDF & Sea level (Seasonal Variation) - Little Sackville River
Geodetic Elevation (m)
Distance along Little Sackville River (m)
River Bottom
1 in 100 Year Rainfall (Spring Conditions)
1 in 100 Year Rainfall (Summer Conditions)
1 in 100 Year Rainfall (Fall Conditions)
1 in 100 Year Rainfall (Winter Conditions)
Little Lake
Feely Lake
Mill Pond
Millwood Dr.
Trail Bridge
Trail Bridge
Beaver Bank Cross Rd.
Gantry Rd.
Beaver Bank Rd.
Old Beaver Bank Rd.
Sackville Dr.
Sackville Cross Rd.
HWY-101
Trail Bridge

Existing IDF & Sea level (Seasonal Variation) - Sackville River (Upper Reach)

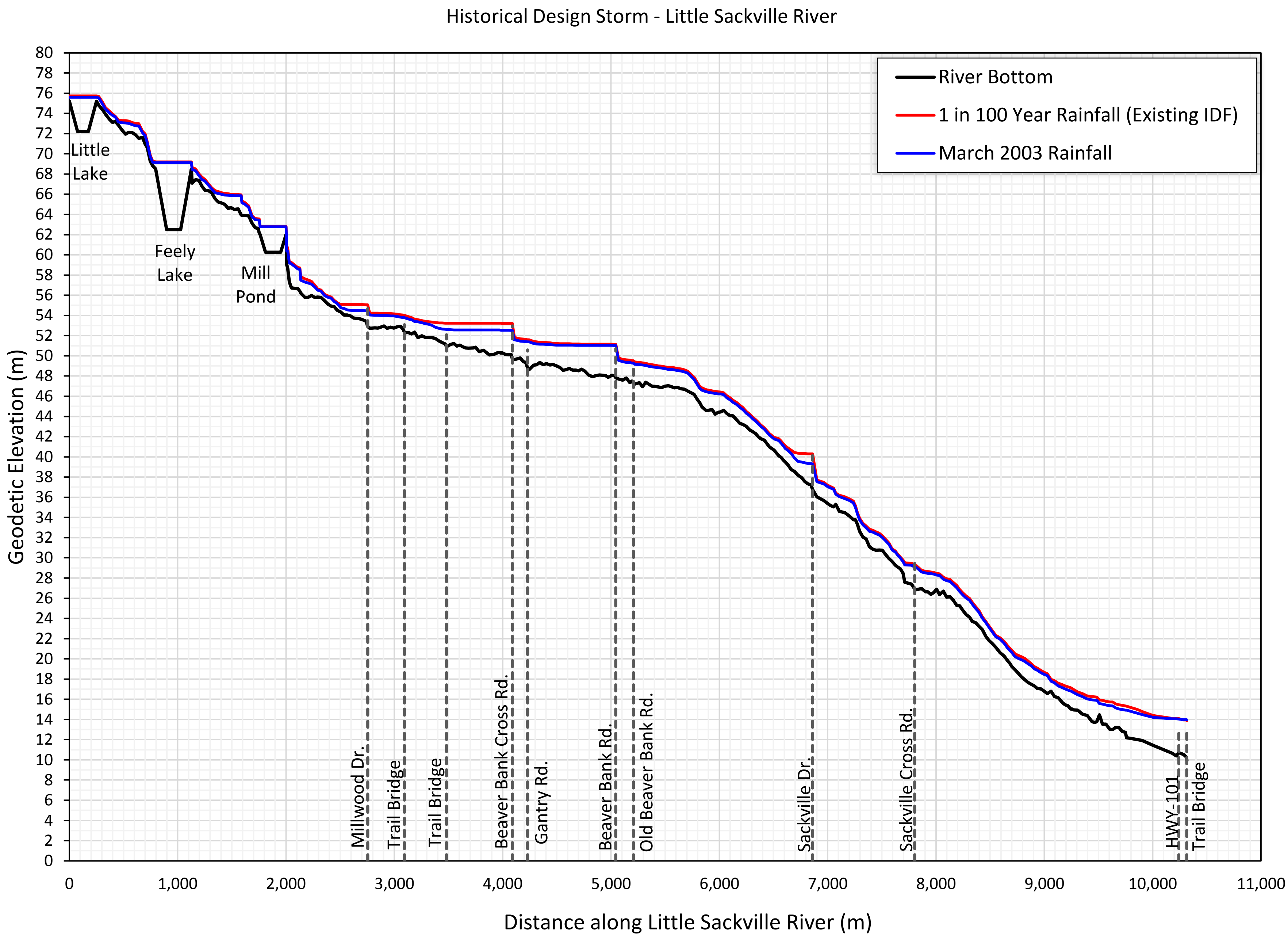


Profile #2

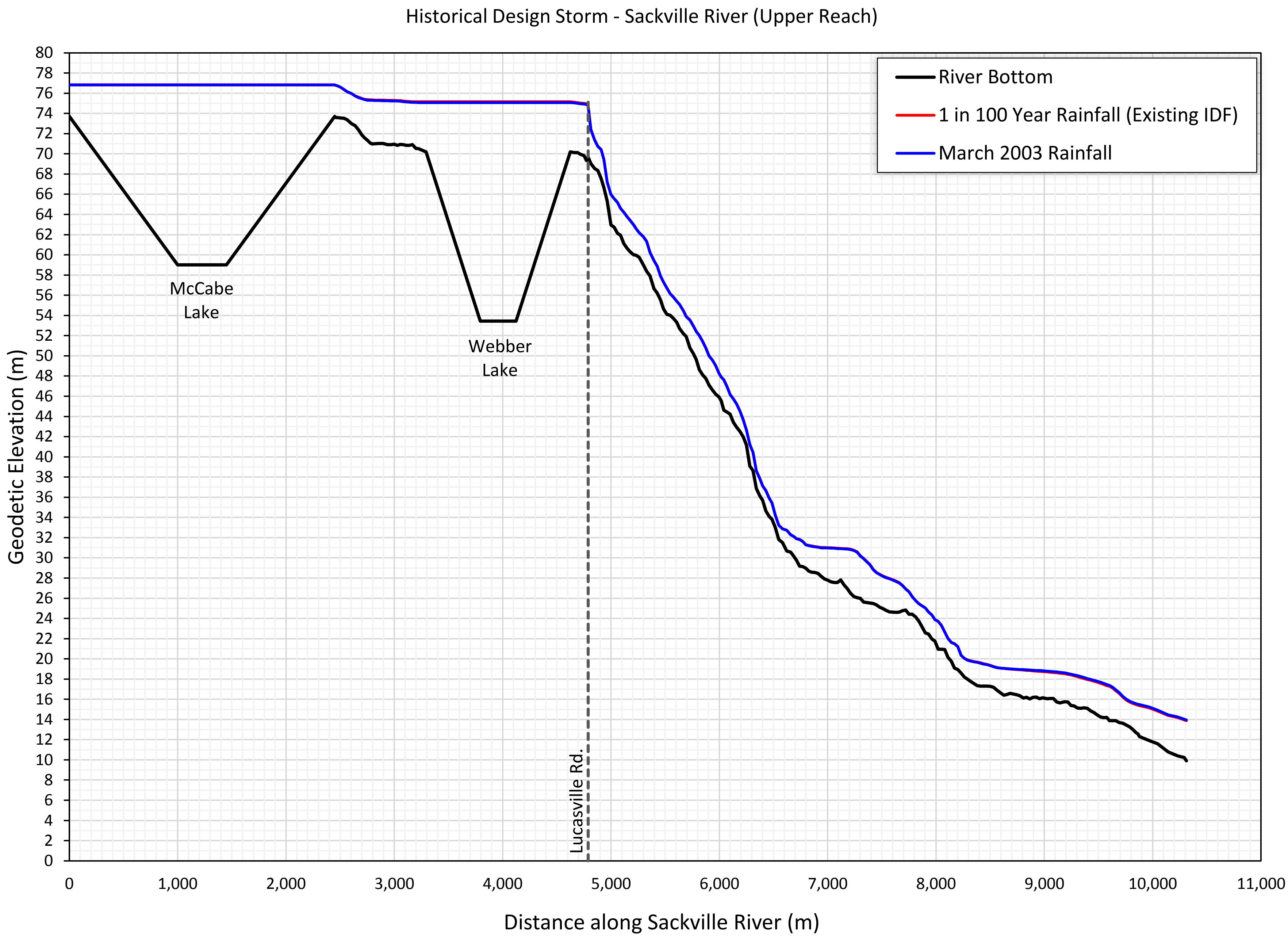


Existing IDF & Sea level (Seasonal Variation) - Sackville River (Lower Reach)
River Bottom
1 in 100 Year Rainfall (Spring Conditions)
1 in 100 Year Rainfall (Summer Conditions)
1 in 100 Year Rainfall (Fall Conditions)
1 in 100 Year Rainfall (Winter Conditions)
Geodetic Elevation (m)
Distance along Sackville River (m)
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
10,250
10,500
10,750
11,000
11,250
11,500
11,750
12,000
12,250
12,500
12,750
13,000
13,250
13,500
13,750
14,000
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.

Profile #3

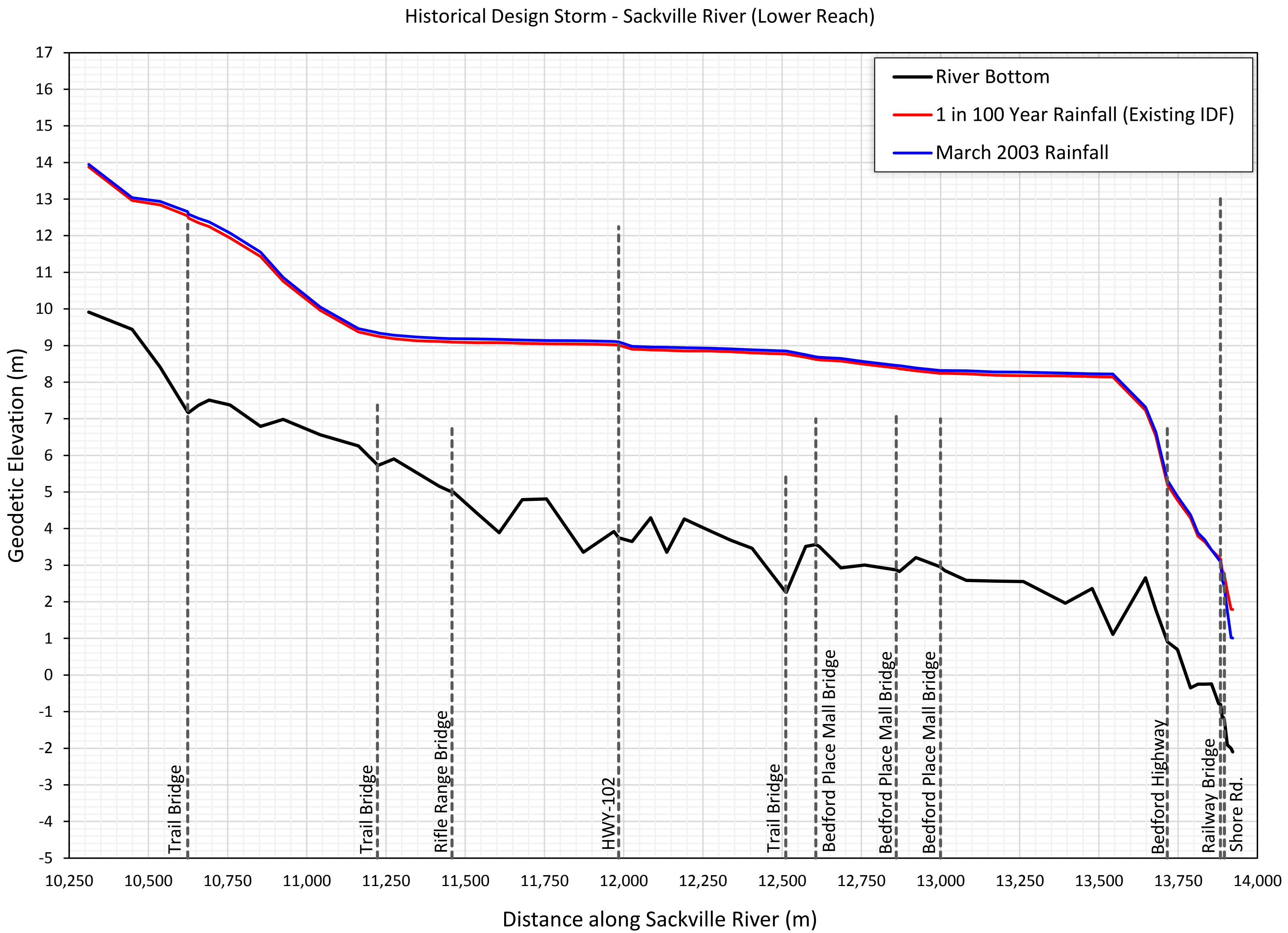

Historical Design Storm - Little Sackville River
Geodetic Elevation (m)
Distance along Little Sackville River (m)
River Bottom
1 in 100 Year Rainfall (Existing IDF)
March 2003 Rainfall
Little Lake
Feely Lake
Mill Pond
Millwood Dr.
Trail Bridge
Trail Bridge
Beaver Bank Cross Rd.
Gantry Rd.
Beaver Bank Rd.
Old Beaver Bank Rd.
Sackville Dr.
Sackville Cross Rd.
HWY-101
Trail Bridge

Profile #4



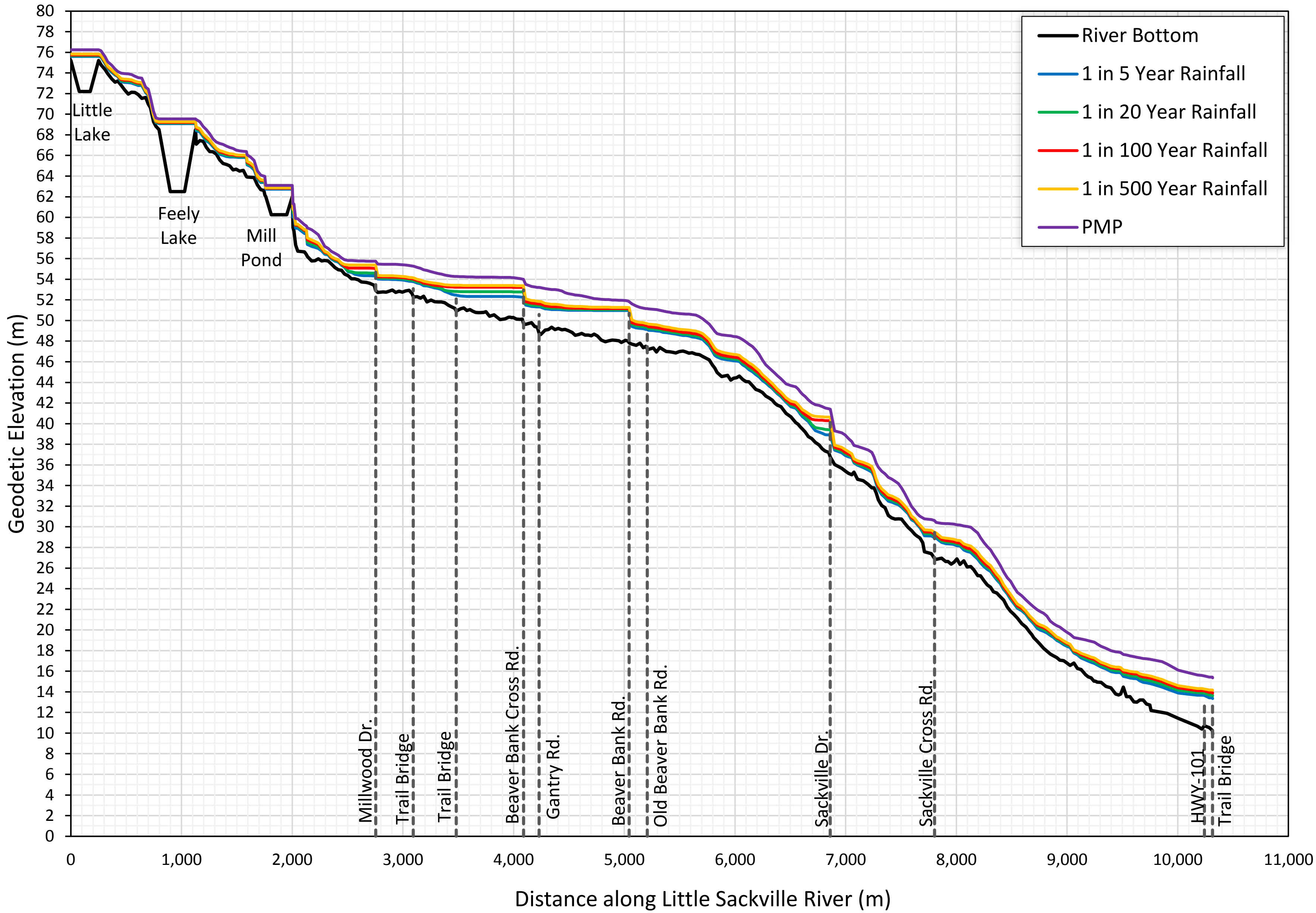
Historical Design Storm - Sackville River (Upper Reach)
River Bottom
1 in 100 Year Rainfall (Existing IDF)
March 2003 Rainfall
McCabe Lake
Webber Lake
Lucasville Rd.
Geodetic Elevation (m)
Distance along Sackville River (m)

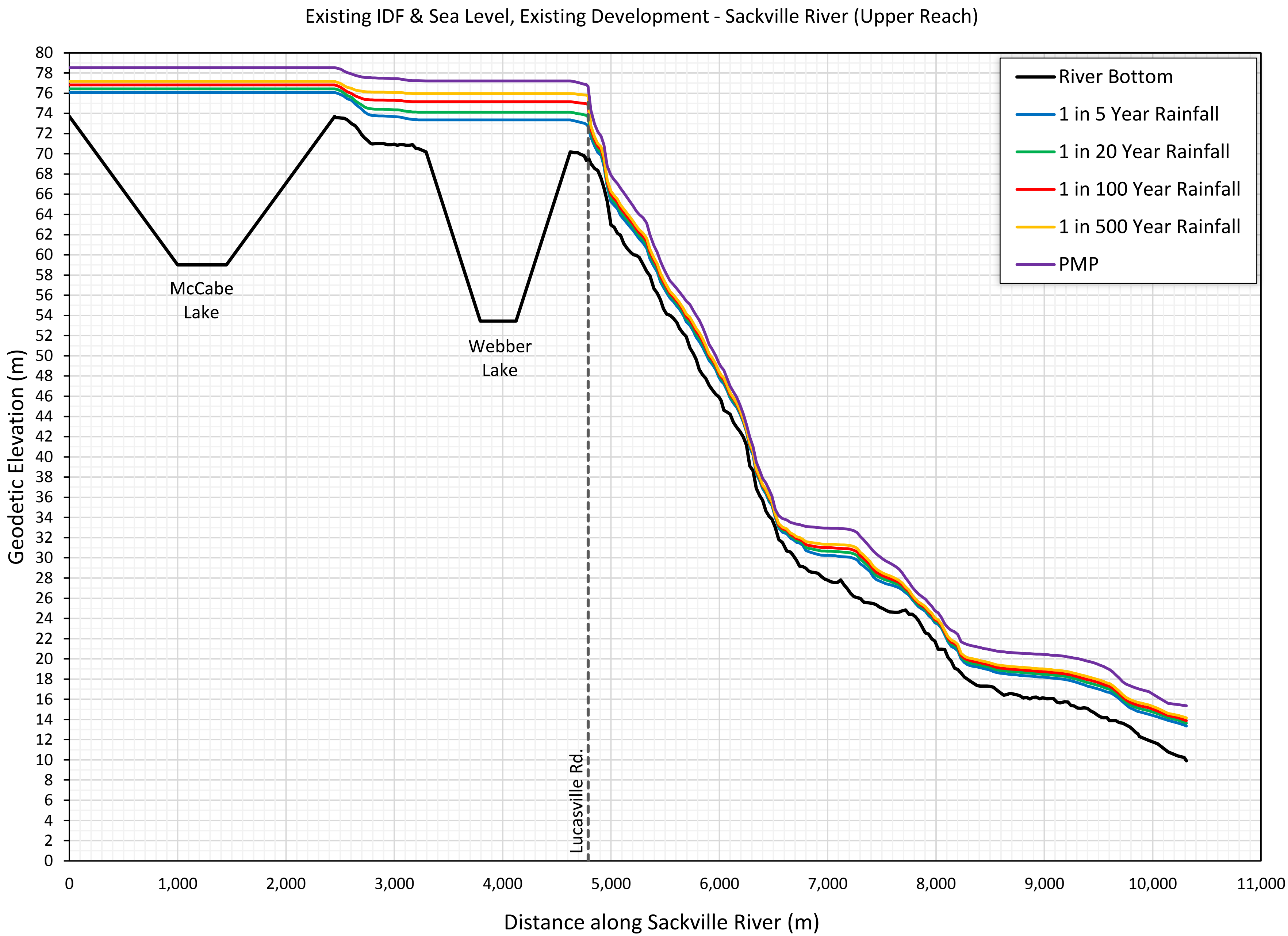
Profile #5



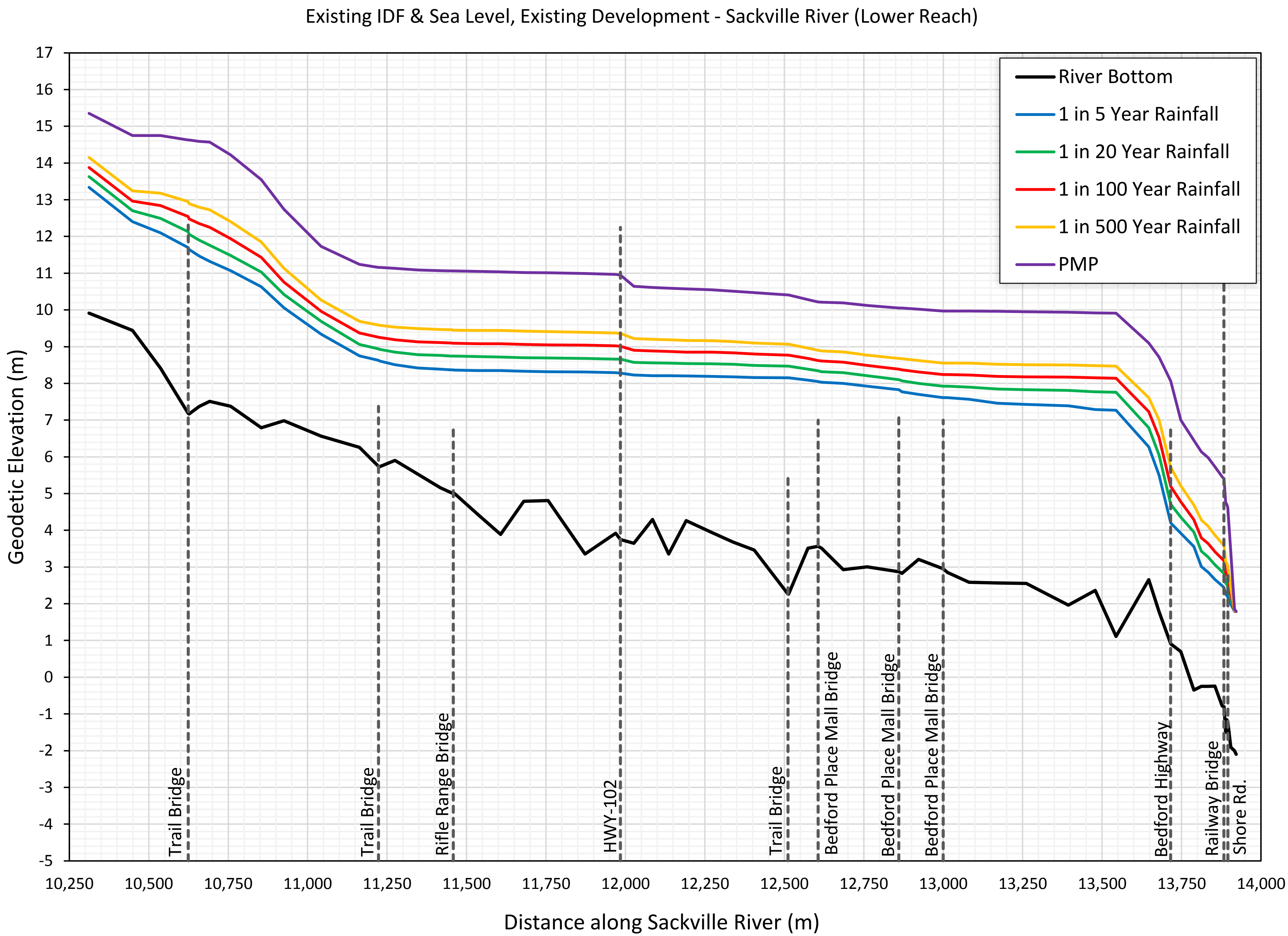
Historical Design Storm - Sackville River (Lower Reach)
Geodetic Elevation (m)
Distance along Sackville River (m)
River Bottom
1 in 100 Year Rainfall (Existing IDF)
March 2003 Rainfall
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.

Existing IDF & Sea Level, Existing Development - Little Sackville River




Existing IDF & Sea Level, Existing Development - Sackville River (Upper Reach)
Geodetic Elevation (m)
Distance along Sackville River (m)
River Bottom
1 in 5 Year Rainfall
1 in 20 Year Rainfall
1 in 100 Year Rainfall
1 in 500 Year Rainfall
PMP
McCabe Lake
Webber Lake
Lucasville Rd.

Profile #8



Existing IDF & Sea Level, Existing Development - Sackville River (Lower Reach)
Geodetic Elevation (m)
Distance along Sackville River (m)
River Bottom
1 in 5 Year Rainfall
1 in 20 Year Rainfall
1 in 100 Year Rainfall
1 in 500 Year Rainfall
PMP
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.
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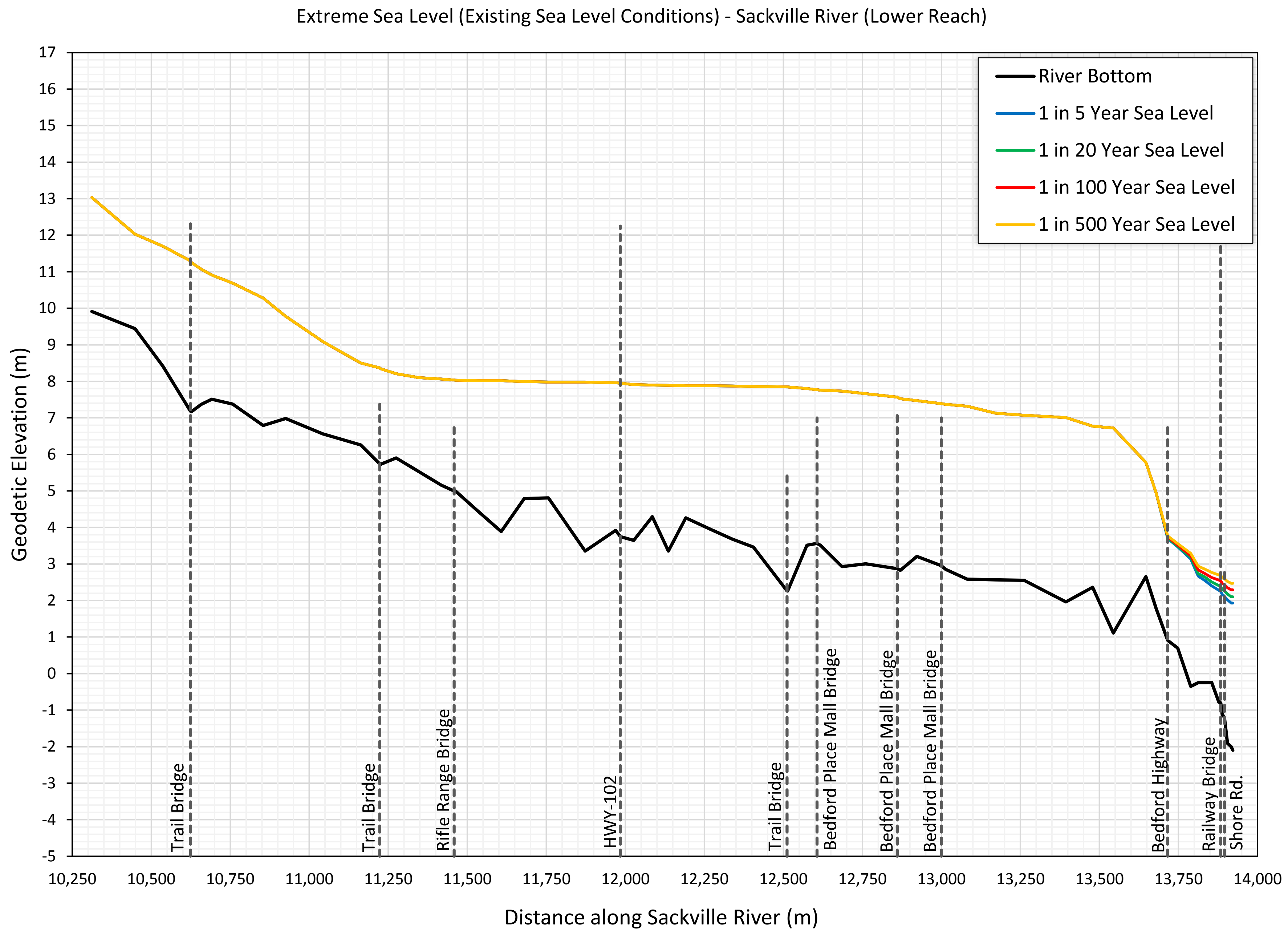
Profile #9



Extreme Sea Level (Existing Sea Level Conditions) - Sackville River (Lower Reach)
Geodetic Elevation (m)
Distance along Sackville River (m)
River Bottom
1 in 5 Year Sea Level
1 in 20 Year Sea Level
1 in 100 Year Sea Level
1 in 500 Year Sea Level
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.
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-1
-2
-3
-4
-5
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10,500
10,750
11,000
11,250
11,500
11,750
12,000
12,250
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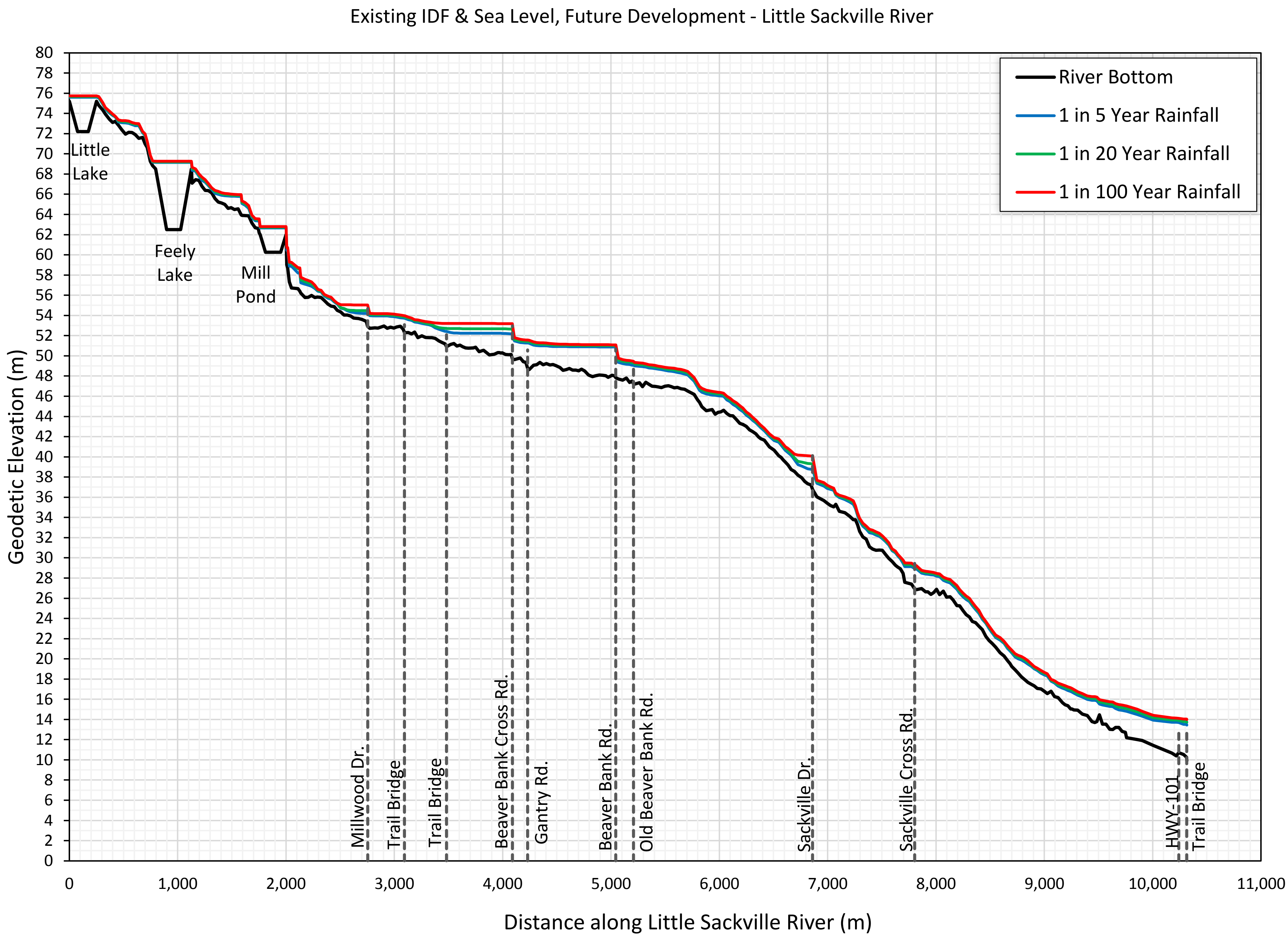
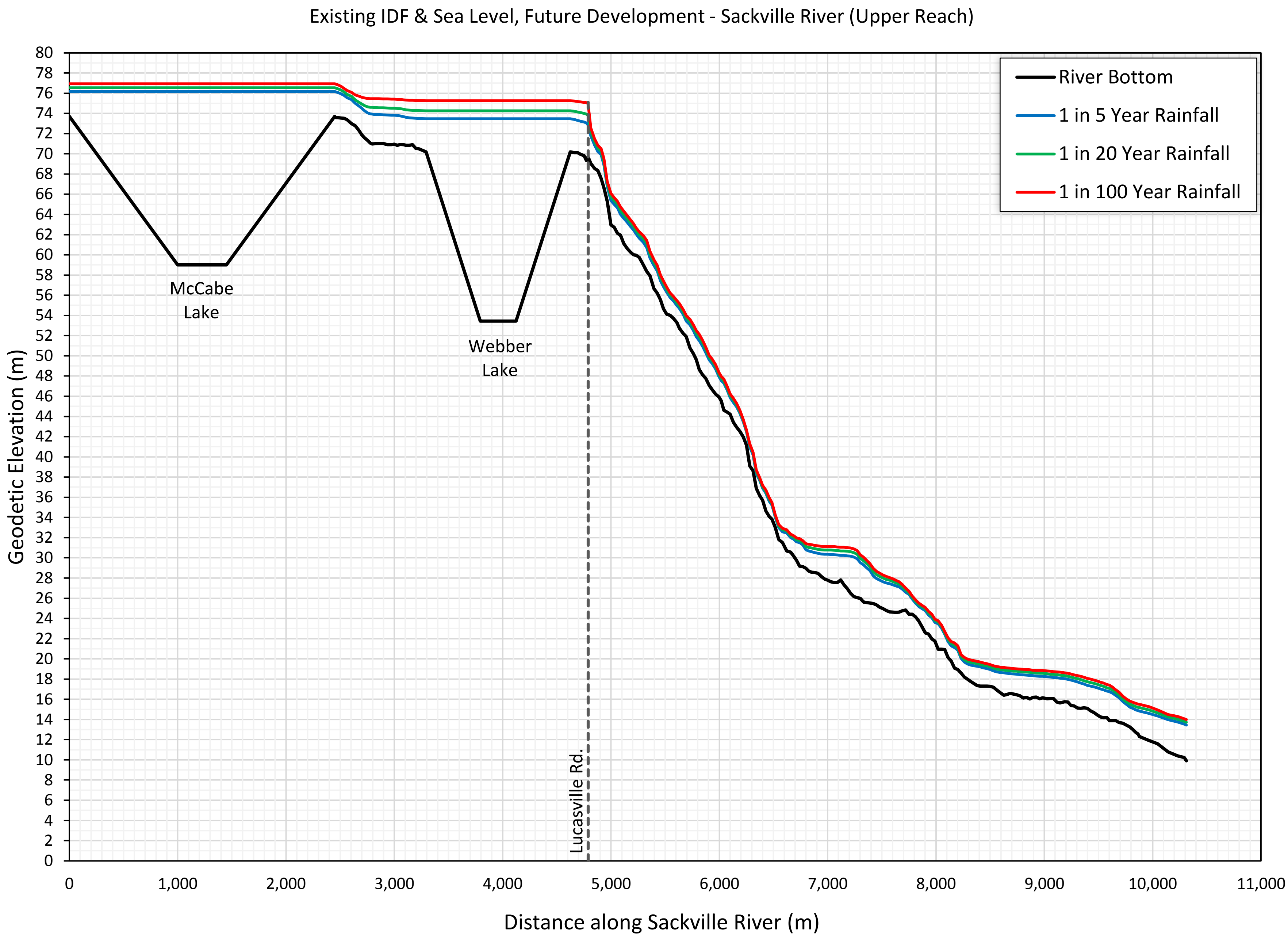
13,250
13,500
13,750
14,000



Existing IDF & Sea Level, Future Development - Little Sackville River
Geodetic Elevation (m)
Distance along Little Sackville River (m)
River Bottom
1 in 5 Year Rainfall
1 in 20 Year Rainfall
1 in 100 Year Rainfall
Little Lake
Feely Lake
Mill Pond
Millwood Dr.
Trail Bridge
Trail Bridge
Beaver Bank Cross Rd.
Gantry Rd.
Beaver Bank Rd.
Old Beaver Bank Rd.
Sackville Dr.
Sackville Cross Rd.
HWY-101
Trail Bridge

Profile #11


Existing IDF & Sea Level, Future Development - Sackville River (Upper Reach)
Geodetic Elevation (m)
Distance along Sackville River (m)
River Bottom
1 in 5 Year Rainfall
1 in 20 Year Rainfall
1 in 100 Year Rainfall
McCabe Lake
Webber Lake
Lucasville Rd.
0
1,000
2,000
3,000
4,000
5,000
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7,000
8,000
9,000
10,000
11,000
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76
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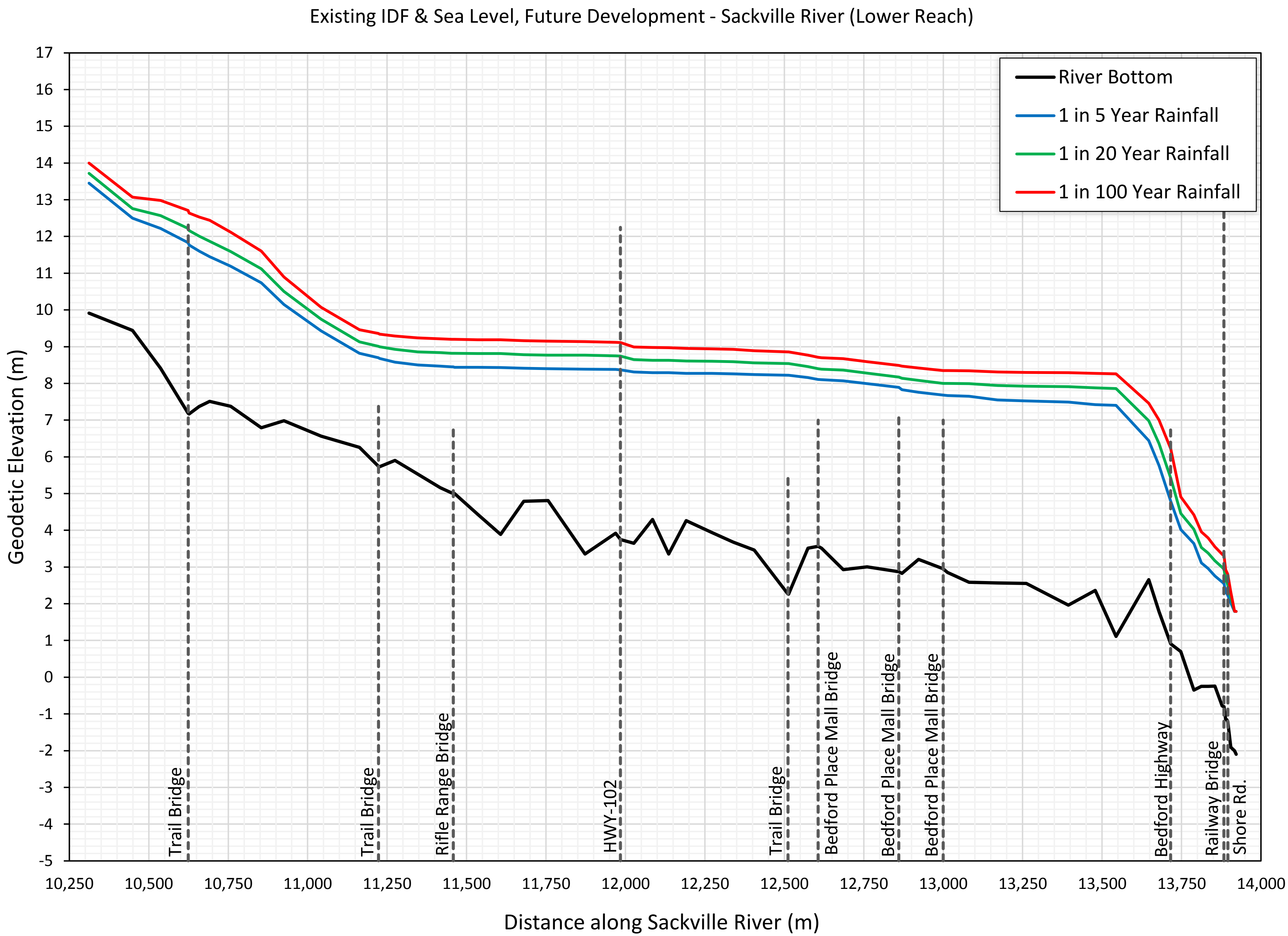
Existing IDF & Sea Level, Future Development - Sackville River (Lower Reach)
River Bottom
1 in 5 Year Rainfall
1 in 20 Year Rainfall
1 in 100 Year Rainfall
Geodetic Elevation (m)
Distance along Sackville River (m)
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.
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13,750
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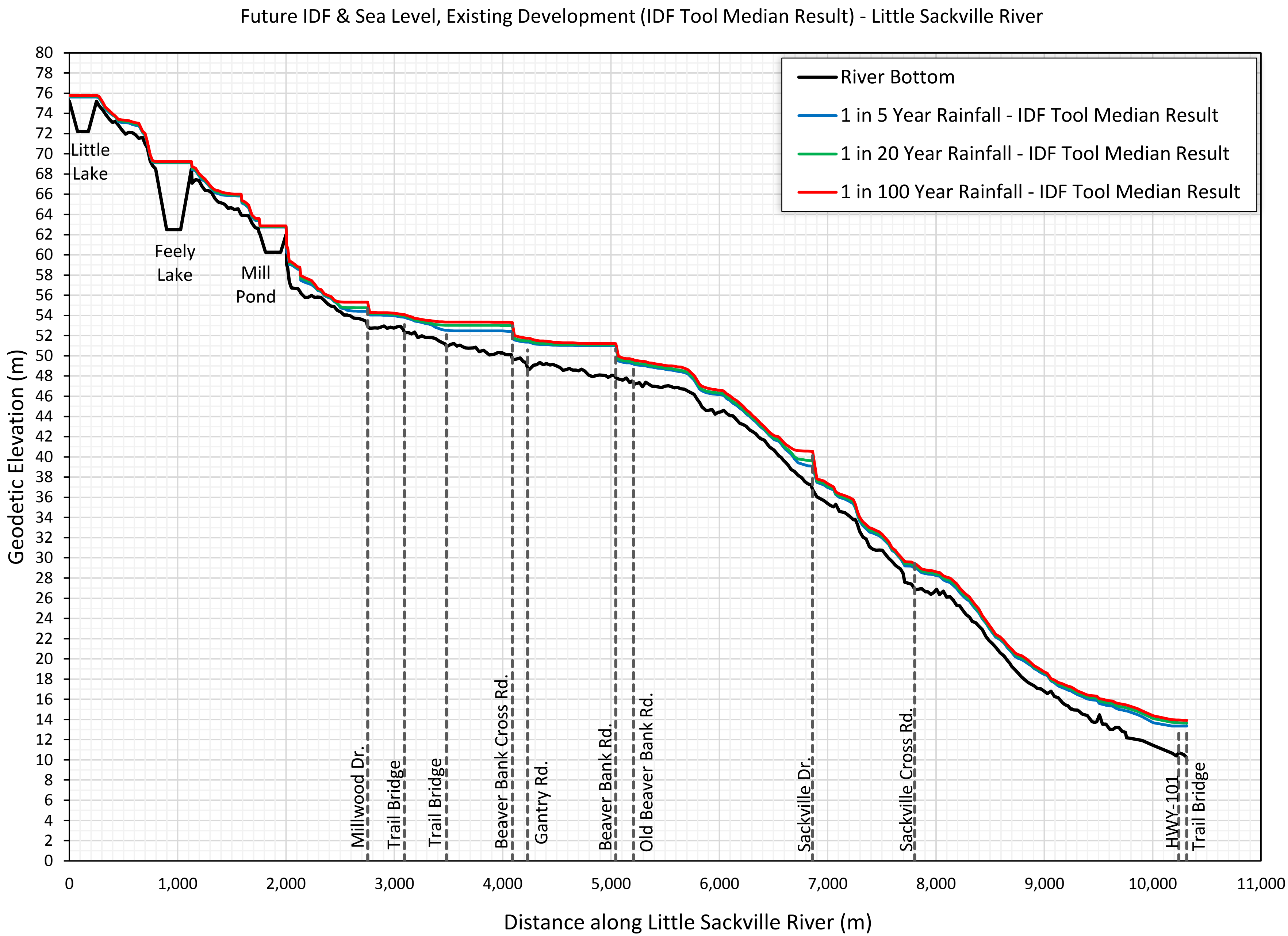
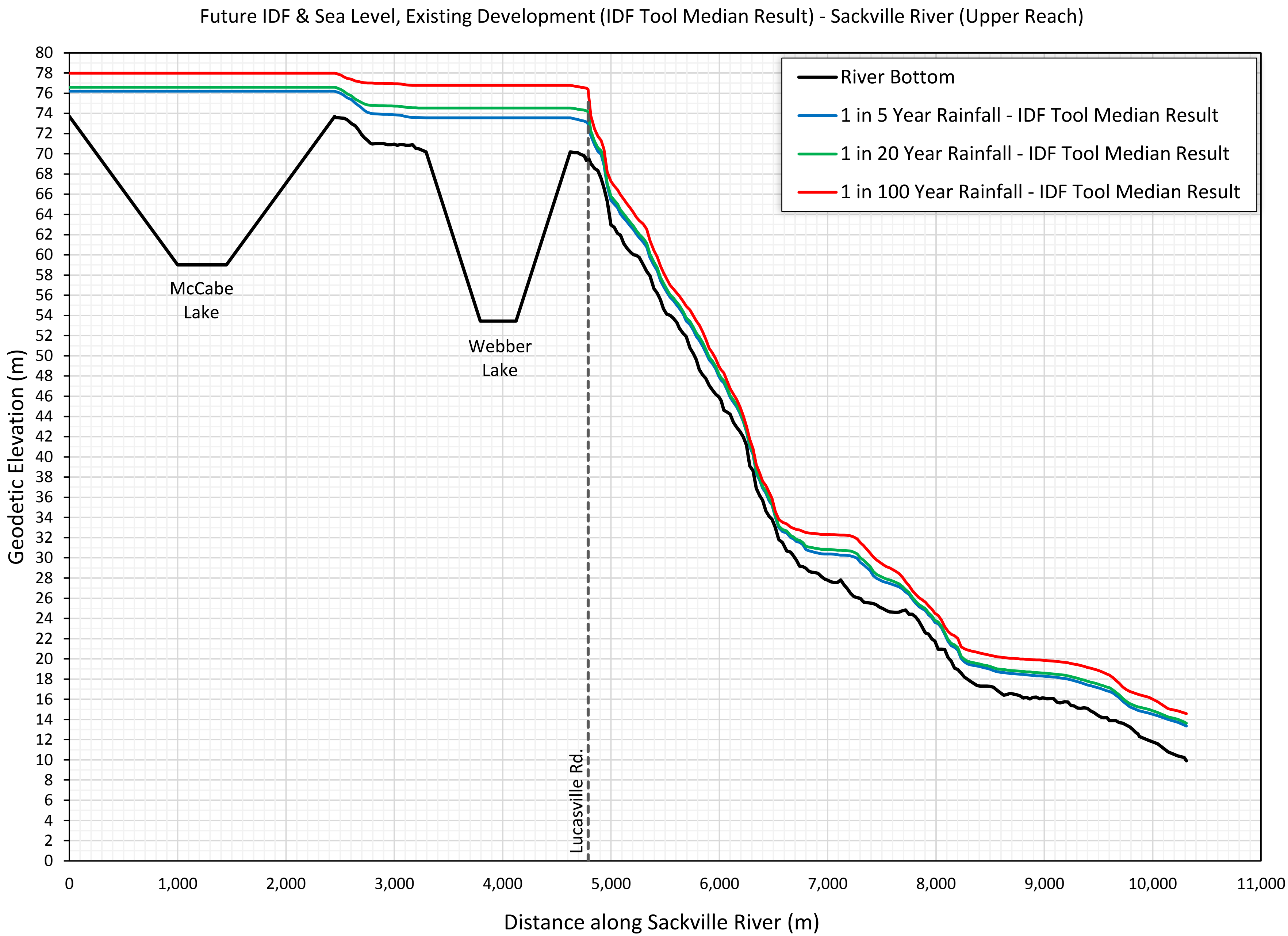

Future IDF & Sea Level, Existing Development (IDF Tool Median Result) - Little Sackville River
River Bottom
1 in 5 Year Rainfall - IDF Tool Median Result
1 in 20 Year Rainfall - IDF Tool Median Result
1 in 100 Year Rainfall - IDF Tool Median Result
Geodetic Elevation (m)
Distance along Little Sackville River (m)
Little Lake
Feely Lake
Mill Pond
Millwood Dr.
Trail Bridge
Trail Bridge
Beaver Bank Cross Rd.
Gantry Rd.
Beaver Bank Rd.
Old Beaver Bank Rd.
Sackville Dr.
Sackville Cross Rd.
HWY-101
Trail Bridge



Future IDF & Sea Level, Existing Development (IDF Tool Median Result) - Sackville River (Upper Reach)
River Bottom
1 in 5 Year Rainfall - IDF Tool Median Result
1 in 20 Year Rainfall - IDF Tool Median Result
1 in 100 Year Rainfall - IDF Tool Median Result
McCabe Lake
Webber Lake
Lucasville Rd.
Geodetic Elevation (m)
Distance along Sackville River (m)
0
1,000
2,000
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6,000
7,000
8,000
9,000
10,000
11,000
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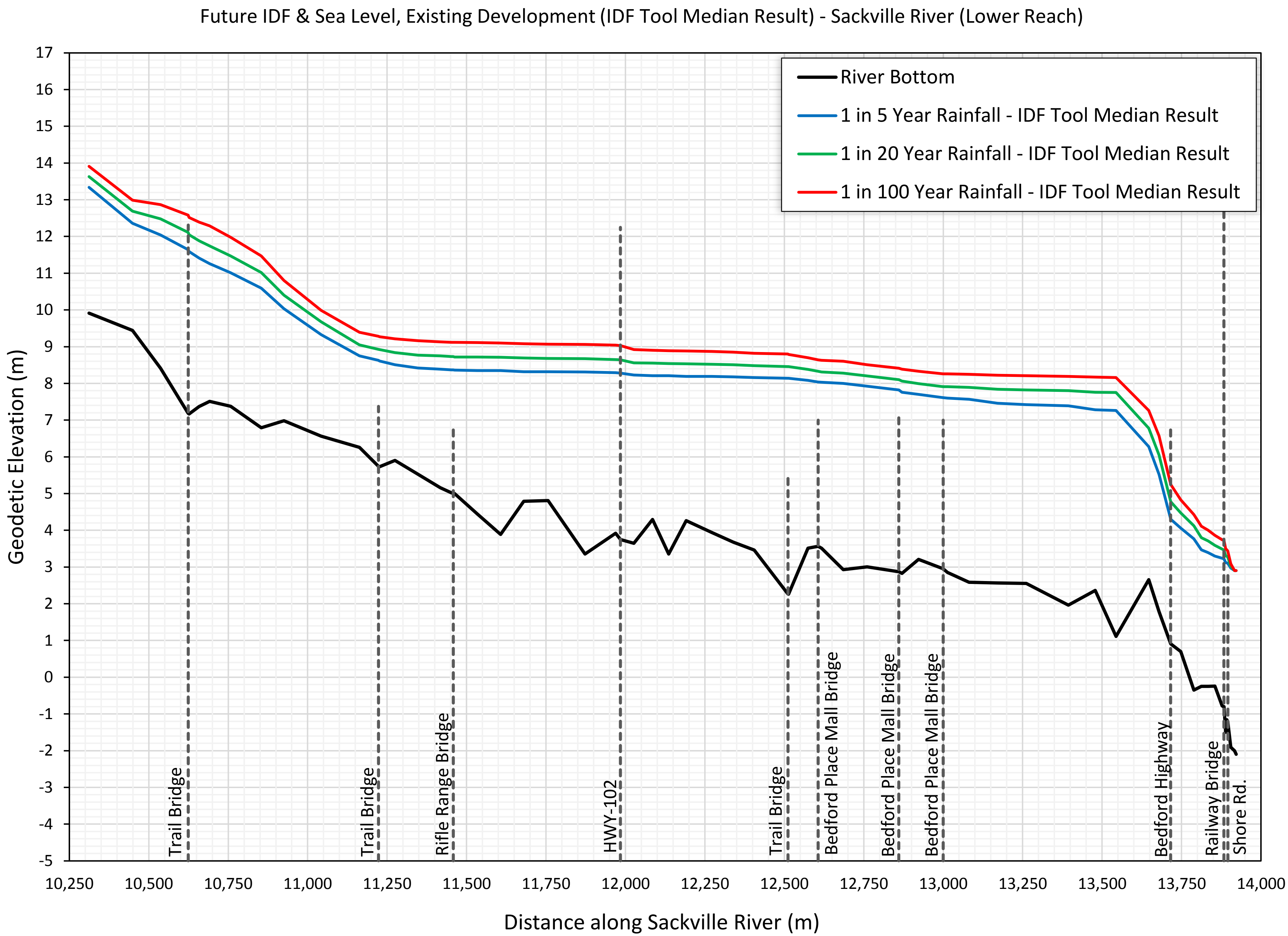
Profile #15



Future IDF & Sea Level, Existing Development (IDF Tool Median Result) - Sackville River (Lower Reach)
River Bottom
1 in 5 Year Rainfall - IDF Tool Median Result
1 in 20 Year Rainfall - IDF Tool Median Result
1 in 100 Year Rainfall - IDF Tool Median Result
Geodetic Elevation (m)
Distance along Sackville River (m)
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.
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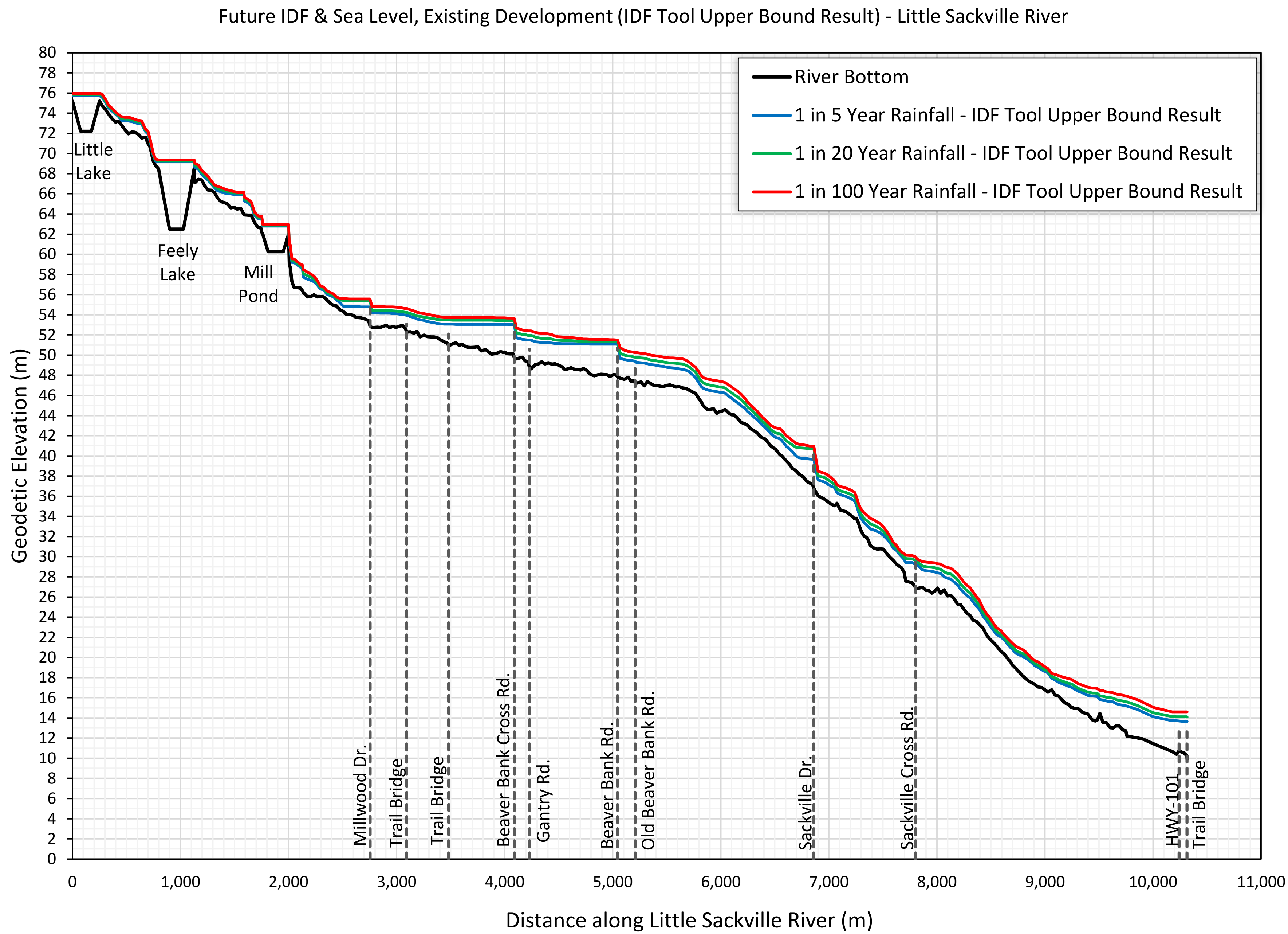
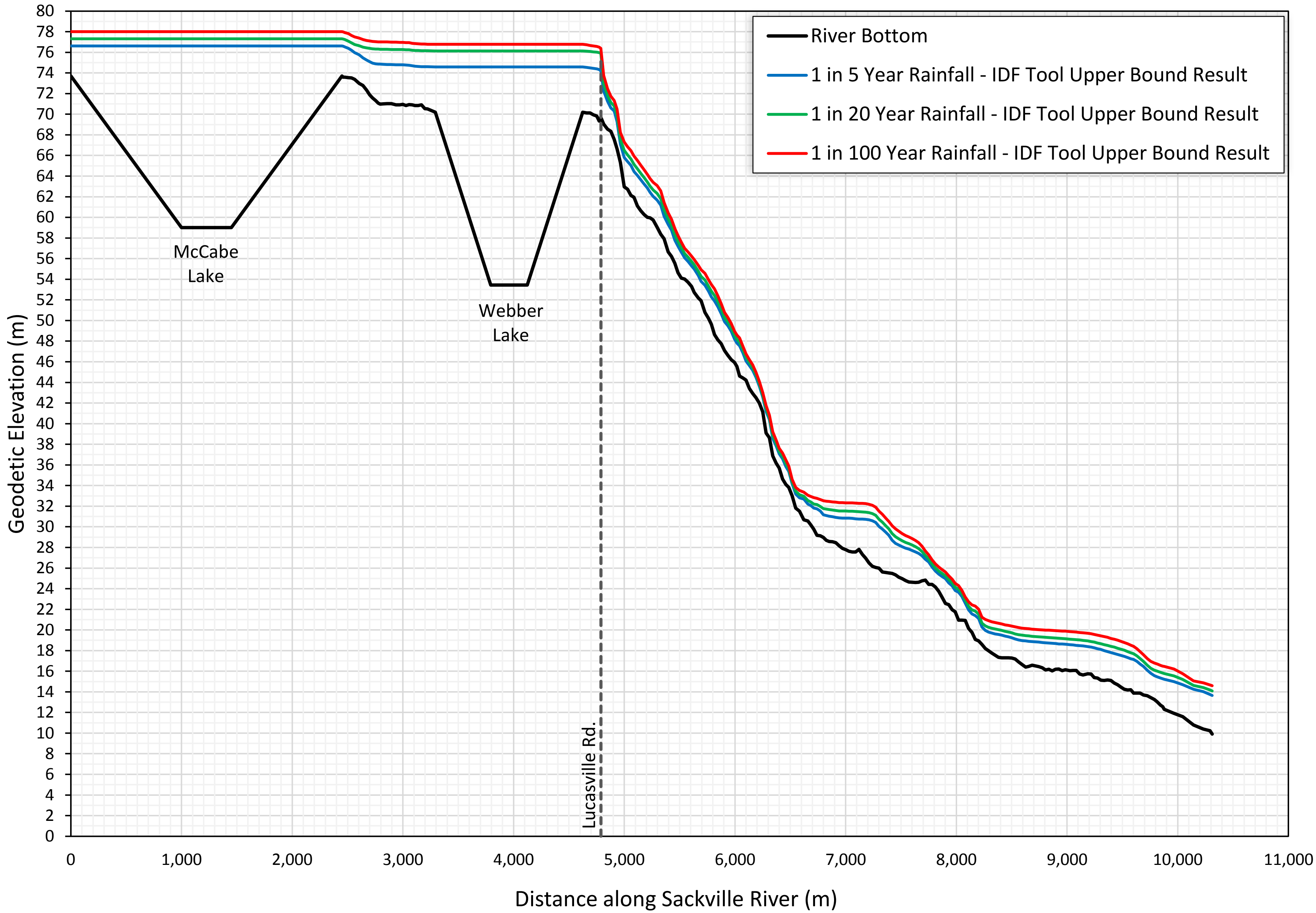
Profile #16



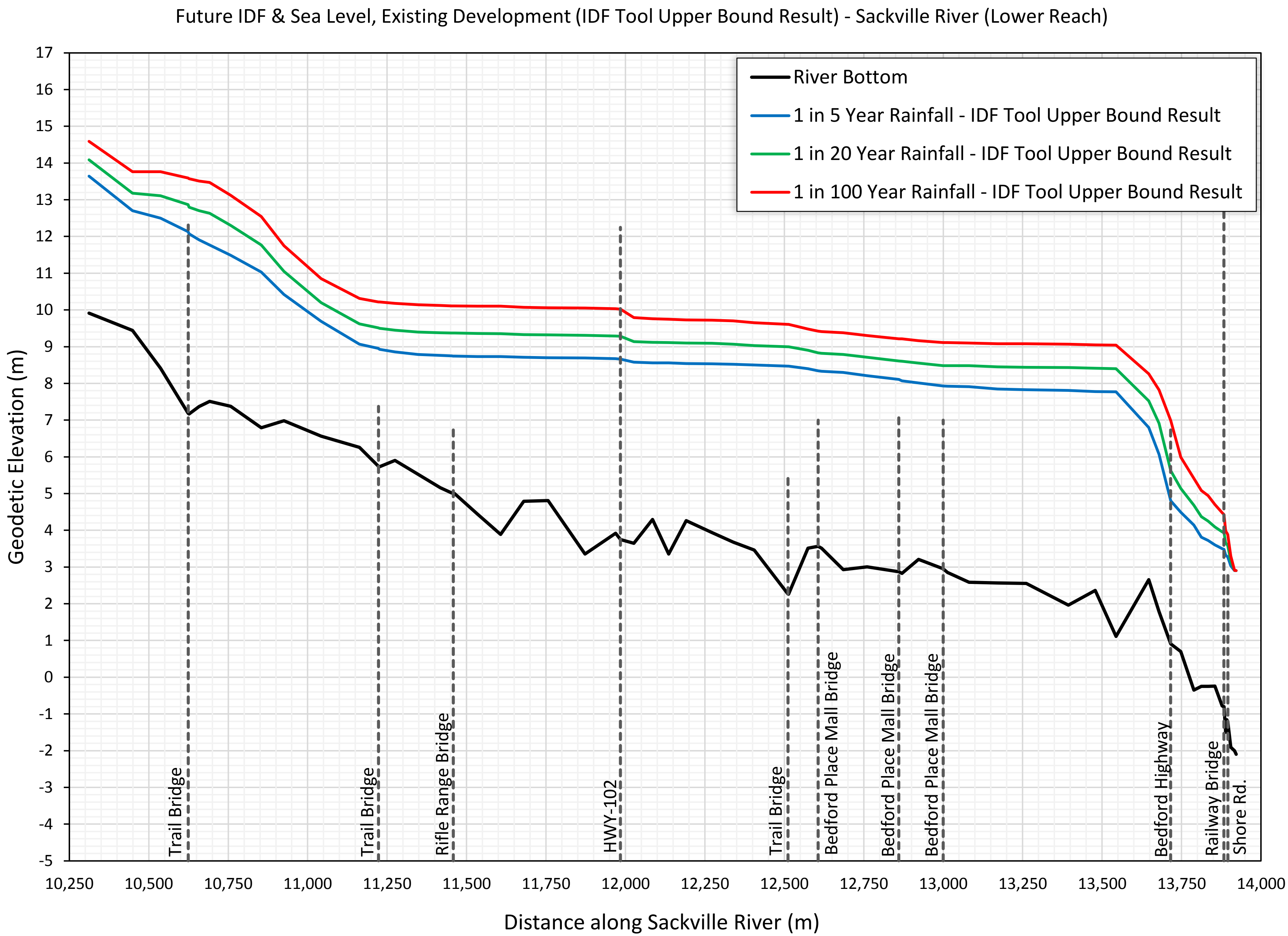
Future IDF & Sea Level, Existing Development (IDF Tool Upper Bound Result) - Little Sackville River
River Bottom
1 in 5 Year Rainfall - IDF Tool Upper Bound Result
1 in 20 Year Rainfall - IDF Tool Upper Bound Result
1 in 100 Year Rainfall - IDF Tool Upper Bound Result
Little Lake
Feely Lake
Mill Pond
Millwood Dr.
Trail Bridge
Trail Bridge
Beaver Bank Cross Rd.
Gantry Rd.
Beaver Bank Rd.
Old Beaver Bank Rd.
Sackville Dr.
Sackville Cross Rd.
HWY-101
Trail Bridge
Geodetic Elevation (m)
Distance along Little Sackville River (m)

Profile #17

Future IDF & Sea Level, Existing Development (IDF Tool Upper Bound Result) - Sackville River (Upper Reach)



Profile #18



Future IDF & Sea Level, Existing Development (IDF Tool Upper Bound Result) - Sackville River (Lower Reach)
River Bottom
1 in 5 Year Rainfall - IDF Tool Upper Bound Result
1 in 20 Year Rainfall - IDF Tool Upper Bound Result
1 in 100 Year Rainfall - IDF Tool Upper Bound Result
Geodetic Elevation (m)
Distance along Sackville River (m)
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.
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10,250
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10,750
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11,250
11,500
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12,250
12,500
12,750
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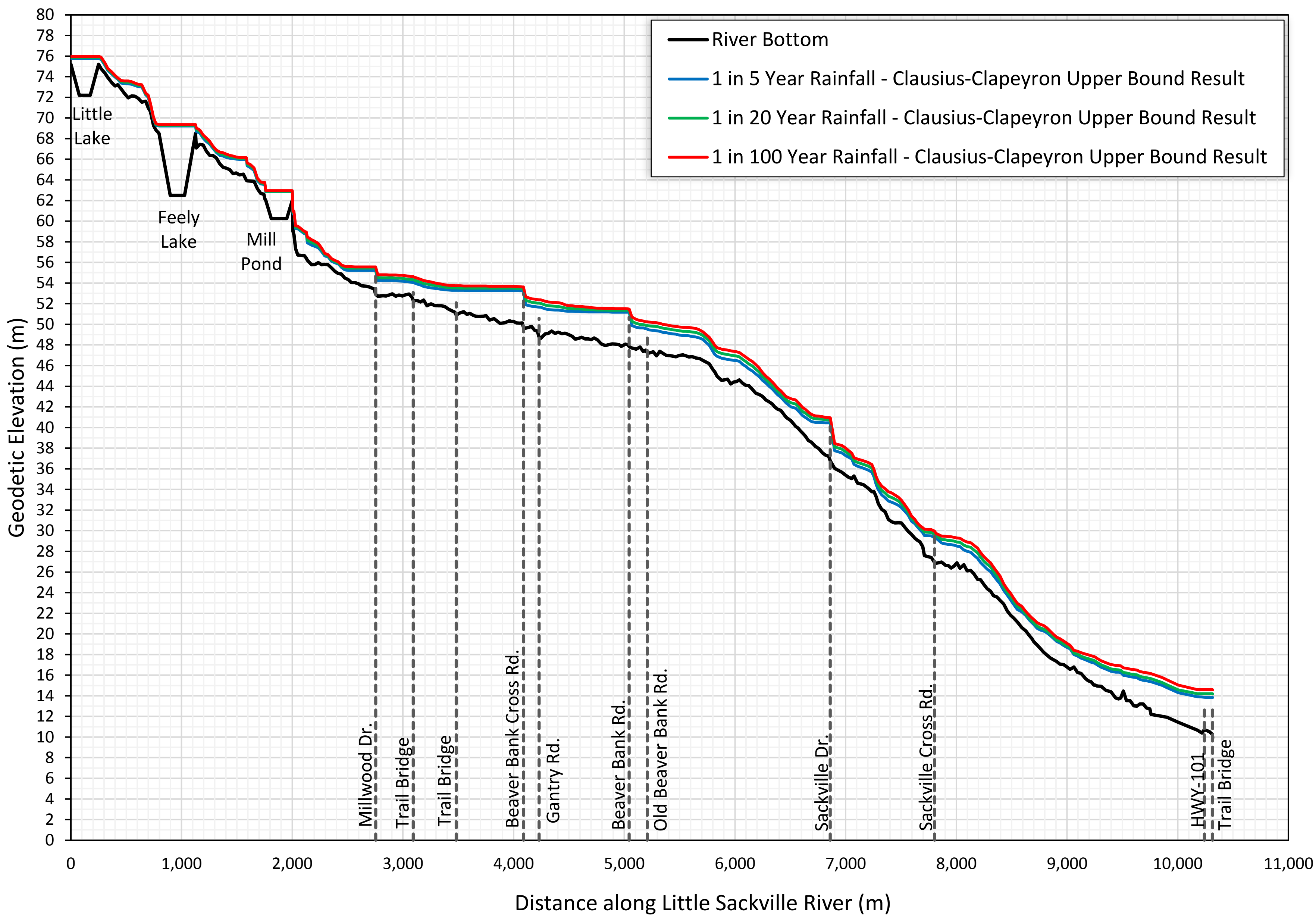
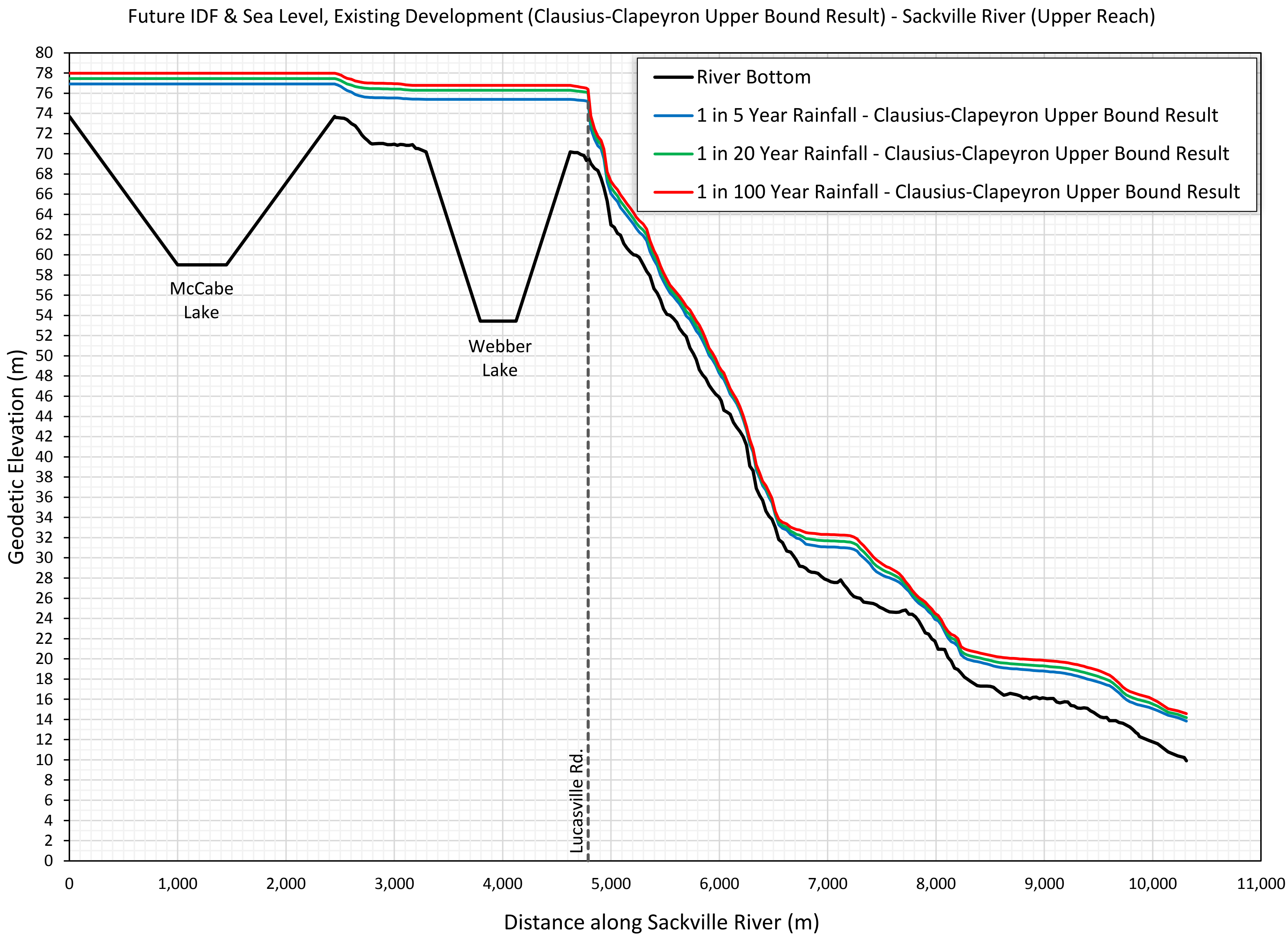
Profile #19

Future IDF & Sea Level, Existing Development (Clausius-Clapeyron Upper Bound Result) - Little Sackville River



Profile #20



Future IDF & Sea Level, Existing Development (Clausius-Clapeyron Upper Bound Result) - Sackville River (Upper Reach)
River Bottom
1 in 5 Year Rainfall - Clausius-Clapeyron Upper Bound Result
1 in 20 Year Rainfall - Clausius-Clapeyron Upper Bound Result
1 in 100 Year Rainfall - Clausius-Clapeyron Upper Bound Result
McCabe Lake
Webber Lake
Lucasville Rd.
Geodetic Elevation (m)
Distance along Sackville River (m)
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11,000
80
78
76
74
72
70
68
66
64
62
60
58
56
54
52
50
48
46
44
42
40
38
36
34
32
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24
22
20
18
16
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12
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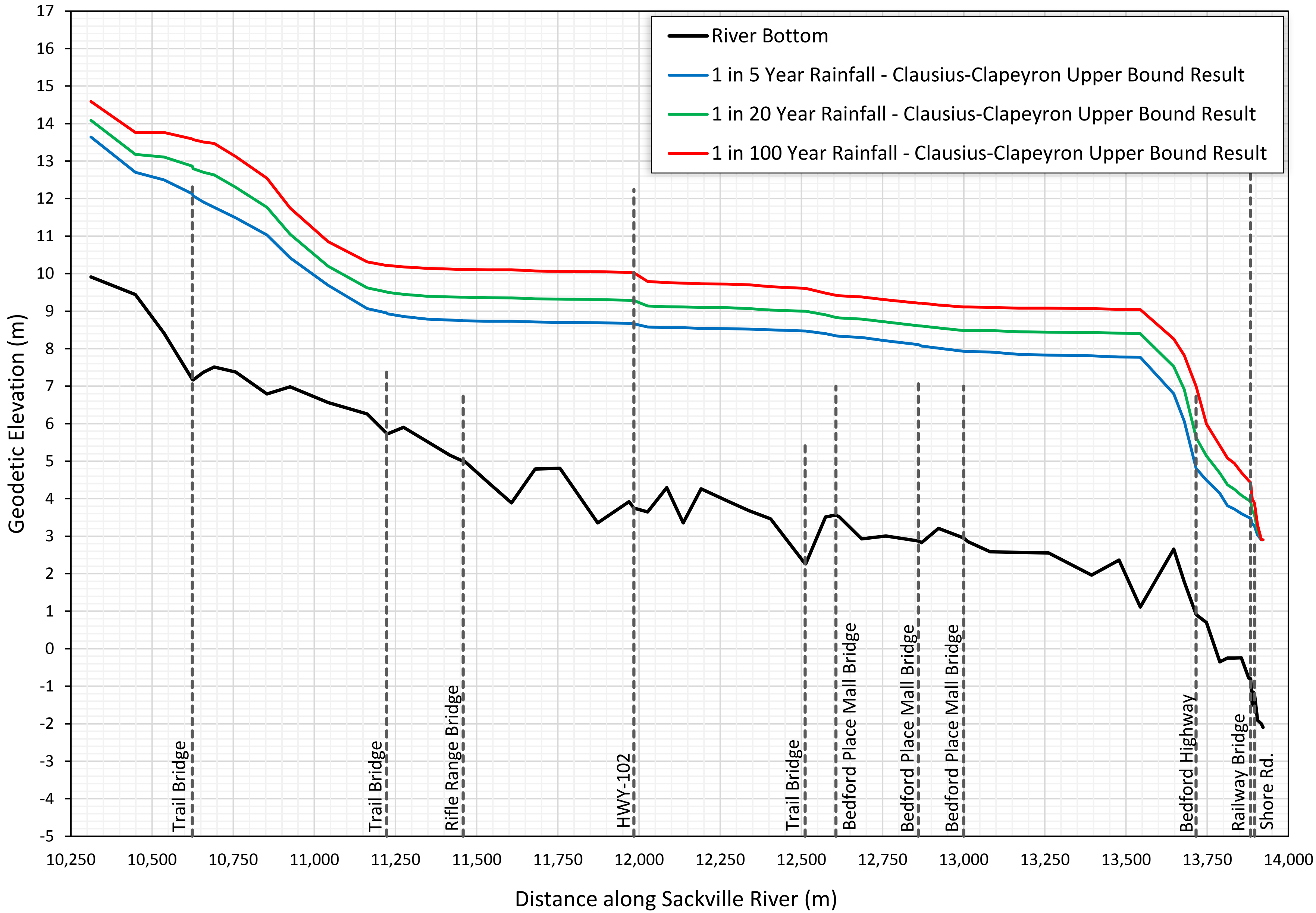
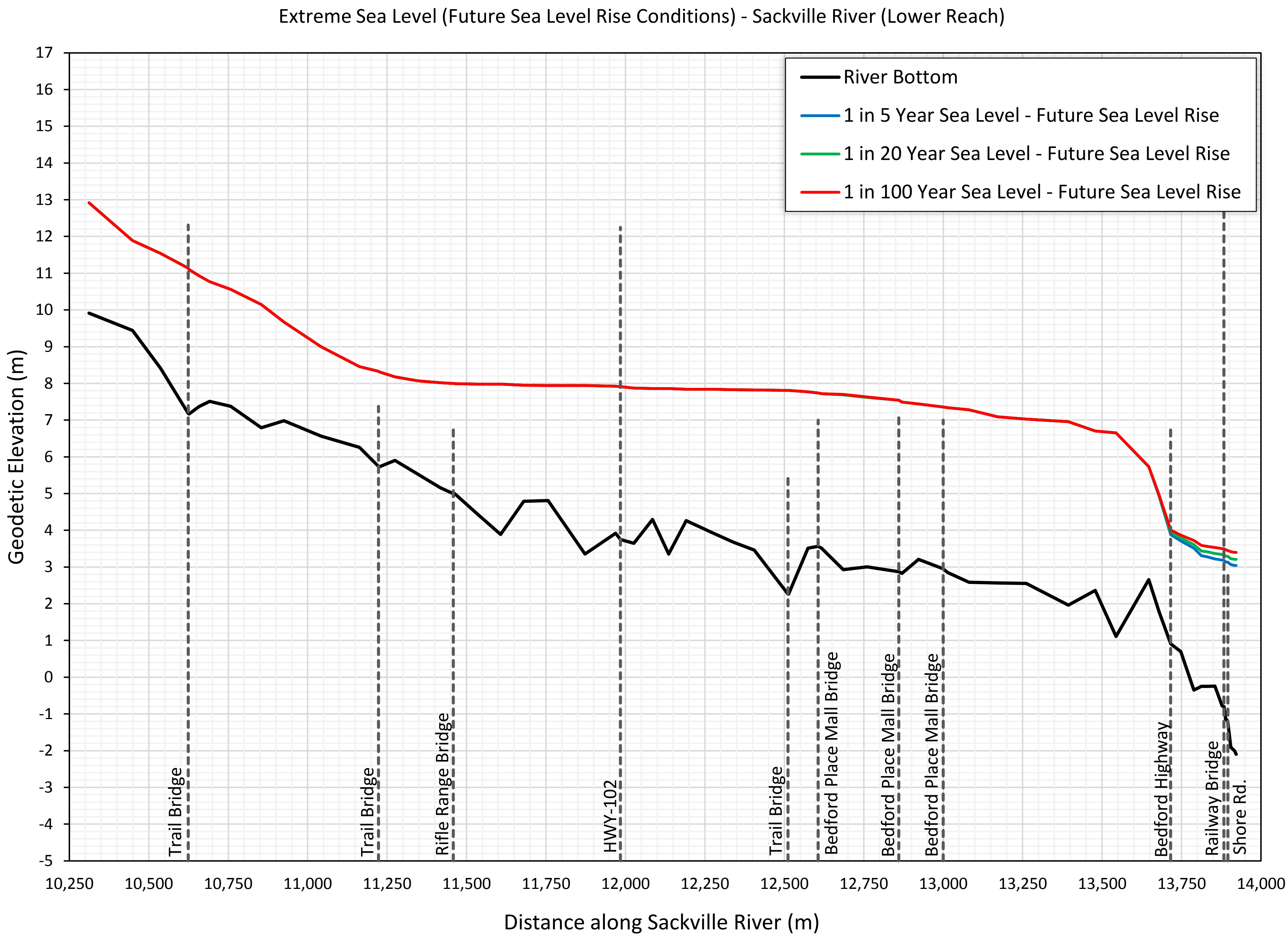
8
6
4
2
0

Future IDF & Sea Level, Existing Development (Clausius-Clapeyron Upper Bound Result) - Sackville River (Lower Reach)



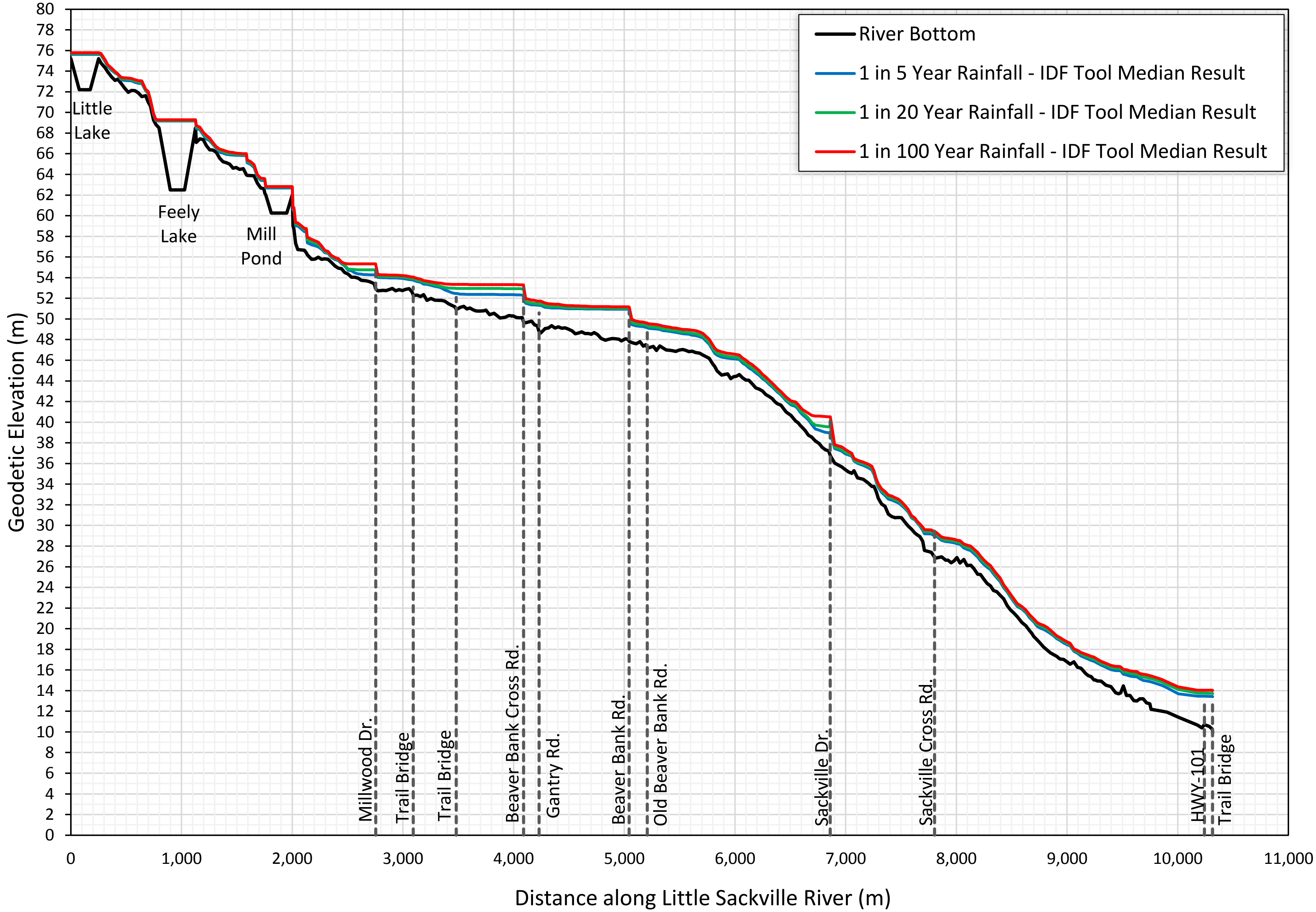
Profile #22



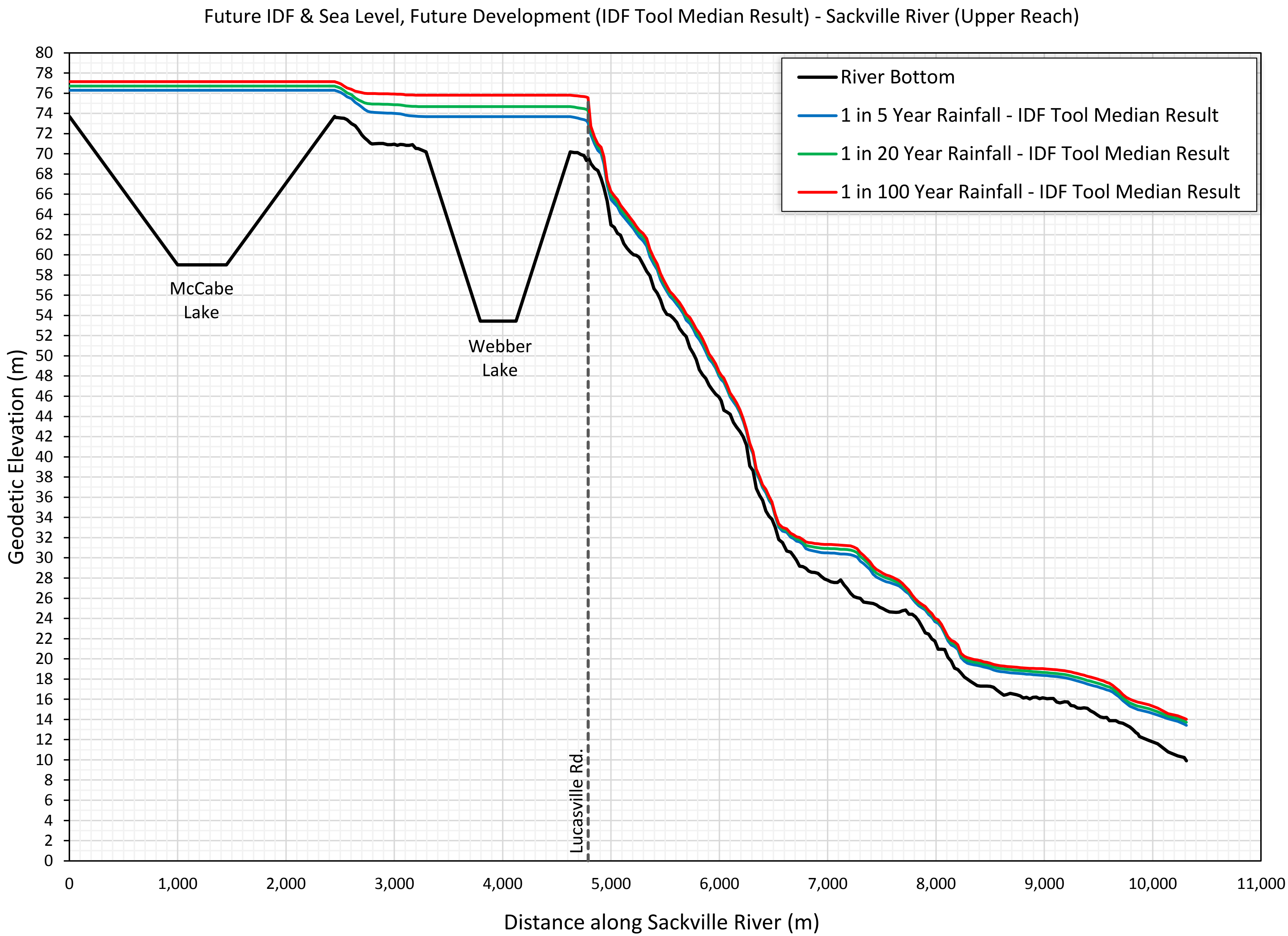
Extreme Sea Level (Future Sea Level Rise Conditions) - Sackville River (Lower Reach)
River Bottom
1 in 5 Year Sea Level - Future Sea Level Rise
1 in 20 Year Sea Level - Future Sea Level Rise
1 in 100 Year Sea Level - Future Sea Level Rise
Geodetic Elevation (m)
Distance along Sackville River (m)
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
10,250
10,500
10,750
11,000
11,250
11,500
11,750
12,000
12,250
12,500
12,750
13,000
13,250
13,500
13,750
14,000

Profile #23

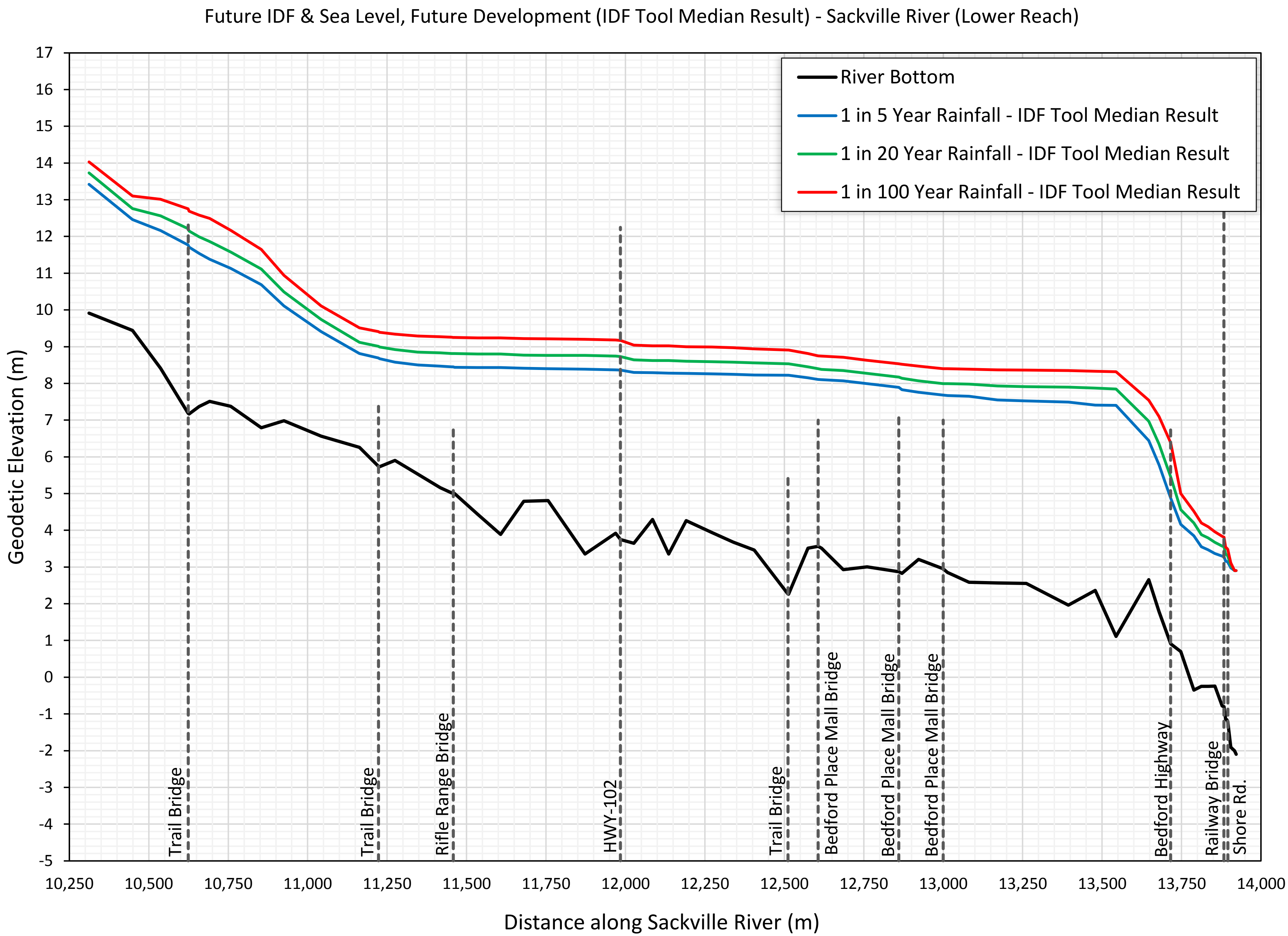
Future IDF & Sea Level, Future Development (IDF Tool Median Result) - Little Sackville River



Profile #24

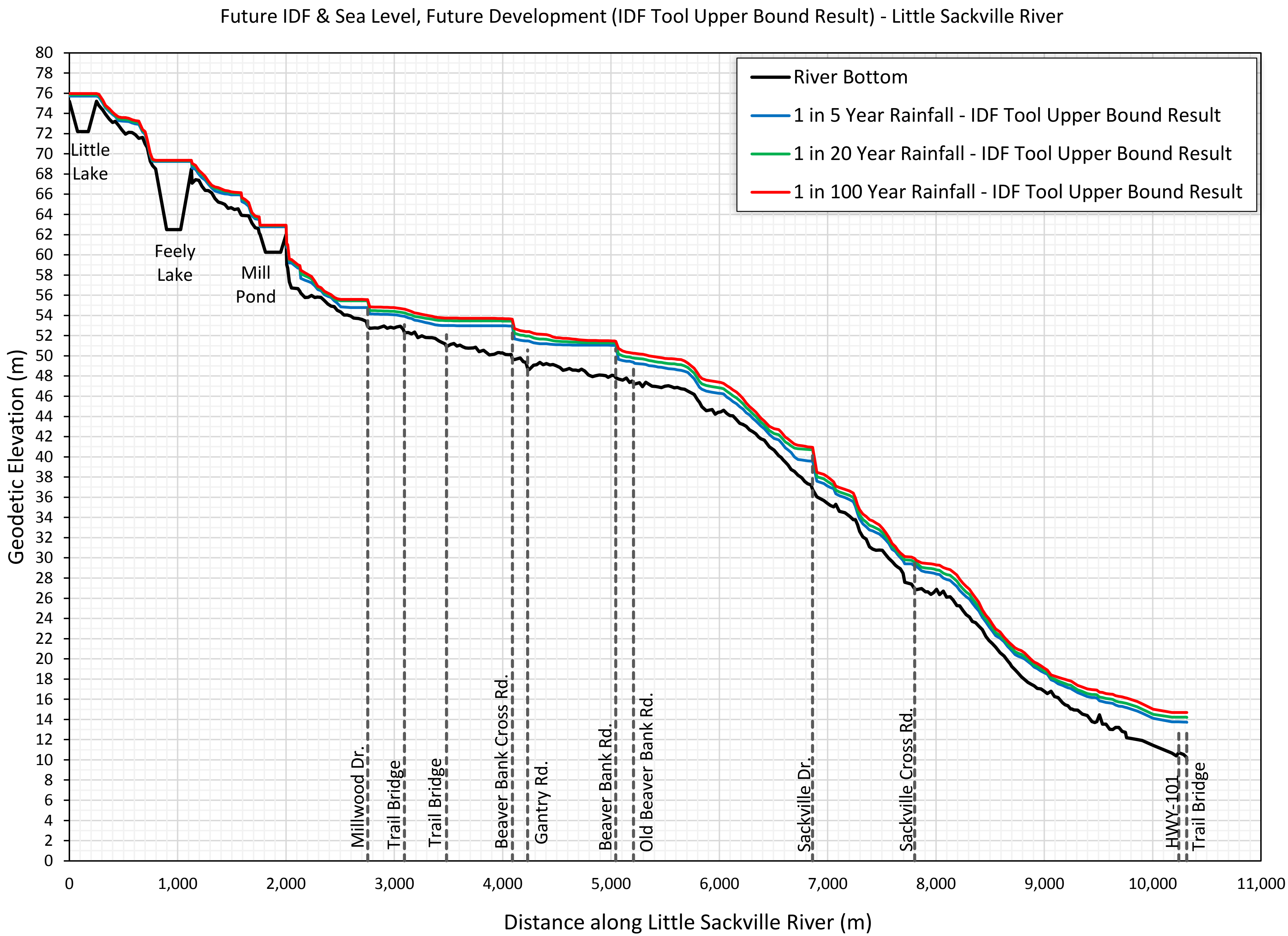

Future IDF & Sea Level, Future Development (IDF Tool Median Result) - Sackville River (Upper Reach)
River Bottom
1 in 5 Year Rainfall - IDF Tool Median Result
1 in 20 Year Rainfall - IDF Tool Median Result
1 in 100 Year Rainfall - IDF Tool Median Result
McCabe Lake
Webber Lake
Lucasville Rd.
Geodetic Elevation (m)
Distance along Sackville River (m)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
72
74
76
78
80
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11,000

Profile #25



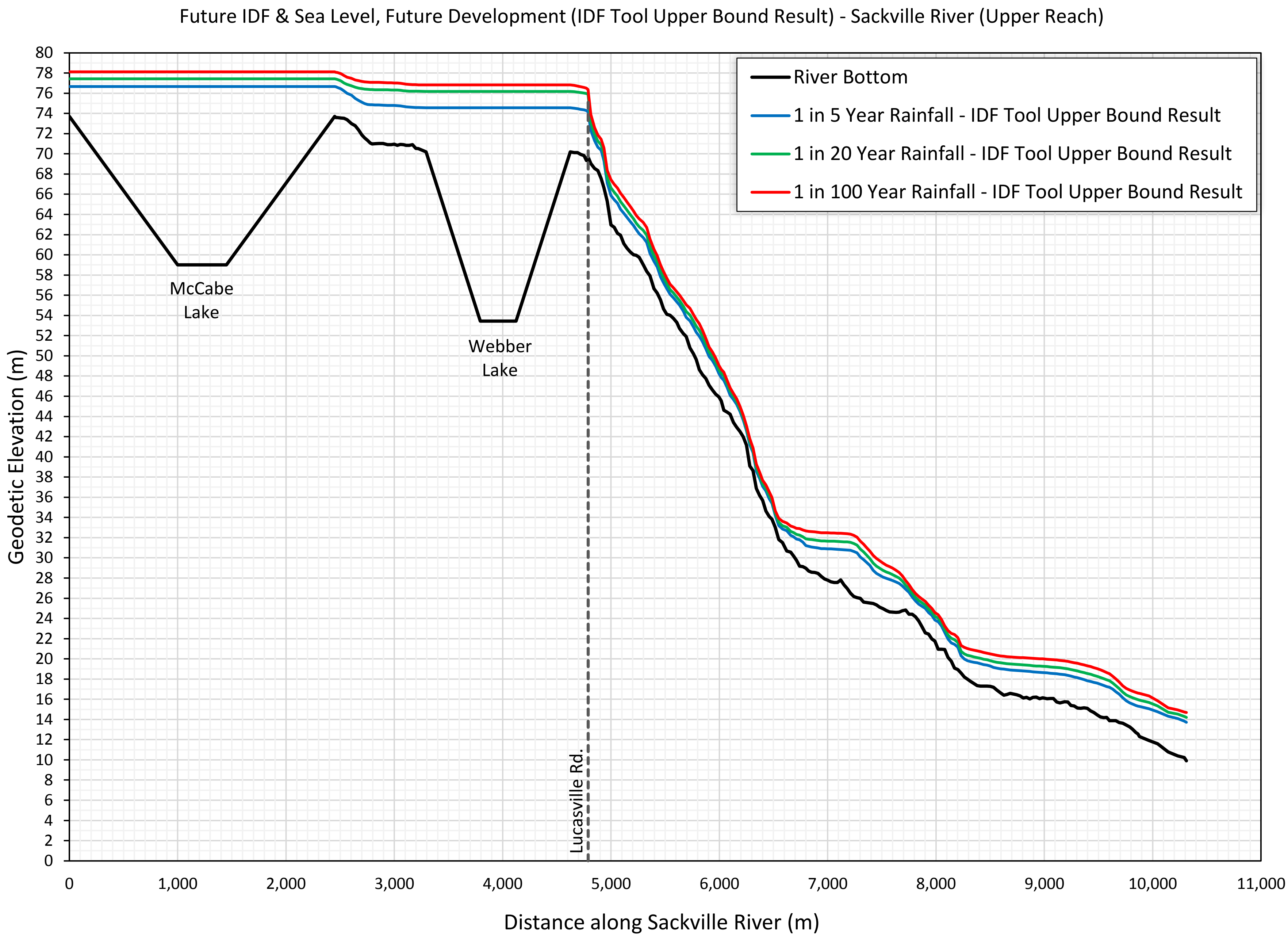
Future IDF & Sea Level, Future Development (IDF Tool Median Result) - Sackville River (Lower Reach)
River Bottom
1 in 5 Year Rainfall - IDF Tool Median Result
1 in 20 Year Rainfall - IDF Tool Median Result
1 in 100 Year Rainfall - IDF Tool Median Result
Geodetic Elevation (m)
Distance along Sackville River (m)
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
10,250
10,500
10,750
11,000
11,250
11,500
11,750
12,000
12,250
12,500
12,750
13,000
13,250
13,500
13,750
14,000
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.

Profile #26



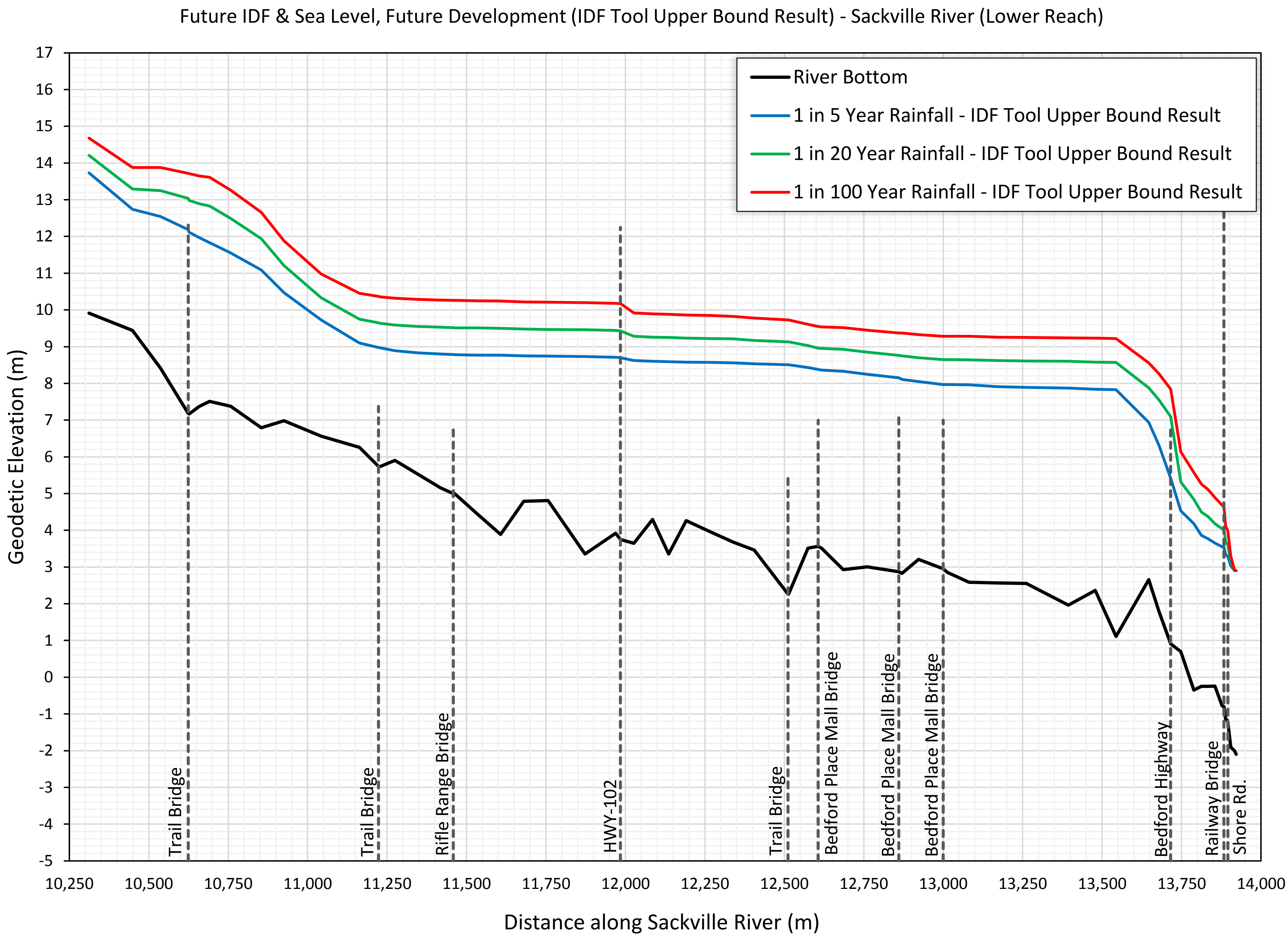
Future IDF & Sea Level, Future Development (IDF Tool Upper Bound Result) - Little Sackville River
River Bottom
1 in 5 Year Rainfall - IDF Tool Upper Bound Result
1 in 20 Year Rainfall - IDF Tool Upper Bound Result
1 in 100 Year Rainfall - IDF Tool Upper Bound Result
Little Lake
Feely Lake
Mill Pond
Millwood Dr.
Trail Bridge
Trail Bridge
Beaver Bank Cross Rd.
Gantry Rd.
Beaver Bank Rd.
Old Beaver Bank Rd.
Sackville Dr.
Sackville Cross Rd.
HWY-101
Trail Bridge
Geodetic Elevation (m)
Distance along Little Sackville River (m)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
72
74
76
78
80
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11,000

Profile #27



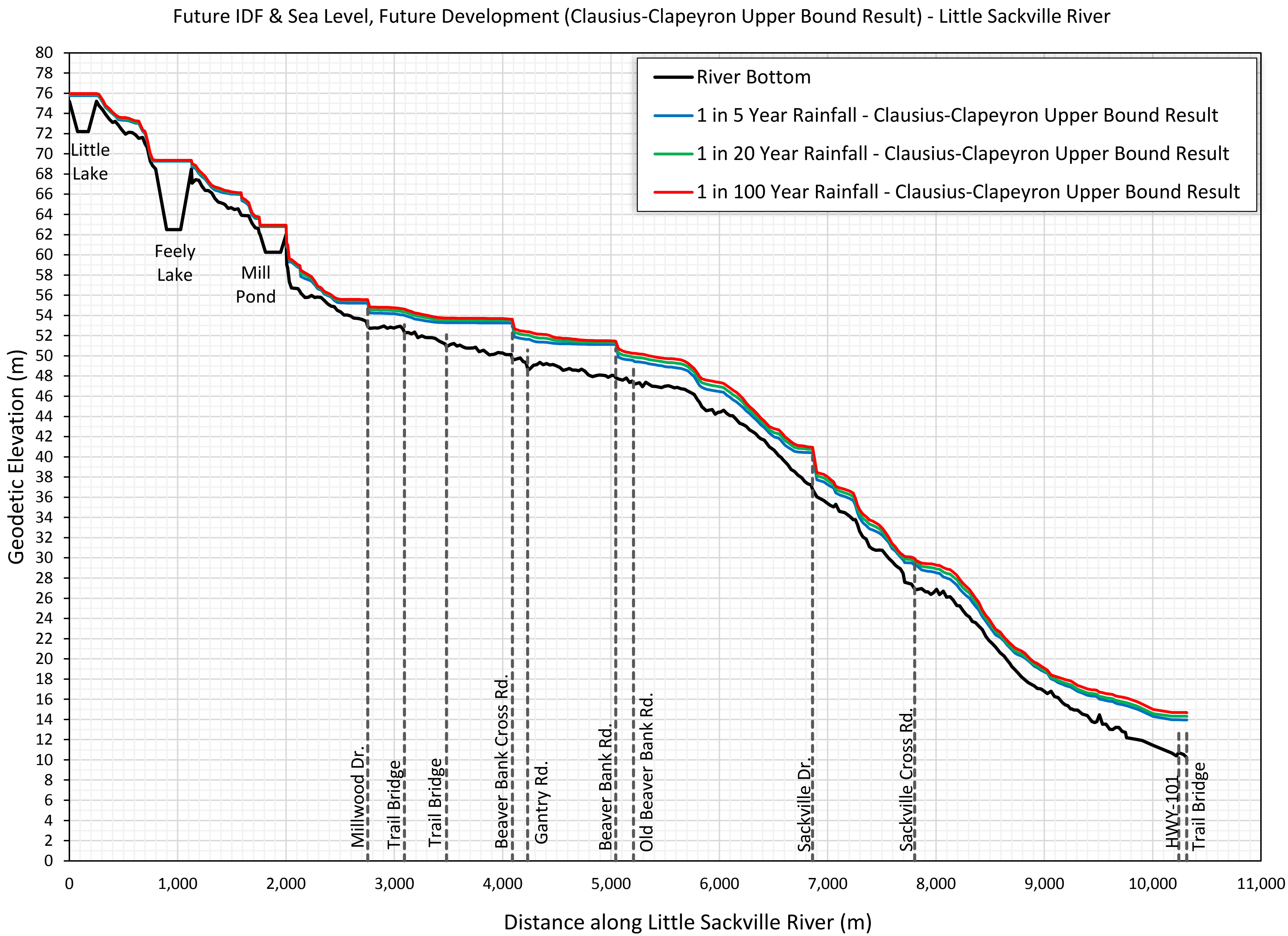
Future IDF & Sea Level, Future Development (IDF Tool Upper Bound Result) - Sackville River (Upper Reach)
Geodetic Elevation (m)
Distance along Sackville River (m)
River Bottom
1 in 5 Year Rainfall - IDF Tool Upper Bound Result
1 in 20 Year Rainfall - IDF Tool Upper Bound Result
1 in 100 Year Rainfall - IDF Tool Upper Bound Result
McCabe Lake
Webber Lake
Lucasville Rd.
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
72
74
76
78
80
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11,000

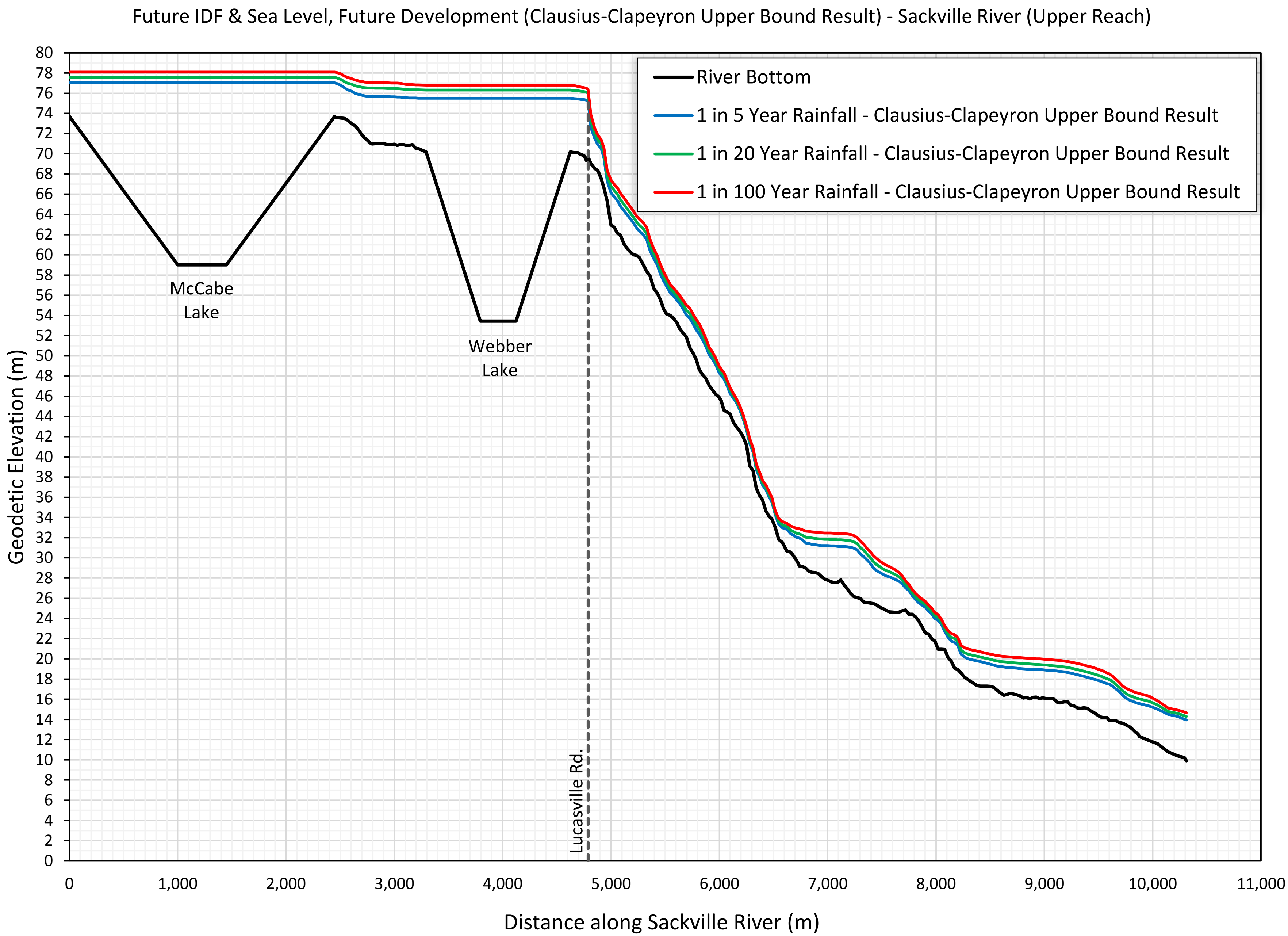
Profile #28



Future IDF & Sea Level, Future Development (IDF Tool Upper Bound Result) - Sackville River (Lower Reach)
River Bottom
1 in 5 Year Rainfall - IDF Tool Upper Bound Result
1 in 20 Year Rainfall - IDF Tool Upper Bound Result
1 in 100 Year Rainfall - IDF Tool Upper Bound Result
Geodetic Elevation (m)
Distance along Sackville River (m)
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
10,250
10,500
10,750
11,000
11,250
11,500
11,750
12,000
12,250
12,500
12,750
13,000
13,250
13,500
13,750
14,000
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.

Profile #29

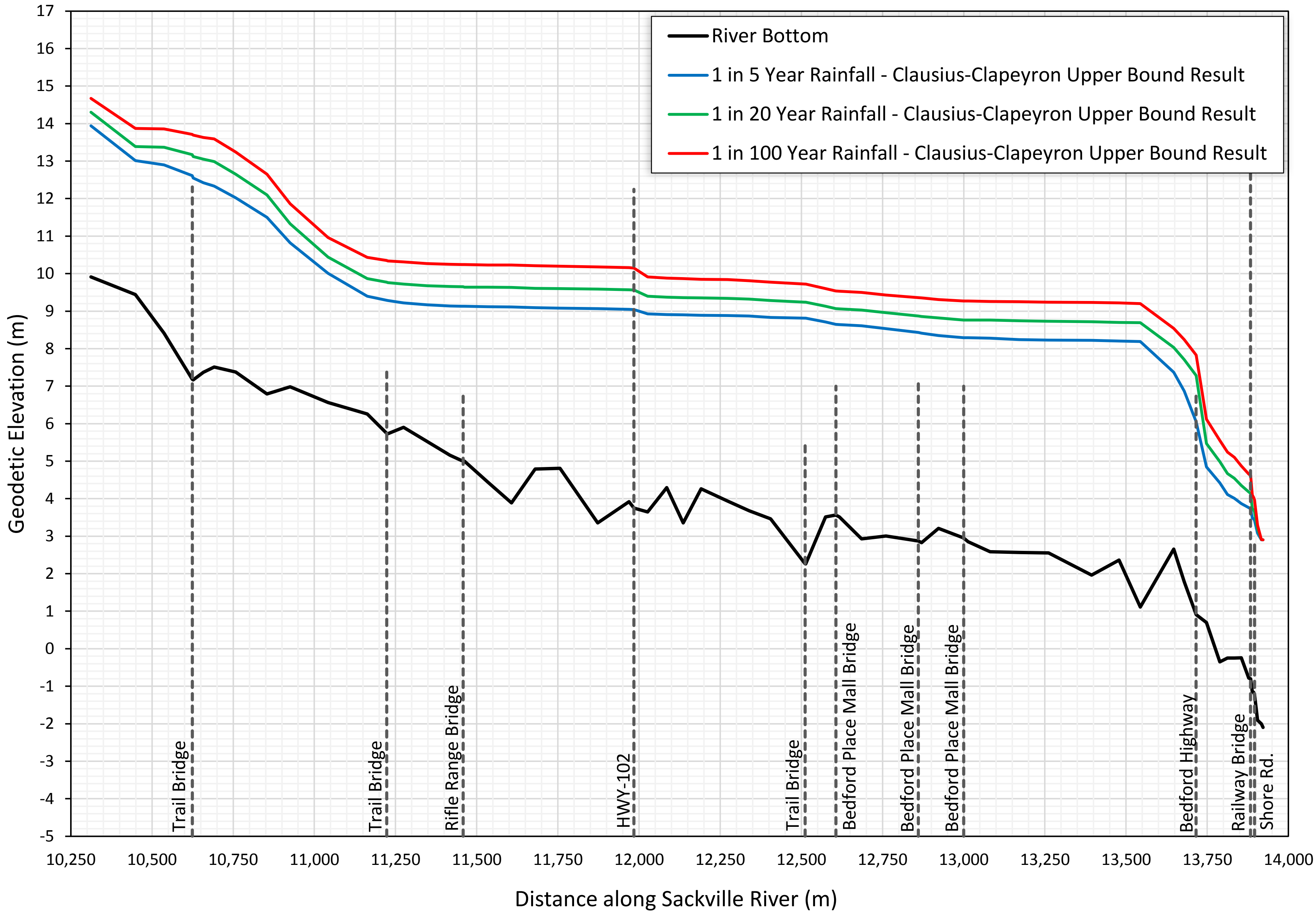

Future IDF & Sea Level, Future Development (Clausius-Clapeyron Upper Bound Result) - Little Sackville River
River Bottom
1 in 5 Year Rainfall - Clausius-Clapeyron Upper Bound Result
1 in 20 Year Rainfall - Clausius-Clapeyron Upper Bound Result
1 in 100 Year Rainfall - Clausius-Clapeyron Upper Bound Result
Geodetic Elevation (m)
Distance along Little Sackville River (m)
Little Lake
Feely Lake
Mill Pond
Millwood Dr.
Trail Bridge
Trail Bridge
Beaver Bank Cross Rd.
Gantry Rd.
Beaver Bank Rd.
Old Beaver Bank Rd.
Sackville Dr.
Sackville Cross Rd.
HWY-101
Trail Bridge



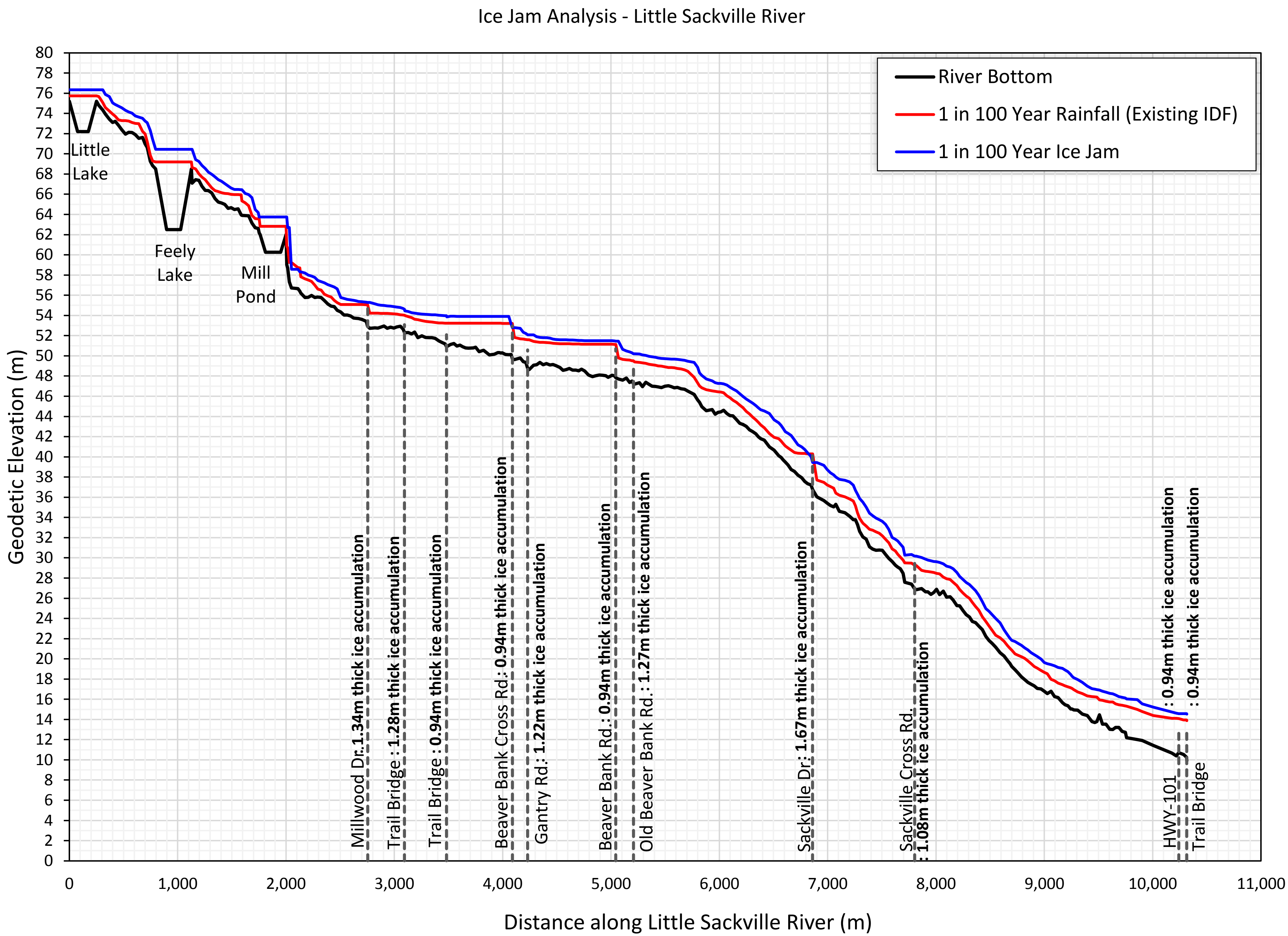
Future IDF & Sea Level, Future Development (Clausius-Clapeyron Upper Bound Result) - Sackville River (Upper Reach)
River Bottom
1 in 5 Year Rainfall - Clausius-Clapeyron Upper Bound Result
1 in 20 Year Rainfall - Clausius-Clapeyron Upper Bound Result
1 in 100 Year Rainfall - Clausius-Clapeyron Upper Bound Result
McCabe Lake
Webber Lake
Lucasville Rd.
Geodetic Elevation (m)
Distance along Sackville River (m)

Profile #31

Future IDF & Sea Level, Future Development (Clausius-Clapeyron Upper Bound Result) - Sackville River (Lower Reach)



Profile #32


Ice Jam Analysis - Little Sackville River
Geodetic Elevation (m)
Distance along Little Sackville River (m)
River Bottom
1 in 100 Year Rainfall (Existing IDF)
1 in 100 Year Ice Jam
Little Lake
Feely Lake
Mill Pond
Millwood Dr.: 1.34m thick ice accumulation
Trail Bridge : 1.28m thick ice accumulation
Trail Bridge : 0.94m thick ice accumulation
Beaver Bank Cross Rd.: 0.94m thick ice accumulation
Gantry Rd.: 1.22m thick ice accumulation
Beaver Bank Rd.: 0.94m thick ice accumulation
Old Beaver Bank Rd.: 1.27m thick ice accumulation
Sackville Dr.: 1.67m thick ice accumulation
Sackville Cross Rd. : 1.08m thick ice accumulation
HWY-101 : 0.94m thick ice accumulation
Trail Bridge : 0.94m thick ice accumulation
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
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72
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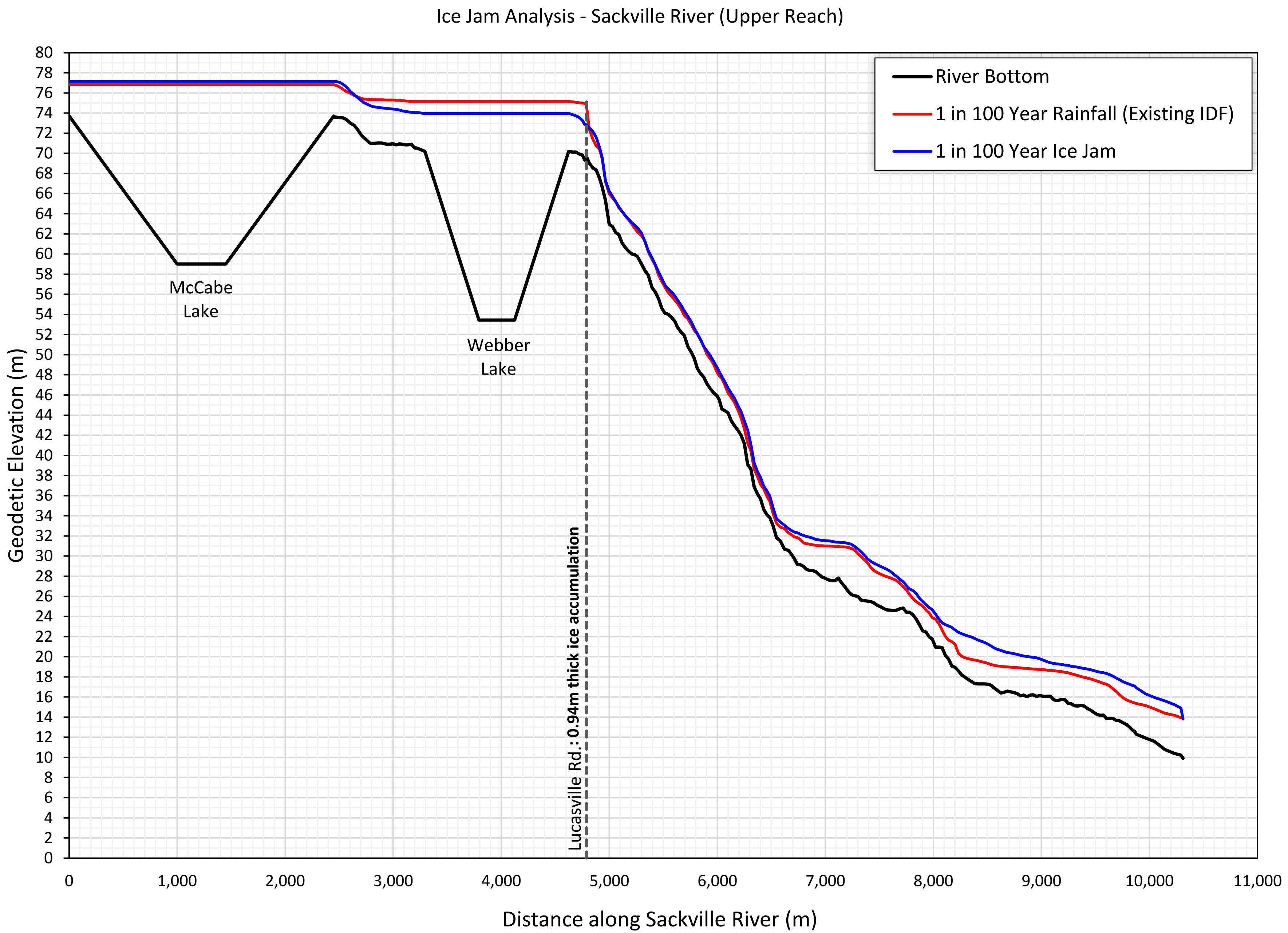
Profile #33



Ice Jam Analysis - Sackville River (Upper Reach)
River Bottom
1 in 100 Year Rainfall (Existing IDF)
1 in 100 Year Ice Jam
Geodetic Elevation (m)
Distance along Sackville River (m)
McCabe Lake
Webber Lake
Lucasville Rd.: 0.94m thick ice accumulation
0
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
9,000
10,000
11,000
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
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56
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60
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66
68
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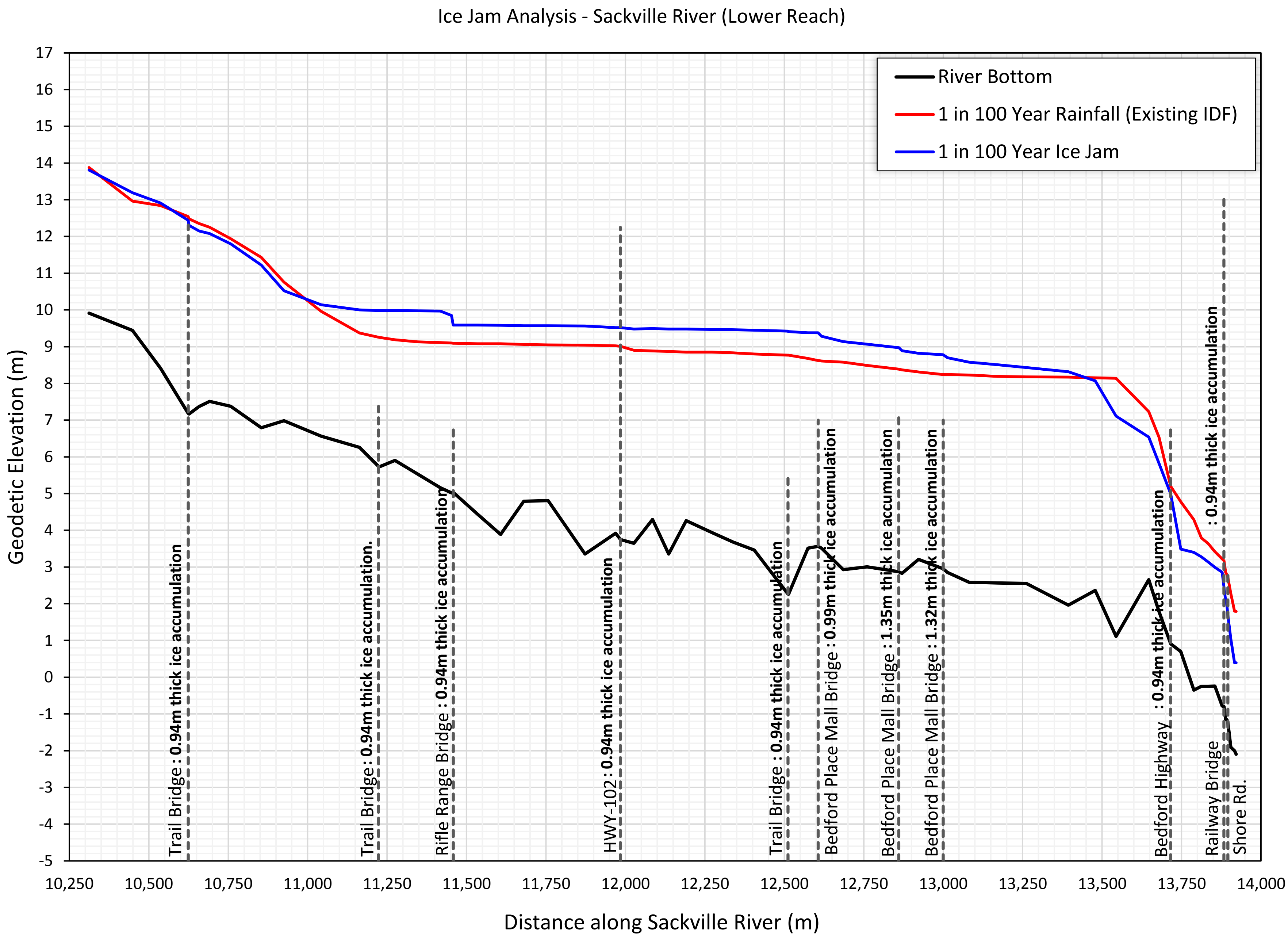
72
74
76
78
80

Profile #34



Ice Jam Analysis - Sackville River (Lower Reach)
River Bottom
1 in 100 Year Rainfall (Existing IDF)
1 in 100 Year Ice Jam
Geodetic Elevation (m)
Distance along Sackville River (m)
Trail Bridge : 0.94m thick ice accumulation
Trail Bridge : 0.94m thick ice accumulation.
Rifle Range Bridge : 0.94m thick ice accumulation
HWY-102 : 0.94m thick ice accumulation
Trail Bridge : 0.94m thick ice accumulation
Bedford Place Mall Bridge : 0.99m thick ice accumulation
Bedford Place Mall Bridge : 1.35m thick ice accumulation
Bedford Place Mall Bridge : 1.32m thick ice accumulation
Bedford Highway : 0.94m thick ice accumulation
Railway Bridge : 0.94m thick ice accumulation
Shore Rd.
17
16
15
14
13
12
11
10
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8
7
6
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4
3
2
1
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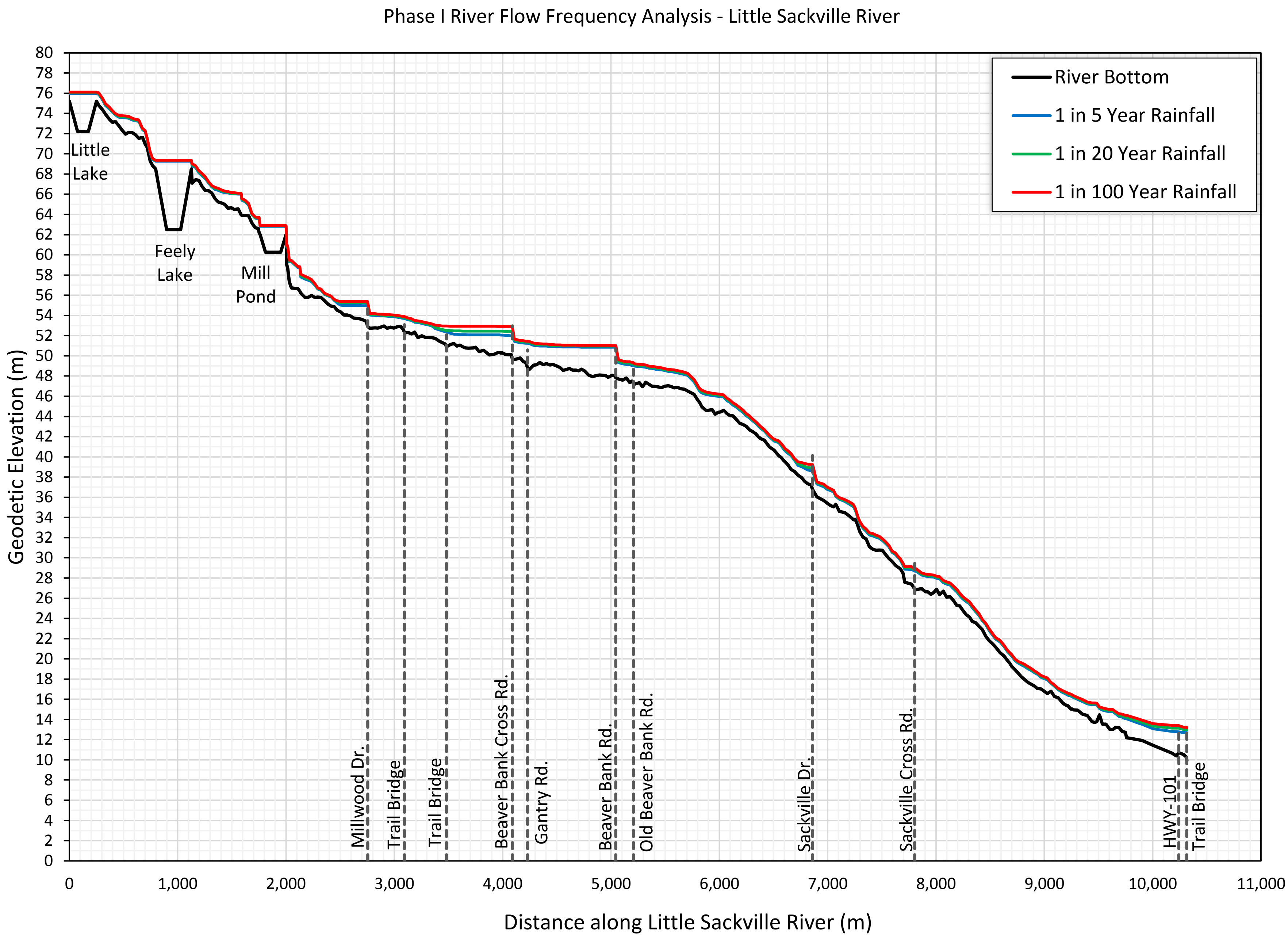
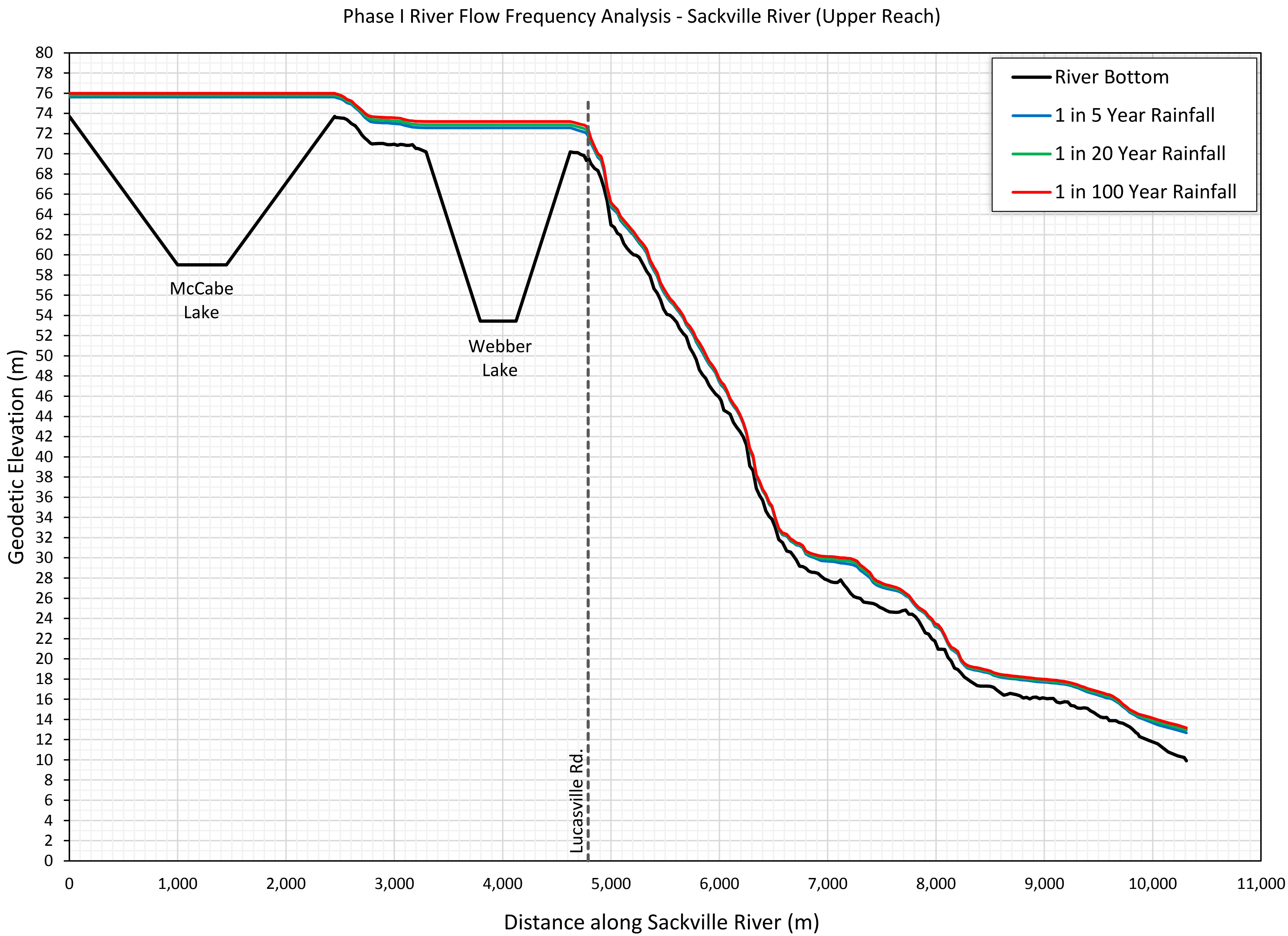
-1
-2
-3
-4
-5
10,250
10,500
10,750
11,000
11,250
11,500
11,750
12,000
12,250
12,500
12,750
13,000
13,250
13,500
13,750
14,000

Profile #35



Phase I River Flow Frequency Analysis - Little Sackville River
River Bottom
1 in 5 Year Rainfall
1 in 20 Year Rainfall
1 in 100 Year Rainfall
Geodetic Elevation (m)
Distance along Little Sackville River (m)
Little Lake
Feely Lake
Mill Pond
Millwood Dr.
Trail Bridge
Trail Bridge
Beaver Bank Cross Rd.
Gantry Rd.
Beaver Bank Rd.
Old Beaver Bank Rd.
Sackville Dr.
Sackville Cross Rd.
HWY-101
Trail Bridge

Profile #36


Phase I River Flow Frequency Analysis - Sackville River (Upper Reach)
River Bottom
1 in 5 Year Rainfall
1 in 20 Year Rainfall
1 in 100 Year Rainfall
McCabe Lake
Webber Lake
Lucasville Rd.
Geodetic Elevation (m)
Distance along Sackville River (m)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
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48
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64
66
68
70
72
74
76
78
80
1,000
2,000
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7,000
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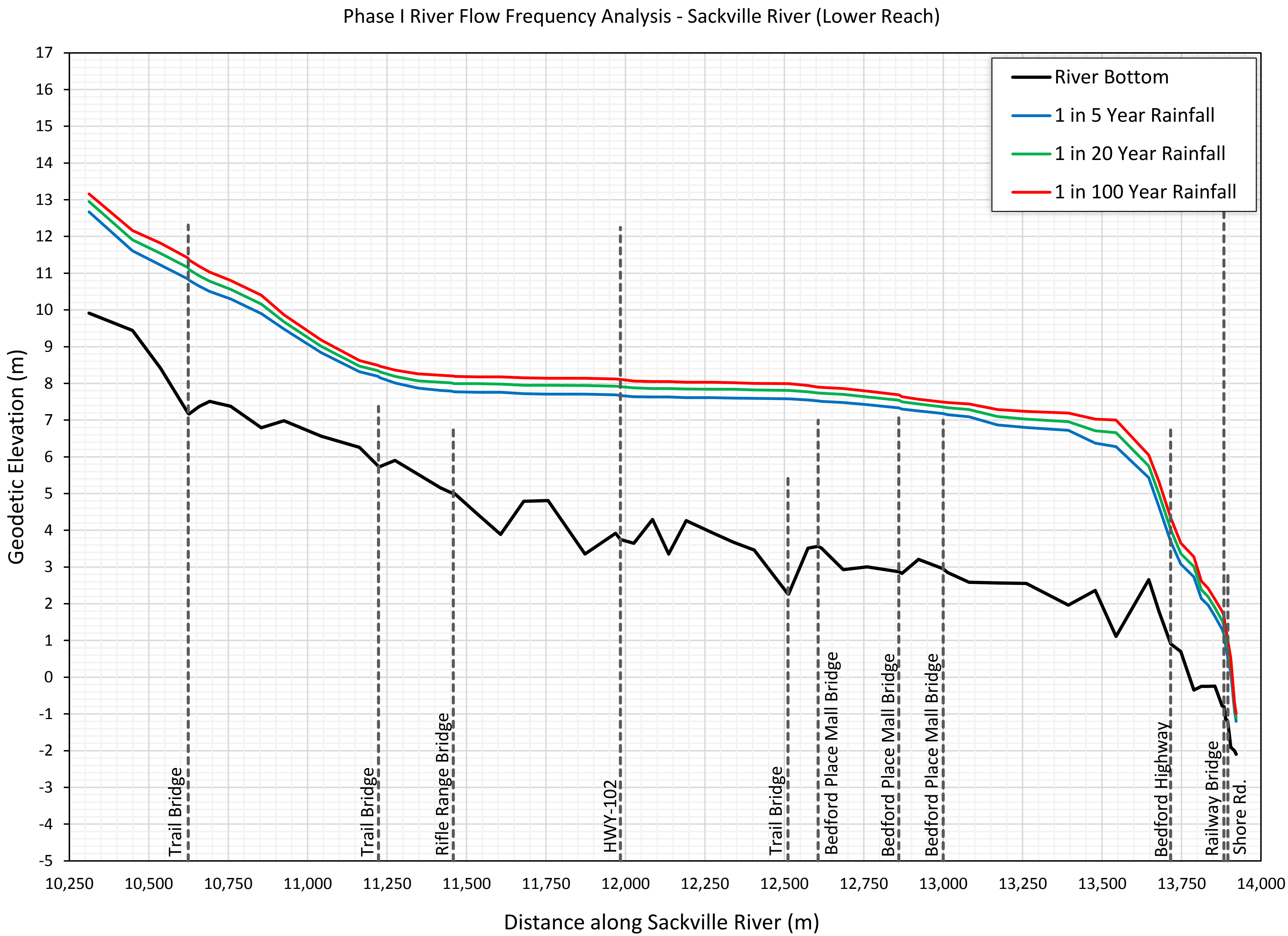
9,000
10,000
11,000

Profile #37


Phase I River Flow Frequency Analysis - Sackville River (Lower Reach)
River Bottom
1 in 5 Year Rainfall
1 in 20 Year Rainfall
1 in 100 Year Rainfall
Geodetic Elevation (m)
Distance along Sackville River (m)
Trail Bridge
Trail Bridge
Rifle Range Bridge
HWY-102
Trail Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Place Mall Bridge
Bedford Highway
Railway Bridge
Shore Rd.
17
16
15
14
13
12
11
10
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8
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6
5
4
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2
1
0
-1
-2
-3
-4
-5
10,250
10,500
10,750
11,000
11,250
11,500
11,750
12,000
12,250
12,500
12,750
13,000
13,250
13,500
13,750
14,000

APPENDIX E

Flood Line Delineation Maps