VALDOR ENGINEERING INC.
Municipal • Land Development • Water Resources
Site Development • Project Management • Contract Administration
Consulting Engineers - est. 1992

741 Rowntree Dairy Road, Suite 2
Woodbridge, Ontario L4L 5T9
TEL (905) 264-0054
FAX (905) 264-0069
info@valdor-engineering.com
www.valdor-engineering.com

# Floodplain Mapping in Jane and Wilson Special Policy Area, Black Creek

**Toronto and Region Conservation Authority**
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Prepared By:

**Valdor Engineering Inc.**

Prepared For:

**The Toronto and Region Conservation Authority**

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# Executive Summary

## FLOODPLAIN MAPPING IN JANE AND WILSON SPECIAL POLICY AREA, BLACK CREEK

### *Introduction and Background*

The current approved floodplain map sheets for the Jane-Wilson study area were prepared in 2005 (J.F. Sabourin & Associates) for the TRCA using the one dimensional (1D) HEC-RAS model and the Regional flow was calculated using SWMHYMO from the *Humber River Watershed Hydrology Update* (Aquafor Beech, November 2002). Due to the flat topography of the existing development within the floodplain and numerous hydraulic issues including several spill areas, multidirectional flow along roads and around buildings and flow split conditions, Valdor Engineering Inc. was retained by the TRCA to develop an integrated 1D/2D hydraulic model based on the latest available LiDAR surface data and flow data and to complete a flood study for the existing flood vulnerable area located in the vicinity of the Jane-Wilson Special Policy Area (SPA) including all areas within the extents of the Regional floodplain. The TRCA recently retained a consultant to update the Humber River flows using Visual OTTHYMO (VO3) and the results of this are provided in the *Humber River Hydrology Update* (Civica, June 2015). These updated flows were used in the integrated 1D/2D hydraulic model prepared as part of this study.

### *Existing Conditions Hydraulic Model Development and Assessment*

Using updated digital elevation mapping (DEM) derived from recently obtained LiDAR data supplemented with the completed topographic survey for channel sections and hydraulic structures, updated land use data, and updated flow data, an integrated 1D/2D hydraulic model was prepared using Mike Flood. The results of the Mike Flood model were found to be similar to the current approved extents of flooding based on HEC-RAS in many areas, however, the flooding in flat areas was generally less extensive and spill areas were much better defined. The Mike Flood model provided better representation of the depths and velocities of flooding in the streets, on properties, and around buildings in the spill areas.

### *Model Validation and Comparison with Existing HEC-RAS Model*

The Mike Flood model was validated using water level data from the stream gauge operated by the TRCA on Black Creek located at Downsview Avenue. The Mike Flood model was found to agree well with the gauged water level data. A comparison of the Mike Flood model with the existing HEC-RAS model was completed. The results of the Mike Flood model compared well in some areas with the existing HEC-RAS model but not in all areas. It was noted that HEC-RAS and Mike Flood are very different hydraulic models. Upon further inspection, it was found that the use of ineffective flow areas in the existing HEC-RAS model was not appropriate in many areas. A revised HEC-RAS model was prepared using more appropriate ineffective flow areas and better defined Manning's roughness values. The Mike Flood model was found to agree reasonably well with the revised existing HEC-RAS model. It is noted that the comparison with HEC-RAS was completed only with the Mike 11 (1D) channel profile for a consistent comparison and that the Mike 21 (2D) results could be different.

### *Floodplain Mapping*

The results of the Mike Flood model using the steady Regional flow hydrographs provided by the TRCA were used to delineate the Regional floodplain. The floodplain calculated using Mike Flood was used to update the engineered floodplain map sheets through the study area (Humber River Map Sheets 8 and 9).

*Characterization and Risk Assessment of the Floodplain*

A characterization and risk assessment of the floodplain was completed to identify low, moderate and high risk areas within the floodplain and to identify high risk flood zones. A total of six (6) high risk flood zones were identified. The extent of high risk flooding within each flood zone was calculated for the 350-yr return period design storm and the Regional storm (Hurricane Hazel).

*Identification of Hydraulic Constraints*

An analysis of flood flows and the flooding process was undertaken to determine the key hydraulic constraints contributing to the flooding in each of the identified flood zones. The identified hydraulic constraints included undersized hydraulic structures (*e.g.* culverts, bridges with poor conveyance capacity), undersized channels (*e.g.* watercourse with poor conveyance capacity) and constraints due to topography (*e.g.* spill points, low lying or flat, poorly drained topography). The frequency at which flooding occurs for the identified hydraulic constraints was determined using the results of the Mike Flood model. Possible mitigation options were recommended for future consideration and detailed analysis for the key hydraulic constraints within each of the 6 flood zones. Recommended mitigation options included bridge or culvert conveyance capacity improvements, channel conveyance capacity improvements, spill containment and overland flow improvements.

*Recommendations*

It is recommended that the revised Humber River Floodplain Map Sheets 8 and 9 based on Mike Flood should be used to replace the existing floodplain map sheets completed previously for the study area. It is also recommended that a detailed 1D/2D integrated hydraulic study should be undertaken to confirm and assess the possible options identified in this report (see **Table 8.1**) to improve the current flooding conditions within the study area. The study should include systematic investigations to confirm and assess the governing factors for the possible mitigation options to enable the determination of the best single mitigation option or combination of mitigation options within each identified flood zone and for the full extent of flooding within the study area as a whole. Finally, in moving forward, it is recommended that the Mike Flood model should be used to assess potential off-site flood impacts from proposed infrastructure and development projects.

# 1.0 INTRODUCTION

Valdor Engineering Inc. was retained by the TRCA to develop an integrated one-dimensional (1D)/two-dimensional (2D) hydraulic model using Mike Flood (Mike Flood) based on the latest available LiDAR surface data and 2015 flow data and to complete a flood study for the existing flood vulnerable area located in the vicinity of the Jane-Wilson Special Policy Area (SPA) including all areas within the extents of the Regional floodplain. Mike Flood couples the 1D and 2D hydrodynamic modules respectively referred to as Mike 11 and Mike 21.

## 1.1 Study Area

The study area, as shown in **Figure 1.1**, includes the Jane-Wilson SPA and is generally centered along Jane Street extending north to Gravenhurst Avenue and south to approximately 100 m south of Lawrence Avenue West. The study area extends west of Highway 400 and Black Creek Drive and east to approximately Montana Avenue and Fleetwood Avenue. In order to ensure that the simulated water surface elevations (WSEL's) and flows are not influenced by the boundary condition assumptions and model domain extents, it was necessary to extend the 2D hydraulic model beyond the identified study area although not to the same level of detail. The watercourse that traverses the study area is Black Creek which is a tributary of the Humber River.

## 1.2 Project Background

In June 2005, J.F. Sabourin & Associates completed floodplain mapping of the Jane-Wilson SPA for the TRCA. At that time, HEC-RAS was the model chosen to delineate the floodplain. Since that time, concerns arose that the complex hydraulics, which consist of several different spill locations, flow around buildings, and several locations where water is flowing in multiple directions, are beyond the capabilities of 1D hydraulic modelling software. The hydraulics through the study area are complex and accurate representation of these complex flow paths using the 1D HEC-RAS model is beyond its capability, as the 1D models assume that the flow is in one direction only, and there is no direct modeling of changes in flow distribution, cross-section shape, flow direction, or other two and three-dimensional properties of the flows. It has been proven that 1D channelized flows are represented accurately using 1D Hydraulic model, such as HEC-RAS and Mike 11 and these 1D-hydraulic models simulate flows over and through a large range of hydraulic structures with greater accuracy. The use of a model in a manner that contravenes the approximations upon which the model is based, can lead to gross errors in the model predictions. The consequences may lie anywhere between negligible and catastrophic potentially leading to property damage and loss of life. As the 2D hydraulic models make no implicit assumptions about flow direction or magnitude during analysis and capture the flooding caused by momentum and wave propagation during a flood event, the 2D hydraulic model can accurately represent local variations of velocity and water levels and local changes in flow direction through the study area. Given the difficulties in providing accurate and justifiable floodplain information generated from existing 1D HEC-RAS model to effectively manage the flooding risk to the people and property within this area, TRCA initiated a coupled 1D/2D hydraulic modeling project to accurately represent the river channel flow and the complex hydraulics within the floodplain of the study area. The advantage of using a coupled 1D/2D flood model is that it maintains a highly accurate representation of 1D flow, including bridge structures, along the main channel(s) while the overbank flow is handled by a fully hydrodynamic 2D flow model with detailed topographic information of the floodplain area. TRCA selected Mike Flood, due to the fact that it provides a dynamic coupling between a 1D river flow model and a fully 2D overland flow model.

The original design of the Black Creek Channel used a much lower design standard than what is currently used for regulation, and as such, the current HEC-RAS modelling shows water spilling from the concrete lined channel into the urban area. As such, a significant percentage of the Regional flow is not conveyed within the channel. The system is further complicated by several low lying roadway bridges that contribute to limiting the capacity in the channel. Based on the current HEC-RAS modelling, the predominant hydraulic constraint within the study area is the arch culvert passing under Highway 401 which causes ponding upstream.

In 2014, the TRCA retained the services of Civica Infrastructure Inc. to complete the *Humber River Hydrology Update* which provided updated flow information. Currently, the study area is mapped on TRCA's Floodplain Mapping Program, Humber River Sheets 8 and 9 and the topographic information for these maps is based on vertical air photos flown in 1997 and compiled by Northway Map Technology.

In 2013, TRCA acquired high-resolution digital elevation mapping (DEM) using LiDAR for the study area. In addition, the TRCA acquired 2005 3D ESM topographic mapping, which includes all buildings, roads, river outlines, and other information typically shown on TRCA's floodplain mapping.

There is a TRCA Real-Time Flood Monitoring Network water level gauge, located within the study area north of Highway 401, on the upstream side of Downsview Avenue. Water levels have been collected at this location since July 2007. The data is primarily used for flood warning purposes, and as such, a rating curve has not been developed. Water level data from this stream gauge were used in hydraulic model validation.

## 1.3 Purpose of Study

The purpose of the study is to develop a Mike Flood model to define the extent of flooding for the 2-yr through 350-yr design storms and the Regional storm (Hurricane Hazel), and to prepare new engineered floodplain mapping for study area. In addition, the study is intended to assess the causes of flooding and to determine the flood risk level conditions.

## 1.4 Study Scope

The scope of this project is limited to the following components:

- Review all available background information to be provided by the TRCA.
- Complete field surveys to verify the drainage network, topographic features (spill points, underpass/overpasses, flood barriers, bridge road decks, berms, retaining walls, high points, flow split areas, and any other obstructions such as concrete walls, solid railings, etc.), and to prepare an inventory of the structures.
- Acquire additional data to fill the data gap/needs (*e.g.* buildings, parking lots, extend DEM, etc.) for the bathymetry.
- Finalize data layers that identify the main topographic features to be considered in 1D and 2D model development and floodplain mapping.
- Create an existing condition hydraulic model of the study area using Mike Flood.
- Complete an initial model run to check the adequacy of the model domain extent considering possible increased flooding extent, appropriateness of boundary locations, and further data needs such as extended DEM data, digitizing additional buildings, other land use layers, etc. Create a final model based on the identified adjustment requirements.
- Perform a validation exercise using observed water levels for three storm events on 14 September 2008, 28 May 2013 and 08 July 2013.
- Run the model using the steady peak flow and unsteady flow hydrograph for Hurricane Hazel and compare the Mike Flood results with existing HEC-RAS WSEL's.
- Create and run the final Mike Flood model of the study area for each scenarios based on the 2-yr through 350-yr storms and the Hurricane Hazel.
- Post process the results using GIS and Mike View Plot Composer for flood depth maps,

velocity maps, depth-velocity product maps, flow direction maps, flood risk maps and create video animations.

- Prepare the characterization and risk assessment of the floodplain.
- Identify hydraulic constraints.
- Prepare floodplain mapping showing the depth and extent of flooding corresponding to the steady peak Hurricane Hazel simulation, and prepare updated engineered floodplain map sheets for the study area to replace the current HUM 08 and HUM 09 map sheets.
- Provide conclusions and recommendations.

## 1.5 Previously Completed Available Studies and Information

A review of the following studies and key design drawings provided by the TRCA was completed in preparing the report for the *Floodplain Mapping in Jane and Wilson Special Policy Area, Black Creek*:

- Aquafor Beech, *Humber River Watershed Hydrology Update*, November 2002.
- Civica, *Humber River Hydrology Update*, June 2015.
- DHI, *A Modeling System for Rivers and Channels – Reference Manual*, 2011.
- DHI, *Mike 21 Flow Model, Hydrodynamic Module, Scientific Documentation*, 2011.
- J.F. Sabourin, *Humber River Floodplain Map Sheets 7, 8 and 9*, 2005.
- TRCA, HEC-RAS Model for the Humber River, February 2011.

# 2.0 EXISTING CONDITIONS HYDRAULIC MODEL DEVELOPMENT AND ASSESSMENT (USING MIKE FLOOD)

## 2.1 Data Review and Preparation

### *2.1.1 Available Data Sets*

Available data were collected from various sources as follows:

TRCA:

- High resolution (0.5m X 0.5m) raster surface derived from LiDAR Survey (2012). The LiDAR data was captured on 27 and 28 November 2012.
- TRCA smoothened contours for the LiDAR data extent.
- Topographic data - 10m grid point elevation data (2002).
- 2011 digital orthophoto.
- GIS data layers for land use, water course locations, road paved areas, large parking lots, building footprints.
- Existing TRCA approved floodline map sheets (HUM 07, HUM 08 and HUM 09, 2005) including photogrametrically corrected base mapping 2006 for the study area.
- Current TRCA HEC-RAS Model, October 2015.
- Flow data: 2, 5, 10, 25, 50, 100, 350-yr unsteady hydrographs and flow node maps extracted from the *Humber River Hydrology Update* (Civica, June 2015).
- Hurricane Hazel unsteady hydrographs extracted from the *Humber River Hydrology Update* (Civica, June 2015).
- 14 September 2008, 28 May 2013 and 08 July 2013 storm event hydrographs for validation extracted from the *Humber River Hydrology Update* (Civica, June 2015).

City of Toronto:

- As-constructed bridge drawings.

MTO:

- Highway 401 as-constructed drawings including culverts, overpasses/underpasses.

### *2.1.2 Digital Elevation Model*

Two main sources of elevation data were integrated to create a base digital elevation model in GIS. These data sources were the LiDAR elevation surface (2012), used for the majority of the study area, and the 10 m grid point elevation data (2002), used at the edge of the study area. The information extracted from the survey data (by the TRCA and Valdor) and the available drawings (by MTO and the City of Toronto) was used to upgrade, update and eventually transform the base digital elevation model into a 2D overland area bathymetry and 1D channel cross-sections. This was necessary to accurately represent various localized elevation features such as road deck surfaces, solid railings, under/overpasses, berms, channel cross-sections immediately upstream and downstream of culverts, bridges, highway ramps, low flow channel sections and drop structures in the channel, retaining walls, etc. **Figure 2.1** illustrates the various data layers that were prepared and used in the model development.

*LiDAR Elevation Surface*

The TRCA acquired high-resolution digital elevation data using LiDAR for the study area. The data acquisition was conducted on 27 and 28 November 2012. The collected LiDAR mass point elevation data was processed into a hydro-enforced DEM using break lines for rivers and streams. Valdor received this hydro-DEM data that was converted into a 0.50 m x 0.50 m raster surface for the study area. Road deck surfaces at overpass locations were removed to ensure the presence of appropriate water flow paths at those locations. **Figure 2.1** shows the extents of the LiDAR DEM. It is important to note that the elevation in the LiDAR DEM for the low flow channel area does not represent the bathymetry of the low flow channel, rather it corresponds to the elevation of the water surface at the time the LiDAR data was obtained. The road surface elevations at the culverts and bridges were removed and replaced by the low flow WSEL's. Verification of the LiDAR was completed using a number of points obtained during the topographic survey. Based on this verification, the LiDAR was found to be within less than 5 cm of the surveyed data (see **Table A.1** in **Appendix A**).

*10 m Points Elevation Data*

The surrounding areas between the LiDAR data extents and the 2D model area extents was not covered by the LiDAR elevation surface. The TRCA supplied the GIS file with 10 m points elevation data was converted into a raster DEM to represent this area. The 10 m point elevation data was converted into a raster grid with the same spatial resolution and map projection system as those used in the LiDAR coverage area. The elevation surface of these surrounding areas was integrated with the LiDAR surface to create a DEM with 0.50 x 0.50 m grid resolution (see **Figure 2.2**) for the entire 2D model extents.

### *2.1.3 Base Mapping Data Layers and Land Use for Roughness Map*

The TRCA acquired 2005 3D ESM topographic mapping, which includes all buildings, roads, river outlines, and other information typically shown on the TRCA's floodplain mapping. The planimetric data used to update base mapping were compiled by First Base Solutions from aerial photography flown in 2006. The TRCA updated the land use data layers such as buildings, roads, parking lots, natural and overbank areas using available 2011 digital orthophoto and 2013 imagery. Finally, the TRCA prepared new base mapping for the study area that combines the LiDAR topographic information (*i.e.* contours and spot elevations) with the 3D topographic features. Using the TRCA's base mapping updated polygon, a roughness map was prepared using GIS (see **Figure 2.3**). Each category of the roughness polygon was assigned with a roughness values using TRCA's standard roughness table.

The roughness value for urban residential composite area (*i.e.* area includes lawns, misc. hard surfaces within lots, boulevards with driveways and sidewalks but excludes buildings) was determined based on weighted average calculations using data from two sample sites. The calculations and the sample areas are shown in **Appendix A** (see **Table A.2a** and figures for Sample Site 1 and Sample Site 2 areas)

The roughness value for urban industrial/commercial (*i.e.* area includes misc. small boulevard areas including grass, driveways and sidewalks but excludes buildings, large parking and natural areas) was based on weighted average calculations for a representative location within the study area using standard TRCA values as shown in **Table A.2b.**

### *2.1.4 Additional Site Survey and Hydraulic Structure Inventory*

A topographic survey was completed by Calder Engineering Ltd. in June 2015. Information from the field survey was used to verify and confirm the crossings within the study area on Black Creek, supplement cross section geometry data for the hydraulic model, and to verify the LiDAR data. The survey included confirmation of the number of crossings, number, location and size of all openings and invert/low chord elevations. An inventory of key hydraulic structures was prepared and is provided in

**Table 2.2** and details are given in the hydraulic structure inventory sheets provided in **Appendix A**. Cross section surveys of the creek at various locations were used to supplement the LiDAR data in the channel where the accuracy of the LiDAR is less reliable, in particular where there is flowing or standing water which LiDAR cannot penetrate.

## 2.2 Mike 21 2D Overland Flow Model

The following are the main elements of the Mike 21 model setup:

- Bathymetry for the entire model extent
- Boundary conditions
- 2D roughness data

### *2.2.1 Bathymetry*

Bathymetry is, in general, a digital elevation surface representing the entire 2D modelled area in the Mike 21 modelling system. All computations (such as velocities $v_x$, $v_y$, flow fluxes $q_x$, $q_y$, etc.) in the Mike 21 overland flow model are based on the bathymetry. Accurate bathymetry is crucial to achieve accurate model results. The resolution and extent of the bathymetry are two important parameters that have significant influences on the model results.

Accurate bathymetry can be obtained from accurate topographical information, which includes land elevation information as well as other topographical features (*e.g.* extents and elevation of buildings, berms/retaining walls, road deck surfaces, solid railings etc.) that have significant influences on water movement.

The bathymetry extent, which is the same as the 2D model extent as shown in **Figure 2.1,** was defined with consideration to the following issues:

- Study area extent – the 2D model bathymetry was extended beyond the study area (see **Figure 2.1**). An adequate "buffer" reach upstream and downstream of the study area was maintained to avoid any undesirable boundary influences on the model results within the study area.
- The full extent of the available LiDAR data was utilized in the model bathymetry.
- Availability of important land use data layers such as buildings, road and parking paved areas, structure information and survey data

The bathymetry was created from the 0.50 m x 0.50 m raster DEM described in Section 2.1.2. This bathymetry covers the area of LiDAR coverage and beyond to the entire 2D model extent. Bathymetry was created based on flexible mesh (See **Figure 2.4**) using a 10 $m^2$ triangulation within the floodplain (+ assigned buffer) and a 50 $m^2$ triangulation outside of the floodplain (+ assigned buffer). The entire floodplain area was covered by the high resolution mesh to achieve better accuracy in the model computation. The low resolution area (*i.e.* 50 $m^2$ triangulation) is in general beyond the tentative floodplain area. A smaller mesh requires a much smaller time step interval (such as 0.05 seconds compared to the currently used 0.20 seconds) to avoid any instability during a model run. A smaller mesh size also results in a very large output file size as well as requires very long computer computation time. Several trial runs were completed using various time step intervals to reduce the overall run time. The 1D/2D link lines, which are used to define the integration point between the 1D and 2D coupled Mike Flood model, were prepared in GIS as much as possible in smaller segments (*i.e.* such as ten segments) to enable the model to create linkages at appropriate locations. The building layer and the two link lines defining the 1D channel were used to exclude areas for bathymetry (see **Figure 2.5**) generation. The building areas and the areas within the 1D channel were used as non-computational cells in hydrodynamic computations. The elevation above 155 m were assigned as land value in the DEM as those cells would never be flooded.

### *2.2.2 Boundary Conditions*

Boundary conditions for the Mike 21 model define how the flow and water levels will be controlled at the peripheral edges of the 2D model domain defined by the bathymetry limits. In Mike Flood, the 2D boundary is typically a condition at the outer edges to specify how the edges of the model domain will behave during the model run. The upstream inflow boundary is typically defined in the Mike 11 model, while a link in the upstream between Mike 11 and Mike 21 needs to be specified. Similarly, another link between Mike 11 and Mike 21 at the downstream boundary is required to specify the location where the Mike 11 channel is discharging its routed flow into the 2D model domain. In addition, the 2D initial surface needs to be provided as an initial surface from which the 2D overland flow solver begins computations. Typically, in Mike Flood, the following boundaries need to be defined:

- The upstream inflow, $Q_{in}$ boundary is for the Mike 11 channel
- The downstream water level, WL (or outflow, $Q_{out}$) boundary for the Mike 11 channel
- One single or multiple boundaries, as appropriate, for all open edges of the Mike 21 model domain
- An appropriate initial water surface is required when the 2D model domain has large water bodies, lakes, river estuaries, bays or an artificial pool

In this study, the Mike 11 downstream (WL) boundary, the Mike 21 outer edge (WL) boundaries and the Mike 21 initial water surface to represent the artificial pool surface were derived from the HEC-RAS Regional WSEL provided by the TRCA. The Regional WSEL 115.51 m was used as a boundary for the Hurricane Hazel hydrograph simulation and the corresponding steady peak flows were used as the inflow boundaries as shown in **Table 2.1.**

### *2.2.3 Roughness Parameters*

The Mike 21 overland flow solver uses roughness parameters for each grid cell when completing computations. A roughness map (**Figure 2.3**) was prepared based on available land use/land cover information. In Mike 21, the roughness was defined in terms of Mike system's Manning's Resistance number (M), which is effectively the inverse (*i.e.* 1/n) of the Manning's roughness coefficient value (see **Figure 2.6**).

Roughness values and corresponding Resistance numbers used in Mike 21 are:

- Overbank area: 0.08 (M = 12.50)
- Road and large parking area: 0.025 (M = 40)
- Urban large pervious: 0.05 (M = 20)
- Natural area: 0.08 (M = 12.50)
- Urban residential composite (see **Table A.2a**): 0.039 (M = 25.64)
- Urban industrial composite (see **Table A.2b**): 0.043 (M = 23.26)

## 2.3 Mike 11 1D River Model

The Mike 11 Hydrodynamic (HD) module was used to model the low flow channel of Black Creek. The main elements of the Mike 11 model setup are:

- Establishing the channel network and creating cross-sections
- Structure modelling
- Boundary setup
- Roughness parameters

### *2.3.1 Channel Network and Cross Sections*

As accurate representation of the channel geometry is critical to represent flows in the channel, it was decided that the Mike 11 cross sections should be generated using the three available sources of information including the LiDAR DEM surface, the collected survey data and the existing HEC-RAS model for downstream and upstream locations to complete the model description in the Mike 11 model.

Using this approach, all the cross-sections for the 1D channel were drawn directly in the LiDAR DEM surface as per the Mike 11 model extents, river network line, two bank lines and cross-section cut marks shown in **Figure 2.1**. The cross-sections were cut at an interval of 10 m to 15 m. Higher density cross-sections were usually located near the areas of high interest as well as close to the bend areas in the 1D channel to define with more accuracy. MIKE Hydro, which is a map based graphical user interface for Mike products, was used to extract the cross-section data. It is noted that the LiDAR surface for the low flow channel area does not necessarily represent elevations of the channel bed/bathymetry. Therefore, each cross section generated using LiDAR data was modified at the bottom low flow channel portion, where the elevation values were replaced with the field survey information and observations.

### *2.3.2 Structure Modelling*

The structure information obtained from the survey (and in some cases the HEC-RAS model for lower and upper reaches) was manually inserted into the Mike 11 model. There are a total of 16 culverts/bridges C1 through C16 (see **Table 2.2**) with the model domain. Fifteen (15) of these culverts/bridges lie within or immediately upstream and downstream of the study area (see **Figure 2.1**). An additional bridge (C16 at the Lawrence Avenue crossing) is coded in the lower reaches of the modelled area to ensure model stability within the study area. Culverts/bridges C1 (at pedestrian bridge behind the Walmart plaza north of Wilson Ave.) to C15 (at Lawrence Ave W) are located within the study area where floodlines are to be delineated. Bridge C16 is located at Black Creek Dr. close to the downstream boundary in Mike11.

Culverts C1 through C5, C7 through C10, and C12 through C16 were incorporated in the model using the typical culvert structure approach of Mike 11. The road decks were incorporated using the typical weir modelling approach. Culvert C6 (at Hwy 401) was long, curved with variable sizes in upstream and downstream faces. Culvert C6 was incorporated using the closed irregular modelling option of Mike 11. The road deck at this location was used as a part of the 2D overland area surface. Unlike the typical structure modelling approach, the closed irregular sections (used for C6) enables Mike 11 to use of full hydrodynamic equations which computes velocities at every computational location over the length of the culvert and accounts for all internal changes in dimensions/size, slopes etc. As per the Mike 11 reference manual, these options are preferred to eliminate instabilities, specifically when the culverts are long. Since hydrodynamic equations are used in the computations by the Mike 11 system, the actual (variable) velocities at different internal locations of the culverts are calculated based on variations in internal dimensions/sizes, slopes, and surface properties over the full culvert length. The bridge/culvert C11 was not included in Mike 11 as it was evaluated to have no significant impact on the channel flow computation. The road deck is very high compared to the high water level. However, the road deck surface was removed from the bathymetry to allow flow through this channel section location.

### 2.3.3 Boundary Conditions Setup and Mike 11 Inflow Hydrographs

Two boundaries need to be specified for the Mike 11 river model. The upstream boundary is typically a constant or time series discharge, while the downstream boundary is usually a constant or time series water level. The inflow hydrographs for all the return period events including 350-yr and Hurricane Hazel were used as the Mike 11 inflow boundaries at five different locations in the 1D channel. The Mike 11 inflow boundary locations are described in **Table 2.1.** The flow node locations corresponding to these Mike 11 inflow boundaries are shown in **Figure 2.8**. The inflow and downstream boundaries used in various model simulation scenarios are provided in **Table 2.3.**

**Table 2.1: Mike 11 Model Steady Peak Inflow Boundary**

| Mike 11 Inflow Boundary Locations | Mike 11 Input (Regional Storm) | Mike 11 Input (350-yr Storm) | Remarks |
|---|---|---|---|
| U/S boundary of the 2D Model Extent (Node N1) | 344.21 cms | 254.13 cms | Open inflow boundary |
| Black Creek Tributary at the East Boundary of the 2D Model Extent (N2) | 24.03 cms | 21.73 cms | Open inflow boundary |
| Between Hwy 401 and Black Creek Drive (Node N3) | 83.11 cms | 74.78 cms | Point source inflow boundary |
| Between Black Creek Drive and Lawrence Avenue (Node N4) | 64.72 cms | 56.44 cms | Point source inflow boundary |
| At Lawrence Avenue (Node N5) | 50.16 cms | 43.73 cms | Point source inflow boundary |
| Mike 11 D/S End | 115.51 m | 114.29 m | Open water level boundary |

TRCA provided the inflow hydrographs (see **Figures A.1** to **A.8** in **Appendix A**) for all return periods corresponding to the five inflow nodes (N1, N2, N3, N4, and N5). The inflow hydrographs corresponding to future land use were used in the Mike Flood existing condition models. The downstream boundaries were WSEL's from the existing HEC-RAS model. The Mike 11 D/S water level boundary was located downstream of the Black Creek Dr. bridge where the Mike 11 channel has a standard link (used to enable the coupling at the upstream or downstream end of the 1D channel with the 2D overland area) with the Mike 21 bathymetry. There are a number of ways to set boundary conditions that are appropriate and we used the method whereby the WSEL was set using an artificial pool.

Sensitivity analyses were completed, whereby the WSEL at the boundary locations for both Mike 11 and Mike 21 were varied by over 2.0 m above the HEC-RAS WSEL. It was confirmed that there are no significant impacts to WSEL's at the study area limit up to Lawrence Ave. which was included in the TRCA's floodplain mapping coverage. Regardless of the approach taken to describe the boundary conditions, given the large variation in WSEL that we are using for the sensitivity analysis, there would be no change in the conclusions drawn to confirm the limit of the study area and floodplain mapping extent and be confident in the results. In addition, any changes in the results downstream of the study area limits due to different considerations in setting the boundary conditions is not relevant since we are not using any results (*i.e.* beyond the study area limits) and the reach length beyond the study area limits is only provided to achieve model stability.

#### *2.3.4 Roughness Parameters*

The TRCA standard Manning's roughness coefficients were used for the channel sections and the structures as follows:

- Natural channel (low flow): 0.035
- Overbank area: 0.08
- Concrete channel: 0.015
- Concrete bridge: 0.013

### 2.4 Mike Flood – Model Simulation and Output

#### *2.4.1 Coupling the 1D and 2D Models*

The final steps of the Black Creek 2D model setup was the integration of the 1D Mike 11 channel model with the 2D overland area Mike 21 model using the Mike Flood interface model. This integration in Mike Flood allows a seamless flow exchange between the 1D river and the 2D overland areas thereby enabling the space and time-dependent dynamic simulation of flows as occurs physically in real-world hydraulic systems. This integration is facilitated by coupling together the 1D Mike 11 model and the 2D Mike 21 model in two ways as follows:

- A lateral link is set up that enables the coupling along the left bank and right bank of the 1D channel with the 2D overland areas. The model allows for dynamic exchange internally in both directions between the 1D channel and 2D floodplain flow components. Flow through the Mike 11 domain into the Mike 21 domain, and vice-versa, is via a lateral boundary that is applied to Mike 21 via a source term. Flow through the link is dependent on a structure equation and the water levels in Mike 11 and Mike 21. Flow through the link is distributed among several Mike 11 water level points and several Mike 21 grid cells.
- A standard link is set up that enables the coupling at the upstream or downstream end of the 1D channel with the 2D overland area

**Figure 2.7** shows the final bathymetry of the Black Creek Mike Flood model. The 1D river area was blocked by the white cells (after the link was established between the 1D and the 2D models) so that these cells were not used in the Mike 21 computations since the computations at these cell locations is performed by the Mike 11 model.

#### *2.4.2 Simulation Parameters*

The total duration of the hydrodynamic simulation was 15 hours for the steady peak flow simulation. The first six (6) hours were used to create a mild rising limb of the hydrograph and the remaining nine (9) hours were used to run the simulation with steady peak inflow. A mild sloped rising limb was used to avoid possible generation of any undesirable momentum that could result from a steeper rising limb of the hydrograph. It was determined that seven (7) hours of simulation time was adequate to allow the model to convey the peak flows through all the 1D and 2D model without accounting for the effect of storage elements within the floodplain area, such as depression storage, and sinks.

The intent of the study is to map maximum water levels – the additional run time is not necessary as the water level does not increase beyond that point. In general, the total number of time steps used for a steady peak flow simulation was 270,000. The time step interval was set to 0.20 seconds for every simulation, which ensured adequate capture of the peak flow response by each of the smallest 2D grid cells and the 1D channel computational points.

### *2.4.3 Model Output*

The key Mike Flood model outputs are dynamic flood depth (H) and flow flux in the x-direction ($q_x$) and the y-direction ($q_y$). Post-processing was completed within the Mike system to create flood velocity in the x-direction ($V_x$) and the y-direction ($V_y$), resultant flood velocity ($V_r$) and 2D WSEL for each of the model run scenarios. The outputs were converted into ArcGIS format. ArcGIS was used to process the output layers to prepare the Regional flood depth maps which show the extent and depth of flooding. A velocity map was created using ArcGIS. Flood animations were created using $V_x$, $V_y$ velocity vector components with the dynamic flood depth in the background that shows flow path and flow direction. The flood animations provide a better understanding of how the flood water propagates over the study area in space and time for any changes in the basic and hydro-dynamic parameters.

## 2.5 Existing Conditions Regional Flood Assessment (Using Mike Flood)

### *2.5.1 Existing Drainage Conveyance System*

Black Creek flows in a south-easterly direction through the Jane Street and Wilson Avenue SPA, across Highway 401, Jane Street, Black Creek and Lawrence Avenue. The creek consists of a natural channel north of the Sheridan Mall (located north-west of the Jane Street and Wilson Avenue intersection and south of approximately Queen's Drive. In between these locations, the creek consists of a concrete lined channel with a top width of approximately 8 to 18 meters. A total of 15 bridges and culverts are located along this reach of Black Creek that were identified for inclusion in the Mike Flood model. An additional bridge under Black Creek Dr. south of Lawrence Ave. was included in the model downstream of the study area. There are two major storm sewers that outlet to Black Creek within the study area. The north storm sewer is enclosed from north of Heathrow drive to the south of William Cragg Drive at the outfall at Black Creek. The south storm sewer extends from Maple Leaf/Edison Circle to an outfall at Black Creek located north of Lawrence Avenue and west of Black Creek Drive.

The study area (including reach lengths immediately upstream and downstream of the study area) consist of culverts and bridges on Black Creek which are described in **Table 2.2**.

**Table 2.2: Culvert Locations and Details**

| Structure ID | Structure Location | Structure Type |
|---|---|---|
| C1 | Black Creek West of Jane St. and North of Wilson Ave. - North of Sheridan Mall Parking Area | Concrete Pedestrian Bridge |
| C2 | Black Creek West of Jane St. and North of Wilson Ave. – Access road to Sheridan Mall | Concrete Bridge |
| C3 | Black Creek at Jane St.North of Wilson Ave., East of Sheridan Mall | Concrete Bridge |
| C4 | Black Creek at Wilson Avenue, East of Jane St. | Concrete Bridge |
| C5 | Black Creek at Downsview Ave., East of Jane and North of Hwy 401 | Concrete Bridge |
| C6 | Black Creek at Hwy 401 | Concrete Culvert |
| C7 | Black Creek at Jane St., South of Hwy 401 | Concrete Bridge |
| C8 | Black Creek at Gordon Mackay Road, West of Jane St. | Concrete Bridge |
| C9 | Black Creek at Jane St. South bound to HWY 400 North Ramp | Concrete Bridge |
| C10 | Black Creek at Jane St, North of Highway 400 South | Concrete Bridge |
| C11 | Black Creek at Jane St. North bound to HWY 400 North Ramp | Concrete Bridge |
| C12 | Black Creek at HWY 400 (and Black Creek Drive) | Concrete Bridge |
| C13 | Black Creek at Maple Leaf Drive, East of Jane St. | Concrete Bridge |
| C14 | Black Creek at Queen's Drive, East of Jane St. | Concrete Bridge |
| C15 | Black Creek at Lawrence Avenue West, West of Black Creek Dr. | Concrete Bridge |
| C16 | Black Creek at Black Creek Drive, South of Lawrence | Concrete Bridge |

Additional culvert details and photos are provided in the hydraulic structure inventory sheets included in **Appendix A**

### *2.5.2 Mike Flood Model Scenarios*

Various scenarios were investigated for the Jane-Wilson area based on actual storms and return period storm events. Different downstream boundary conditions were used for different return period storm event simulations. A list of the model run scenarios completed is provided in **Table 2.3**.

**Table 2.3: List of Scenarios for the Jane-Wilson 2D Model Simulation Runs**

| Scenario No | Storm Event/ Inflow Hydrograph | Downstream Boundary WSEL | Simulation period |
|---|---|---|---|
| S01 | Regional steady peak flow (Hurricane Hazel) | 115.51 m (Regional WSEL at d/s HEC-RAS section) | 15 hours |
| S02 | 350-yr steady peak flow | 114.29 m (350-yr WSEL at d/s HEC-RAS section) | 15 hours |
| S03 | Regional unsteady hydrograph (Hurricane Hazel) | 115.51 m (Regional WSEL at d/s HEC-RAS section) | First 46 hours |
| S04 | 100-yr steady peak flow | 113.95 m (100-yr WSEL at d/s HEC-RAS section) | 15 hours |
| S05 | 50-yr steady peak flow | 113.76 m (50-yr WSEL at d/s HEC-RAS section) | 15 hours |
| S06 | 25-yr steady peak flow | 113.55 m (25-yr WSEL at d/s HEC-RAS section) | 15 hours |
| S07 | 10-yr steady peak flow | 113.32 m (10-yr WSEL at d/s HEC-RAS section) | 15 hours |
| S08 | 5-yr steady peak flow | 112.83 m (5-yr WSEL at d/s HEC-RAS section) | 15 hours |
| S09 | 2yr steady peak flow | 112.43 m (2-yr WSEL at d/s HEC-RAS section) | 15 hours |

The scenario model runs and corresponding results are described in the following sections.

### *2.5.3 Comparison of Mike Flood Results for Steady and Unsteady Input Hydrographs – Regional Storm*

The Mike Flood model was run using both steady and unsteady flow input hydrographs. The results using the steady flow input hydrograph for the Regional storm is provided in **Figure 2.9** and the results using the unsteady flow input hydrograph is provided in **Figure 2.10**. The extent of flooding using the steady flow input hydrograph is generally similar but greater than the results using the unsteady flow input hydrograph. Based on discussions with the TRCA, it was determined that the steady flow input hydrographs would be applied to all model runs for this study.

*Understanding the Mike Flood Results - Points to Consider*

Mike 11 (or Mike 21) flow simulation, using either steady peak or unsteady hydrographs, is based on the fully dynamic wave description (using St. Venant Equations: conservation of mass and conservation of momentum), where flow conditions change over time and space. All the governing forces (*i.e.* gravitational, frictional and static) causing water movement are changing from point to point over space and time based on the water surface slope, bed slope, and roughness characteristics. As a result, the Mike Flood model simulated flow condition (*i.e.* velocity, depth, flow and flow direction) changes from one point to another and with time at every point. This flow type is referred to as unsteady non-uniform flow. The simulation is called unsteady (and non-uniform) flow simulation or fully dynamic flow simulation.

In contrast, steady uniform flow simulation is based on the conservation of mass only, where the flow condition does not change over time and space. Computations are completed step by step for each time step assuming WSEL and velocity are constant between two sections within each time step interval. Unlike unsteady non-uniform flow simulation, there is no consideration of change in momentum, a consideration that results in acceleration to water movement in unsteady simulation.

In Mike Flood, we may have a constant inflow (such as the Hazel peak inflow) input to the Mike 11 model at any upstream location in the channel while simulation results show flow conditions change from point to point and with time at every point as the governing forces are changing at any point over space and time. However, if the steady peak flow input is used for a very long simulation period, any location over a 2D surface or in the channel may achieve a steady state flow condition provided that no other new forces start acting at any location (for example such forces could be due to tidal flow or wall effect at any location). The reason for this steady state condition is that after a long steady input duration, the governing forces at that particular location remain unchanged.

### *2.5.4 Existing Flood Assessment and Results Using Mike Flood – Regional Storm (Steady Flow Input)*

The model run for the Regional storm flood simulation was carried out using the Regional inflow and boundary conditions as shown in **Table 2.2.** The model results in terms of flood depth and extent, velocity, depth-velocity product (flow flux per metre) and flow direction were post-processed by Mike View and GIS to prepare the maps as shown in **Map 2.1** in **Appendix A**, **Figure 2.9** and in **Appendix B** (**Figures B.1, B.2** and **B.3**). The existing conditions model results clearly show that the entire Jane Street corridor between south of Heathrow Drive and Hwy 400 is within the Regional floodplain. The floodplain extends west of Jane Street to Haymarket Road and spills across Highway 401 where areas to the east and west of Jane Street are flooded south to Black Creek Drive. Highway 400 is not overtopped. Downstream of Black Creek Drive, the floodplain is primarily confined to the valley lands, however, Maple Leaf Drive, Queen's Drive and Lawrence Avenue is overtopped. It is noted that the Regional flood spills onto Black Creek Drive at Maple Leaf Drive and Lawrence Avenue.

In **Figure 2.11**, the revised existing conditions flood depth and extent map prepared by Valdor is overlaid with the current approved floodlines (J.F. Sabourin, 2005). The revised flood depth map based on the 2D model results demonstrates how flow spills through the Jane Street and Wilson Avenue SPA, spills over Highway 401, between Highway 401 and Black Creek Drive and to Black Creek Drive south of Maple Leaf Drive. Generally, the flooding extents follow the floodlines as per the 2005 floodplain mapping, however, the extents are reduced in some areas and the extent of flooding at previously identified spill locations is better defined. Areas with no color shading are free of flooding on the revised flood map.

The revised existing flood depth map was prepared based on a completely integrated 1D channel flow system and a 2D overland surface (*i.e.* proposed study area) in a high resolution geo-spatial system while accurately incorporating all major physical features including roads, buildings, depression storage areas, numerous flow paths, all obstructions and any spatial variations in the land surface that affects the movement of water. The modeled area, its parameters and spatial resolution were discussed and confirmed with the TRCA.

### *2.5.5 Existing Flood Assessment and Results Using Mike Flood – 2-yr to 350-yr Storms (Steady Flow Input)*

Additional model runs were completed using the Mike Flood program for the 2-yr to 100-yr design storms and the 350-yr design storm and a summary of the extent of flooding for the various design storms is summarized below.

*350-yr Storm*

The existing conditions model results for the 350-yr storm are similar to those for the Regional storm although flow depths and the extent of flooding is slightly reduced. The entire Jane Street corridor between south of Heathrow Drive and Hwy 400 remains within the 350-yr storm floodplain. The floodplain continuous to extend west of Jane Street to Haymarket Road and spills across Highway 401 where areas to the east and west of Jane Street are flooded south to Black Creek Drive. Downstream of Black Creek Drive, the floodplain is primarily confined to the valley lands, however, Maple Leaf Drive, Queen's Drive and Lawrence Avenue is overtopped. The 350-yr storm flood continuous to spill onto Black Creek Drive at Maple Leaf Drive and Lawrence Avenue. The results for the 350-yr design storm are provided in **Figures B.4** through **B.7** in **Appendix B**.

*100-yr Storm*

The extent of flooding based on the existing conditions model results for the 100-yr storm is noticeably less than for the Regional storm and the 350-yr design storm. A smaller portion of the Jane Street corridor between south of Heathrow Drive and Hwy 400 remains within the 100-yr storm floodplain. The floodplain no longer extends west of Jane Street to Haymarket Road and does not spill across Highway 401. Flooding along the Jane Street corridor between Hwy 401 and Black Creek Drive is less extensive than for the Regional storm and the 350-yr storm. Downstream of Black Creek Drive, the floodplain is primarily confined to the valley lands, however, Maple Leaf Drive, Queen's Drive and Lawrence Avenue is overtopped. The 100-yr storm flood continuous to spill onto Black Creek Drive at Maple Leaf Drive and Lawrence Avenue. The results for the 100-yr design storm are provided in **Figures B.8** through **B.11** in **Appendix B**.

*2-yr to 50-yr Storms*

The extent of flooding based on the existing conditions model results for the 2-yr to 5-yr storms is generally confined to the watercourse that traverses the study area although some flooding occurs along Jane Street between south of Heathrow Drive and north of William Cragg Drive, at Jane Street north of Black Creek Drive, at Maple Leaf Drive and Queen's Drive, and at Lawrence Avenue and south along Black Creek Drive. In addition to the extent of flooding for the 2-yr to 5-yr storms, the 10-yr storm flow results in multiple spills from the watercourse top of banks and several areas of flooding along the Jane Street corridor between south of Heathrow Drive and Black Creek Drive. The extent of flooding for the 25-yr to 50-yr storm flows is noticeably greater along the Jane Street corridor between south of Heathrow Drive and Black Creek Drive. Spill to Black Creek Drive from north of Maple Leaf Drive begins to occur for the 25-yr storm. The results for the 2-yr to 50-yr design storms are provided in **Figures B.12** through **B.16** in **Appendix B**.

# 3.0 MODEL VALIDATION AND COMPARISON WITH EXISTING HEC-RAS MODEL

The results of the Mike Flood model were compared with measured water levels from the TRCA operated flow monitoring gauge located at Downsview Avenue for a range of available storms. In addition, the results of the Mike Flood model were compared with the results of the current approved HEC-RAS model. The methodology and results of these comparisons are described in the sections that follow.

## 3.1 Mike Flood Model Validation

There is an existing stream gauge located at Downsview Ave. that is operated by the TRCA. Based on a review of the available gauge water level data, the TRCA was able to compile data for the following three events:

- 14 September 2008;
- 28 May 2013; and
- 08 July 2013.

It is noted that the stream gauge went offline during the 08 July 2013 event, however, the TRCA requested AMEC to prepare a modelled data set for this event. The data is raw and preliminary with no QA/QC completed, excluding the modelled data prepared by AMEC.

In addition, the TRCA prepared and provided to Valdor simulated hydrograph output for the identified flow nodes within the study area for the selected validation events using the recently updated VO3 Humber River hydrology model. The flow output hydrographs from the updated hydrology model for the three selected rainfall events were input to the Mike Flood model and run to simulate the WSEL's at the same location as the Downsview Ave. stream gauge.

A comparison of the actual (*i.e.* measured at stream gauge) WSEL's and the simulated (*i.e.* using Mike Flood) WSEL's at the location of the Downsview Ave. stream gauge were plotted and compared (see **Figure 3.1**). There is generally good agreement between the measured and simulated WSEL's for the selected validation events. In order to avoid unnecessarily long model run times when simulating the validation events, the model runs were started at a specified time prior to the rise in water level associated with the event and run for approximately 15 to 25 hours. As such, the simulated results in the figure are truncated accordingly. Comparing the peak flows, the difference between the measured and simulated water levels is approximately 0.5 m for the 08 July 2013 event and less than 0.1 m for the 14 September 2008 storm and the 28 May 2013 storm. When comparing points other than the peak, there are greater differences at certain times, however, these could be attributed to a number of factors such as timing calculations in the hydrology model.

Given the generally good replication of measured results by the Mike Flood model for the validation events, it was confirmed that there are no significant errors made or assumptions used in the preparation of the 1D/2D integrated hydraulic model. It is noted that during model simulation both the 14 September 2008 and 28 May 2013 storm flows were contained within the channel (*i.e.* defined by the 1D Mike 11 model) upstream of the Downsview Avenue bridge while the 08 July 2013 storm flow spilled to the floodplain (*i.e.* defined by the 2D Mike 21 model) to the west of the channel. The following uncertainties were noted in the available data for model calibration:

- There could be inaccuracies between the geodetic reference data used for the stream gauge

WSEL's and the WSEL's calculated in the Mike Flood hydraulic model.

- The flow hydrographs available for input to the Mike Flood model are not measured flow data, but rather simulated hydrographs based on the updated VO3 hydrology model. The modeled hydrograph data may not accurately reflect the spatial distribution of the rainfall events that is reflected in the measured data.

On account of the above noted uncertainties, it was determined that calibration of the model is not warranted to further refine the results and that the calibration of the model using this data could potentially worsen the accuracy of the simulated results rather than improve the accuracy.

## 3.2 Comparison of Mike Flood Results with Existing HEC-RAS Model

A comparison between the WSEL's calculated using HEC-RAS were compared to those calculated using Mike Flood. It is noted that in Mike Flood (unlike HEC-RAS), the WSEL's vary in all directions from cell to cell within the wet areas of the model domain. As such, it is difficult to make a direct comparison with the WSEL's in HEC-RAS which assumes the same WSEL along the entire cross section line. Due to this practical limitation and for a consistent comparison between WSEL's as a result of the unidirectional 1D flow and multi-directional 2D flow computation approaches, it was decided to use the Mike 11 channel profile and compare points along this profile with WSEL's from the HEC-RAS sections at similar locations. The comparison of results is provided in **Figure 3.2** in graphical format. A tabulated summary of the comparison of results is provided in **Table A.3**. As shown, the difference in WSEL's varies between approximately -2.60 m +0.992 m. The agreement between both models varies depending on the location, however, it is noted that the greatest difference in WSEL occurs along the reach between Highway 401 and Black Creek Drive.

### *3.2.1 Possible Factors Contributing to the Differences between HEC-RAS Results and Mike Flood Results*

It is noted that HEC-RAS and Mike Flood are very different hydraulic software programs and vary considerably in their computational methodologies and level of sophistication. A number of factors have been identified that could contribute to the difference in WSEL's between the current approved HEC-RAS model and the Mike Flood Model, including the following:

- How flow splits are applied (or not applied) in HEC-RAS which can result in variations in flow at different locations between models.
- How flow inputs are distributed which can result in variations in flow at different locations between models.
- How the Manning's roughness values are used in these two models vary differently (varies cell by cell over 2D area in Mike Flood, while usually three variations are lumped along the HEC-RAS section).
- How ineffective flow areas are assigned in HEC-RAS which can impact flow conveyance calculations between models.
- Difference in methodology employed for handling flow obstructions which can impact flow conveyance calculations between models.
- Difference in methodology used for flow computations (1D vs 2D and static vs dynamic) which can result in spatial and temporal variations in flood depths between models.
- Location, configuration and number of cross sections used.
- HEC-RAS assumes that the flow direction at any cross-section is perpendicular to the section. The Mike Flood flow direction map (when overlaid with the HEC-RAS cross-section map) identifies many locations between Hwy 401 and Black Creek Drive where the flow direction deviates significantly from the HEC-RAS assumed direction.

- Time step used in Mike Flood.
- Hydrograph ramp duration used in Mike Flood.

### *3.2.2 Investigations Regarding Mike Flood Time Step and Hydrograph Ramp Duration and Manning's Roughness*

A sensitivity test was completed regarding both time step and hydrograph ramp duration in the Mike Flood model. Model runs were completed with a time step varying between 0.20 seconds and 0.50 seconds and ramp duration ranging between 6 hours and 10 hours. Based on the results, it was confirmed that the time step and hydrograph ramp duration used for the Hurricane Hazel storm was appropriate and that the results are not overly sensitive to changes in this regard.

Manning's roughness was also adjusted in the Mike Flood model to match more closely the roughness values assumed in the current approved HEC-RAS model. Although changes in WSEL were observed, the differences were not nearly significant enough to account for the differences in WSEL's observed between the two models, in particular along the reach between Hwy 401 and Black Creek Drive. As such, efforts were then focused on reviewing the current approved HEC-RAS model to determine if the manner in which the model was coded may be contributing to these significant differences in WSEL.

### *3.2.3 HEC-RAS Sensitivity Test*

In order to investigate how differences in model setup and the model parameter values used in the existing approved HEC-RAS model and those used in the Mike Flood model may contribute to differences in WSEL results between the two programs, a sensitivity test was completed using HEC-RAS. The key setup method and parameters selected for analysis included flow input distribution, Manning's roughness, flow obstructions and ineffective flow area. The changes in setup method and parameter values for the sensitivity test were applied between Culvert C2 (Sheridan Mall entrance) and Culvert C14 (Queen's Drive) (*i.e.* the main part of the study area).

*Flow Input Distribution*

It is noted that the flow node locations assigned in the existing HEC-RAS model are different than the locations assigned in the Mike Flood model. The locations of the flow inputs in the existing HEC-RAS model were modified to match the flow distribution assigned in the Mike Flood model to determine if this may be contributing to the observed differences in WSEL results between the two models. Based on the results of this sensitivity test, it was determined that the distribution of the flow inputs has some impact on WSEL but it is relatively minor. The differences in WSEL due to this change is summarized in **Table 3.1** for selected cross section locations.

**Table 3.1: Sensitivity Test (Flow Input Distribution)**

| HEC-RAS Section # | Current Approved HEC-RAS Model (with updated peak flows) (m) | HEC-RAS Model (with refined flow input locations) (m) | Difference (m) |
|---|---|---|---|
| 48.62 | 128.59 | 128.6 | 0.01 |
| 48.59 | 128.51 | 128.51 | 0 |
| 48.57 | 128.4 | 128.4 | 0 |
| 48.546 | 127.84 | 127.84 | 0 |
| 48.5121 | 124.89 | 124.89 | 0 |
| 48.48 | 121.82 | 121.9 | 0.08 |

*Manning's Roughness*

The current roughness values used in the existing HEC-RAS model vary between 0.013 and 0.08. In order to assess the sensitivity of the model results to Manning's roughness, the roughness values outside the channel were reduced to 0.015. As shown in **Table 3.2**, the calculated WSEL's for selected cross section locations were generally reduced by up to 1.13 m. As such, it was concluded that the HEC-RAS model results are quite sensitive to changes in Manning's roughness.

**Table 3.2: Sensitivity Test (Manning's 'n')**

| HEC-RAS Section # | Current Approved HEC-RAS Model (with updated peak flows) (m) | HEC-RAS Model (with reduced Manning's 'n') (m) | Difference (m) |
|---|---|---|---|
| 48.62 | 128.59 | 128.21 | -0.38 |
| 48.59 | 128.51 | 128.13 | -0.38 |
| 48.57 | 128.4 | 128.11 | -0.29 |
| 48.546 | 127.84 | 126.86 | -0.98 |
| 48.5121 | 124.89 | 124.14 | -0.75 |
| 48.48 | 121.82 | 120.69 | -1.13 |

*Flow Obstructions*

The current HEC-RAS model uses the section geometry to simulate the effect of buildings on floodplain hydraulics and WSEL. Given that flows in the Mike Flood model can move in all directions around buildings and other obstructions, a sensitivity test was completed with the existing HEC-RAS model to confirm how sensitive the model is to changes in the extent of the obstructions. It is noted that since HEC-RAS is a one-dimensional program, flow is only allowed to pass from upstream to downstream in between buildings. A sensitivity test was completed using the existing HEC-RAS model and removing all building obstructions. Based on this analysis, it was determined that some changes in WSEL were observed, however, the model results are generally not very sensitive to changes with or without the inclusion of buildings which seems unusual. The results of the sensitivity analysis for flow obstructions are provided in **Table 3.3** and the extent by which WSEL's change due to the removal of the building obstructions is up to 0.07 m. We would suggest that the HEC-RAS model results, in particular between Hwy 401 and Black Creek Drive are governed mostly by the ineffective flow areas applied and that the model results would otherwise be more sensitive to the changes in the extent of obstructions.

**Table 3.3: Sensitivity Test (Obstructions)**

| HEC-RAS Section # | Current Approved HEC-RAS Model (with updated peak flows) (m) | HEC-RAS Model (with obstructions removed) (m) | Difference (m) |
|---|---|---|---|
| 48.62 | 128.59 | 128.59 | 0 |
| 48.59 | 128.51 | 128.5 | -0.01 |
| 48.57 | 128.4 | 128.42 | 0.02 |
| 48.546 | 127.84 | 127.91 | 0.07 |
| 48.5121 | 124.89 | 124.89 | 0 |
| 48.48 | 121.82 | 121.82 | 0 |

*Ineffective Flow Area*

It was observed that the existing HEC-RAS model makes extensive use of ineffective flow areas for many sections other than those sections immediately upstream and downstream of the culverts and bridges. Upon closer inspection of the topography in these areas, it is uncertain why this approach was employed since effective flow should be occurring in many of these overbank areas based on the Mike Flood results. A sensitivity test was completed whereby the ineffective flow areas were removed from most locations other than those sections located immediately upstream and downstream of the various culverts and bridges. Based on this analysis, it was determined that WSEL's are very sensitive to how the ineffective flow areas are applied. The results of the sensitivity test are provided in **Table 3.4** and the effect of this adjustment results in a reduction in WSEL of up to 2.21 m.

**Table 3.4: Sensitivity Test (Ineffective Flow Area)**

| HEC-RAS Section # | Current Approved HEC-RAS Model (with updated peak flows) (m) | HEC-RAS Model (with adjusted ineffective flow areas) (m) | Difference (m) |
|---|---|---|---|
| 48.62 | 128.59 | 127.87 | -0.72 |
| 48.59 | 128.51 | 127.48 | -1.03 |
| 48.57 | 128.4 | 127.21 | -1.19 |
| 48.546 | 127.84 | 125.63 | -2.21 |
| 48.5121 | 124.89 | 124.89 | 0 |
| 48.48 | 121.82 | 121.61 | -0.21 |

### *3.2.4 Comparison between Modified HEC-RAS Model and Mike Flood Results*

In an effort to provide a more reasonable comparison between the HEC-RAS and Mike Flood model results, the current approved HEC-RAS model was modified to match as closely as possible the Mike Flood model regarding flow input distribution, Manning's roughness, flow obstructions and ineffective flow area as follows:

- The flow input locations in the approved HEC-RAS model were revised to reflect the refined flow input locations that were applied in the Mike Flood model.
- Based on a cursory comparison of Manning's roughness between the HEC-RAS model and the Mike Flood model, the range in HEC-RAS roughness values for cross sections between Culvert C3 and Culvert C14 were modified as appropriate in the overbank locations to between 0.03 and 0.08. It is noted that roughness values in Mike Flood are distributed on a cell by cell basis using the delineated land cover, whereas, roughness values in HEC-RAS are typically lumped within the channel and outside the bank stations in the left and right overbank areas for each cross section (*i.e.* simplified approach).
- The methodology employed to code flow obstructions in HEC-RAS is considerably different than that employed by Mike Flood. In HEC-RAS, flow obstructions are coded by assigning obstruction locations using the obstruction command or modifying the section geometry (approach used in current approved HEC-RAS model) which is applied in a one-dimensional plane as flows are conveyed from upstream to downstream along the watercourse reach. In Mike Flood, flow obstructions are coded as two-dimensional entities and flow results are calculated in multiple directions whereby flows can pass around obstructions such as buildings in all directions (*i.e.* the flow path is not limited to only one direction from upstream to downstream). In order to simulate the multi-directional flow capabilities of the Mike Flood model, the obstructions coded in the HEC-RAS model were arbitrarily reduced by approximately 30%.

- It was noted in reviewing the current HEC-RAS model that the ineffective flow areas were coded such that overbank flow was obstructed for much of the floodplain in many areas which does not seem to reflect the existing topography based on inspection and as modeled using Mike Flood. As such, the ineffective flow areas used in the current HEC-RAS model were modified to better reflect existing conditions and which are more representative of those modeled using Mike Flood.

The comparison of results between the modified HEC-RAS model and the Mike Flood model is provided in **Figure 3.3**. A tabulated summary of the comparison of results is provided in **Table A.4**. As shown, the difference in WSEL's varies between approximately -0.372 m and +0.809 m. Based on the results using the modified HEC-RAS model, the agreement between both models is much closer than the comparison made with the current approved HEC-RAS model, in particular along the reach between Hwy 401 and Black Creek Drive. As such, it appears that the noted differences in WSEL's between the two models are largely attributed to the manner in which the ineffective flow areas were applied in HEC-RAS and the difference in Manning's roughness values used, in combination with those additional factors identified earlier.

#### *3.2.5 Conclusions*

Based on the completed comparative analyses and sensitivity tests noted in the previous sections, it is our opinion that the Mike Flood model results are accurate, despite some differences in WSEL when comparing the current approved HEC-RAS model results to the Mike Flood model results. The key factors contributing to the differences in WSEL's, in particular between Hwy 401 and Black Creek Drive, appear to include the manner in which the ineffective flow areas were applied in the current approved HEC-RAS model and to a lesser extent the values used for Manning's roughness. It is noted that the comparison with HEC-RAS was completed only with the Mike 11 (1D) channel profile for a consistent comparison and that the Mike 21 (2D) results could be different.

# 4.0 FLOODPLAIN MAPPING

In Section 2, flood simulation results were compared based on the steady Hurricane Hazel peak flow hydrograph and unsteady Hurricane Hazel hydrograph. Based on the comparative analysis and discussions with the TRCA, in order not to account for the effect of storage elements within the floodplain area and to generate a conservative floodplain, it was determined that the steady Hurricane Hazel input hydrograph was more appropriate to use for floodplain mapping than the unsteady flow hydrograph.

With assistance from the TRCA, two updated map sheets were prepared based on the flood depth maps calculated with the Mike Flood model. These updated map sheets will replace the existing Humber River floodplain Map Sheets 9, 8 and a portion of 7 (to just south of Lawrence Ave. W.). The floodline was created by digitizing the extent of the flood depth map prepared in ArcGIS. The flood depth map in ArcGIS was prepared using the Mike Flood results obtained from the 15-hr steady Hurricane Hazel hydrograph simulation converted and resampled into a 0.5 m x 0.5 m GIS raster surface. The contours used to prepare the base map were created using the LiDAR digital elevation surface by the TRCA. The updated Humber River engineered floodplain Map Sheets 8 and 9 are provided in **Appendix C**.

# 5.0 CHARACTERIZATION AND RISK ASSESSMENT OF THE FLOODPLAIN

The results of the Mike Flood model were used to characterize flooding and to assess the risk of flooding within the study area. The following sections define the criteria used for flood risk assessment and identify zones where high risk flooding occurs for the 350-yr design storm and the Regional storm (Hurricane Hazel). In addition, the return period storm at which the extent of flooding within the identified flood zones becomes significant is assessed.

## 5.1 Defining Flood Risk

Criteria provided in the *Technical Guide River and Stream Systems: Flooding Hazard Limit* prepared by the Ontario Ministry of Natural Resources (MNR) in 2002 along with the frequency of flooding are typically used in defining and assessing flood risk. Another consideration in assessing flood risk is the vulnerability of critical services and infrastructure that may exist within identified flood zones.

### *5.1.1 MNR Flood Risk Criteria*

As per the *Technical Guide River and Stream Systems: Flooding Hazard Limit* (MNR, 2002), the following criteria were used in defining the upper limits of safe access/egress:

- Maximum depth = 0.8 m;
- Maximum velocity = 1.7 m/s
- Maximum depth-velocity product = 0.37 $m^2/s$

Based on work completed recently in other SPA's, the TRCA has revised the flood risk categories and how they are calculated. The revised flood risk categories are divided into low, moderate and high risk and are defined as follows:

- **Low Risk** – Vehicular and pedestrian access/egress is available (depth < 0.30 m);
- **Moderate Risk** – Pedestrian access/egress ONLY available (depth-velocity product $\leq$ 0.37 $m^2/s$ with a maximum depth of 0.80 m and maximum velocity of 1.70 m/s); and,
- **High Risk** – Depth-velocity product > 0.37 $m^2/s$ OR depth > 0.80 m OR velocity > 1.70 m/s.

### *5.1.2 Flood Frequency*

In addition to the MNR flood risk criteria, flood frequency is also an important factor considered in assessing flood risk. Areas that may be exposed to high risk flooding only during the Regional storm, for example, would typically be considered less of a risk than areas exposed to high risk flooding during the 2-yr storm. In a subsequent section, the results of the 1D/2D integrated hydraulic model prepared for the study area are used to calculate the extent of high risk flooding within the identified flood zones for the 350-yr storm and the Regional storm (Hurricane Hazel).

### *5.1.3 Existing Vulnerable Critical Services and Infrastructure*

Another factor considered in characterizing and assessing flood risk is the vulnerability of critical services and infrastructure that may exist within identified flood zones. For example, an area exposed to high risk flooding that consists of isolated open space where there is no risk to the Public would be considered less significant than a fire station, hospital or power plant that provides emergency services or critical services to the Public. In completing the overall characterization of flooding and the assessment of flood risk within the study area, sites that consist of critical services (*e.g.* fire stations, hospitals), schools, vulnerable populations (*e.g.* health clinics or diagnostic medical services), critical infrastructure (*e.g.* power plants), etc. were obtained from the TRCA and included on the base mapping.

## 5.2 Key Identified High Risk Flood Zones

Using the MNR flood risk criteria identified above, key zones of high flood risk were identified within the study area. In identifying the key high risk flood zones, the location and potential impacts of the flooding was taken into consideration. For example, areas that exceed the MNR criteria for high risk flooding but do not impact existing development/transportation routes or areas where the extent of impact to development/transportation routes is minimal were not considered as key flood zones. Key flood zones were considered to be areas where extensive high risk flooding occurs within development areas or along transportation routes. A total of six (6) high risk flood zones were identified within the study area. The location of the key identified high risk flood zones is provided in **Figure 5.3**. The Regional and 350-yr storm flood risk level maps are provided in **Figure 5.1** and **Figure 5.2**, respectively.

### *5.2.1 Flood Zone 1*

Flood Zone 1 is located between Jane St. and Black Creek in the vicinity of Marlington Crescent. Development within this area consists of primarily residential. There is one school (Saint Gerard Majella Catholic School) that lies within this area of high risk flooding. A summary of the key areas impacted and the total area of high risk flooding within this flood zone is provided in **Table 5.1** for the 350-yr storm and the Regional storm. Based on an analysis of the hydraulic model results, the extent of flooding within this zone becomes significant for approximately the 350-yr return period event.

**Table 5.1: High Risk Flood Zone 1**

| Return Period | Key Areas Impacted by High Risk Flooding within the Flood Zone | Extent of High Risk Flooding within the Flood Zone |
|---|---|---|
| **350-yr** | Residential areas and local roads | 4.105 ha |
| **Regional Storm (Hazel)** | Residential areas, local roads and a school (Saint Gerard Majella Catholic School) | 6.594 ha |

### *5.2.2 Flood Zone 2*

Flood Zone 2 extends from Sheridan Mall west to Haymarket Road and lies between Black Creek and just south of Wilson Avenue. Development within this area consists of a combination of residential, commercial and industrial. There are two identified vulnerable population sites (Sheridan Optical Centre and Black Creek Community Health Centre) located in Sheridan Mall that lie within this area of high risk flooding. A summary of the key areas impacted and the total area of high risk flooding within this flood zone is provided in **Table 5.2** for the 350-yr storm and the Regional storm. Based on an analysis of the hydraulic model results, the extent of flooding within this zone becomes significant for approximately the 350-yr return period event.

**Table 5.2: High Risk Flood Zone 2**

| Return Period | Key Areas Impacted by High Risk Flooding within the Flood Zone | Extent of High Risk Flooding within the Flood Zone |
|---|---|---|
| **350-yr** | Residential areas, local roads and two identified vulnerable population sites (Black Creek Community Health Centre and Sheridan Optical Centre) | 2.613 ha |
| **Regional Storm (Hazel)** | Residential areas, local roads and two identified vulnerable population sites (Black Creek Community Health Centre and Sheridan Optical Centre) | 5.722 ha |

### *5.2.3 Flood Zone 3*

Flood Zone 3 is located along the Jane Street corridor north of Highway 401. Development within this area consists of primarily commercial with some residential. There is one school (St. John the Evangelist Catholic School) and an early learning centre (St. Philip Neri Learning) that lie within this area of high risk flooding as well as an identified vulnerable population site (Jane-Wilson Diagnostic SVC) and a fire station (#146). A summary of the key areas impacted and the total area of high risk flooding within this flood zone is provided in **Table 5.3** for the 350-yr storm and the Regional storm. Based on an analysis of the hydraulic model results, the extent of flooding within this zone becomes significant for approximately the 10-yr return period event.

**Table 5.3: High Risk Flood Zone 3**

| Return Period | Key Areas Impacted by High Risk Flooding within the Flood Zone | Extent of High Risk Flooding within the Flood Zone |
|---|---|---|
| **350-yr** | Residential areas, commercial areas, local roads, arterial roads (Jane St. and Wilson Ave.) and an identified vulnerable population site (Jane-Wilson Diagnostic SVC) | 11.613 ha |
| **Regional Storm (Hazel)** | Residential areas, commercial areas, local roads, arterial roads (Jane St. and Wilson Ave.), school (St. John the Evangelist Catholic School), and two identified vulnerable population sites (St. Philip Neri Early Learning and Jane-Wilson Diagnostic SVC) | 14.851 ha |

### *5.2.4 Flood Zone 4*

Flood Zone 4 is located west of Jane Street, north of Highway 401 and extends across Highway 401. Development within this area consists of primarily commercial. A summary of the key areas impacted and the total area of high risk flooding within this flood zone is provided in **Table 5.4** for the 350-yr storm and the Regional storm. Based on an analysis of the hydraulic model results, the extent of flooding within this zone becomes significant for approximately the 350-yr return period event.

**Table 5.4: High Risk Flood Zone 4**

| Return Period | Key Areas Impacted by High Risk Flooding within the Flood Zone | Extent of High Risk Flooding within the Flood Zone |
|---|---|---|
| **350-yr** | Commercial areas and local roads | 1.040 ha |
| **Regional Storm (Hazel)** | Commercial areas, local roads and Highway 401 | 4.716 ha |

### *5.2.5 Flood Zone 5*

Flood Zone 5 is located along the Jane Street corridor between Highway 400 and Highway 401. Development within this area consists of a mix of residential, commercial and industrial. There is one identified vulnerable population site (Toronto Child Care Centre) that lies within this area of high risk flooding. A summary of the key areas impacted and the total area of high risk flooding within this flood zone is provided in **Table 5.5** for the 350-yr storm and the Regional storm. Based on an analysis of the hydraulic model results, the extent of flooding within this zone becomes significant for approximately the 10-yr return period event.

**Table 5.5: High Risk Flood Zone 5**

| Return Period | Key Areas Impacted by High Risk Flooding within the Flood Zone | Extent of High Risk Flooding within the Flood Zone |
|---|---|---|
| **350-yr** | Residential areas, commercial areas, industrial areas, local roads, collector roads, arterial roads (Jane St.), access ramps to/from Hwy 401 and an identified vulnerable population site (Toronto Child Care Centre) | 18.173 ha |
| **Regional Storm (Hazel)** | Residential areas, commercial areas, industrial areas, local roads, collector roads, arterial roads (Jane St.), access ramps to/from Hwy 401 and an identified vulnerable population site (Toronto Child Care Centre) | 24.461 ha |

### *5.2.6 Flood Zone 6*

Flood Zone 6 is located between Maple Leaf Drive and Lawrence Avenue within the study area along Black Creek Drive and includes a portion of Lawrence Avenue west of Black Creek Drive. Development within this area consists of primarily a transportation corridor with some residential. A summary of the key areas impacted and the total area of high risk flooding within this flood zone is provided in **Table 5.6** for the 350-yr storm and the Regional storm. Based on an analysis of the hydraulic model results, the extent of flooding within this zone becomes significant for approximately the 2-yr return period storm for Lawrence Avenue and the 25-yr to 50-yr return period storm for the spill to Black Creek north of Maple Leaf Drive.

**Table 5.6: High Risk Flood Zone 6**

| Return Period | Key Areas Impacted by High Risk Flooding within the Flood Zone | Extent of High Risk Flooding within the Flood Zone |
|---|---|---|
| **350-yr** | Arterial roads (Black Creek Dr. and Lawrence Ave.) | 5,278 ha |
| **Regional Storm (Hazel)** | Arterial roads (Black Creek Dr. and Lawrence Ave.) | 5.738 ha |

# 6.0 IDENTIFICATION OF HYDRAULIC CONSTRAINTS

There are a number of hydraulic constraints that contribute to flooding within the study area. In the sections that follow, these hydraulic constraints are identified, the key causes of flooding within identified flood zones are investigated and possible mitigation options are recommended for further detailed study.

## 6.1 Identifying Hydraulic Constraints

The hydraulic constraints encountered within the study area that have been identified as contributing to flooding are described as follows:

- Undersized hydraulic structures (*e.g.* culverts, bridges with poor conveyance capacity);
- Undersized channels (*e.g.* watercourse with poor conveyance capacity); and
- Constraints due to topography (*e.g.* spill points, low lying or flat, poorly drained topography).

In order to identify possible mitigation options for flooded areas, it is important to have a good understanding of the specific constraints that contribute to the flooding. As such, areas within the identified flood zones were analyzed using flow direction mapping prepared over a series of time intervals to understand the flooding process within each flood zone. A location plan indicating areas where a detailed analysis of the flooding process was completed within each flood zone is provided in **Figure 6.0**. Regional flood flow and flooding process results are provided in **Figures 6.1**, **6.2**, **6.2.1** to **6.2.4**, **6.3** to **6.5**, **6.5.1** and **6.6** to **6.9**.

## 6.2 Key Causes of Flooding for Identified Flood Zones

Based on an analysis of the flood flow and flooding process results described above, the key causes of flooding within each flood zone were identified. A description of the hydraulic constraints identified as contributing to flooding within the identified flood zones in the study area is provided below.

### *6.2.1 Flood Zone 1*

Flood Zone 1 is located in the Marlington Crescent area. Through an analysis of the flood flow (**Figure 6.1**) and flooding process (**Figure 6.2**) mapping, a low point was identified where spill from Black Creek is conveyed from the east bank of the watercourse in the vicinity of Heavitree Drive through the Saint Gerard Majella Catholic School property to the Marlington Crescent residential area. Flooding ponds in this area to a point at which it then spills south through an existing apartment complex back to Black Creek. This spill point along Black Creek in the vicinity of the school south of Heavitree Drive has been assessed as the key identified hydraulic constraint contributing to flooding within Flood Zone 1.

### *6.2.2 Flood Zone 2*

Flood Zone 2 is located in the Sheridan Mall and Kirby Road area. Through an analysis of the flood flow (**Figure 6.2.1**) and flooding process (**Figure 6.2.2**) mapping, a low point in the vicinity of the existing pedestrian bridge north of the Sheridan Mall was identified where spill (primary spill) from Black Creek is conveyed from the south bank of the watercourse around the mall building on the east and west sides. The spill to the east of the mall converges with flow exceeding the capacity of the watercourse further downstream along Jane Street. More notably, the spill that passes to the west of the mall contributes to extensive flooding further south and west of the mall. In addition, there is a secondary spill from Black Creek off Loney Avenue that contributes further to flooding in this area at a slightly later time. It is noted that the flow from these two identified spill points contribute to most of the flooding within Flood Zone 2 and are considered to be the key identified hydraulic constraints within this flood zone.

### *6.2.3 Flood Zone 3*

Flood Zone 3 is located along the Jane Street corridor north of Highway 401. Through an analysis of the flood flow (**Figure 6.2.3**) and flooding process (**Figure 6.2.4**) mapping, it was identified that flooding in this area is mostly the result of inadequate conveyance capacity of the existing bridges and the watercourse. In addition, once flow exceeds the capacity of the watercourse, the flooding spills and extends for a distance to the east and west of Jane Street due to the relatively flat and poorly drained topography. The hydraulic structures that contribute to flooding within this area as they overtop include the bridge at the entrance to the Sheridan Mall off Jane St., the bridge under Jane St., the bridge under Wilson Ave., and the bridge at Downsview Avenue. The culvert under Highway 401 is undersized, however, overtopping does not occur at the location of the culvert but rather further west at a low point where flow overtops Highway 401. Due to the topography, flood flows in this area are generally conveyed south along the Jane St. corridor to the culvert located at Highway 401 where ponding occurs. These identified hydraulic structures and conveyance channel have inadequate capacity to convey the Regional storm flow and, along with low points along the watercourse and poorly drained topography, are the key identified hydraulic constraints within Flood Zone 3.

### *6.2.4 Flood Zone 4*

Flood Zone 4 is located along the north side of Highway 401 west of Jane St. and includes Highway 401. Through an analysis of the flood flow (**Figure 6.3**) and flooding process (**Figure 6.4**) a hydraulic structure constraint was identified at Highway 401 that causes floodwaters to pond north of Highway 401 to a point at which flow is then conveyed to the west to the Jane St. underpass and subsequently further west where it spills across Highway 401 to the south. Spill from Flood Zone 3 along Jane St. also contributes to flooding in this area. It is noted that later in the flooding process, additional spill from Flood Zone 2 contributes further to the flooding in this area. The existing hydraulic structure under Highway 401 including the conveyance channel are inadequate to convey the Regional flow and are the key identified hydraulic constraints within Flood Zone 4 along with contributing spills from Flood Zone 3 and Flood Zone 2.

### *6.2.5 Flood Zone 5*

Flood Zone 5 is located along the Jane Street corridor south of Highway 401. Through an analysis of the flood flow (**Figure 6.5**) and flooding process (**Figure 6.5.1**) mapping, it was identified that flooding in this area is mostly the result of inadequate conveyance capacity of the existing bridges and the watercourse. In addition, once flow exceeds the capacity of the watercourse, the flooding spills and extends for a considerable distance to the east and west of Jane Street due to the relatively flat and poorly drained topography. The hydraulic structures that contribute to flooding within this area as they overtop include the bridge under Jane St., the bridge under Gordon MacKay Rd., the bridge under the ramp from Jane St. South to Hwy 400 North, and a second bridge under Jane St. It is noted that the spill from Flood Zone 3 through the Jane St. underpass and the spill from Flood Zone 4 across Highway 401 contribute to the overall flooding in Flood Zone 5 at a later time. Due to the topography, flood flows in this area are generally conveyed south along the Jane St. corridor to the bridge under Black Creek Drive. These identified hydraulic structures and conveyance channel have inadequate capacity to convey the Regional storm flow and, along with low points along the watercourse and poorly drained topography, are the key identified hydraulic constraints within Flood Zone 5 along with contributing spills from Flood Zone 3 and Flood Zone 4.

### *6.2.6 Flood Zone 6*

Flood Zone 6 is located along Black Creek Drive between Maple Leaf Drive and Lawrence Avenue including a portion of Lawrence Avenue west of Black Creek Drive. Through an analysis of the flood flow (**Figure 6.6**) and flooding process (**Figure 6.7**) mapping at Maple Leaf Drive, flooding at this location results from flow backing up behind the Maple Leaf Dr. bridge that has inadequate capacity to convey the Regional storm and ponds to a low point where it then spills to Black Creek Drive and continues south along Black Creek Drive. Through an analysis of the flood flow (**Figure 6.8**) and flooding process (**Figure 6.9**) mapping at Lawrence Avenue, flooding at this location results from flow backing up behind the Lawrence Avenue bridge that has inadequate capacity to convey the Regional storm and ponds to a low point on Lawrence Ave. where it then spills to Black Creek Drive and continues south along Black Creek Drive. It is noted that spill from the Maple Leaf Drive area converges at a later time with the spill from Lawrence Avenue and both spill areas contribute to the overall flooding of Black Creek Drive. The two identified hydraulic structures that have inadequate capacity to convey the Regional storm flow along with the two identified spill points are the key identified hydraulic constraints within Flood Zone 6.

## 6.3 Summary and Identification of Possible Mitigation Options

Based on an analysis of the completed hydraulic modelling using Mike Flood, a number of hydraulic constraints were identified that contribute to flooding in the identified flood zones within the study area. As noted in the previous section, the hydraulic constraints that contribute to flooding include undersized hydraulic structures, undersized channels and constraints due to topography. The return period at which the identified hydraulic constraints contribute to flooding varies. **Figure 6.10** provides some insight into the frequency at which various locations within the study area become inundated. Through the preliminary analysis of the flooding process, possible mitigation options for further consideration and detailed hydraulic modelling were identified. Possible mitigation options included the following:

- Bridge capacity conveyance improvements;
- Channel conveyance capacity improvements;
- Spill containment; and
- Overland flow improvements.

A summary of the investigations of hydraulic constraints for the identified flood zones, frequency at which flooding occurs and possible mitigation options for further consideration and detailed analysis is provided in **Table 6.1**.

**Table 6.1: Summary of Identified Hydraulic Constraints, Flooding Frequency and Possible Mitigation Options**

| Flood Zone | Hydraulic Constraint (s) | Frequency at which Flooding Occurs | Possible Mitigation Options for Further Detailed Analysis |
|---|---|---|---|
| 1 | • Spill point at Saint Gerard Majella Catholic School south of Heavitree Dr. | • 350-yr | • Spill containment |
| 2 | • Spill point in vicinity of pedestrian bridge north of Sheridan Mall | • 350-yr | • Spill containment |
| | • Spill point in vicinity of Loney Ave. | • 350-yr | |
| 3 | • Jane St. bridge overtops | • 5-yr to 10-yr | • Bridge/culvert conveyance capacity improvements<br>• Channel conveyance capacity improvements<br>• Spill containment<br>• Overland flow improvements |
| | • Spill points along Black Creek | • 10-yr | |
| | • Sheridan Mall bridge overtops | • 10-yr to 25-yr | |
| | • Wilson Ave. bridge overtops | • 25-yr | |
| | • Downsview Ave. bridge overtops | • 10-yr | |
| | • Hwy 401 culvert undersized resulting in ponding | • 5-yr to 10-yr | |
| 4 | • Hwy 401 culvert undersized resulting in ponding and spill to the Jane St. underpass and across Hwy 401 | • 10-yr for spill to Jane and 350-yr for spill across Hwy 401 | • Culvert conveyance capacity improvements<br>• Channel conveyance capacity improvements |
| 5 | • Jane St. bridge south of Hwy 401 overtops | • 10-yr | • Bridge conveyance capacity |

| | | | |
|---|---|---|---|
| | • Jane St. bridge north of Hwy 400 overtops | • 5-yr | improvements<br>• Channel conveyance capacity improvements<br>• Spill containment<br>• Overland flow improvements |
| | • Gordon Mackay Rd. bridge overtops | • 10-yr | |
| | • Spill points along Black Creek | • 5-yr to 10-yr | |
| 6 | • Spill point at Black Cr. Dr. near Maple Leaf Dr. | • 25-yr | • Bridge conveyance capacity improvements<br>• Channel conveyance capacity improvements<br>• Spill containment |
| | • Maple Leaf Dr. bridge overtops | • 2-yr to 5-yr but spills to west of bridge at 2-yr | |
| | • Spill points at Lawrence Ave. and Black Creek Dr. | • 2-yr to 5-yr | |
| | • Lawrence Ave. bridge overtops | • 50-yr but spills to east of bridge at 2-yr | |

## 7.0 SUMMARY AND CONCLUSIONS

Under the direction of the Toronto and Region Conservation Authority, Valdor Engineering has completed the *Floodplain Mapping in Jane and Wilson Special Policy Area, Black Creek Study*. The key findings and results of the study are summarized as follows:

1. Using updated digital elevation mapping (DEM) derived from recently obtained LiDAR data supplemented with the completed topographic survey for channel sections and hydraulic structures, updated land use data, and updated flow data, an integrated 1D/2D hydraulic model was prepared using Mike Flood. The results of the Mike Flood model were found to be similar to the current approved extents of flooding based on HEC-RAS in many areas, however, the flooding in flat areas was generally less extensive and spill areas were much better defined.
2. The Mike Flood model was validated using water level data from the stream gauge operated by the TRCA on Black Creek located at Downsview Avenue. The Mike Flood model was found to agree well with the gauged water level data.
3. A comparison of the Mike Flood model with the existing HEC-RAS model was completed. The results of the Mike Flood model compared well in some areas with the existing HEC-RAS model but not in all areas. It was noted that HEC-RAS and Mike Flood are very different hydraulic models. Upon further inspection, it was found that the use of ineffective flow areas in the existing HEC-RAS model was not appropriate in many areas. A revised HEC-RAS model was prepared using more appropriate ineffective flow areas and better defined Manning's roughness values. The Mike Flood model was found to agree reasonably well with the revised existing HEC-RAS model. It is noted that the comparison with HEC-RAS was completed only with the Mike 11 (1D) channel profile for a consistent comparison and that the Mike 21 (2D) results could be different.
4. The results of the Mike Flood model using the steady Regional flow hydrographs provided by the TRCA were used to delineate the Regional floodplain. The floodplain calculated using Mike Flood was used to update the engineered floodplain map sheets through the study area (Humber River Map Sheets 8 and 9).
5. A characterization and risk assessment of the floodplain was completed to identify low, moderate and high risk areas within the floodplain and to identify high risk flood zones. A total of 6 high risk flood zones were identified. The extent of high risk flooding within each flood zone was calculated for the 350-yr return period design storm and the Regional storm (Hurricane Hazel).
6. An analysis of flood flows and the flooding process was undertaken to determine the key hydraulic constraints contributing to the flooding in each of the identified flood zones. The identified hydraulic constraints included undersized hydraulic structures (*e.g.* culverts, bridges with poor conveyance capacity), undersized channels (*e.g.* watercourse with poor conveyance capacity) and constraints due to topography (*e.g.* spill points, low lying or flat, poorly drained topography). The frequency at which flooding occurs for the identified hydraulic constraints was determined using the results of the Mike Flood model.
7. Possible mitigation options were recommended for future consideration and detailed analysis for the key hydraulic constraints within each of the 6 flood zones. Recommended mitigation options included bridge or culvert conveyance capacity improvements, channel conveyance capacity improvements, spill containment and overland flow improvements.

# 8.0 RECOMMENDATIONS

The following summarizes the report recommendations:

1. The revised Humber River Floodplain Map Sheets 8 and 9 based on the 1D/2D integrated hydraulic model (Mike Flood) should be used to replace the existing floodplain map sheets completed previously for the study area.
2. A detailed 1D/2D integrated hydraulic study should be undertaken to confirm and assess the possible options identified in this report (see **Table 8.1**) to improve the current flooding conditions within the study area. The study should include systematic investigations to confirm and assess the governing factors for the possible mitigation options to enable the determination of the best single mitigation option or combination of mitigation options within each identified flood zone and for the full extent of flooding within the study area as a whole. Investigations should begin by improving the conveyance capacity of identified culverts, then followed by improving the conveyance capacity of the channel, then followed by improvements regarding spill containment and finally by including overland flow improvements. In addition, the subsequent study should include preliminary cost estimates and an evaluation and prioritization of mitigation options within the identified high risk flood zones including possible staging options. Financial mechanisms should be investigated and reviewed that would enable the implementation of the recommended flood mitigation options.
3. In moving forward, it is recommended that the Mike Flood model should be used to assess potential off-site flood impacts from proposed infrastructure and development projects.

**Table 8.1: Recommended Possible Mitigation Options for Further Investigation and Detailed Analysis**

| Flood Zone | Hydraulic Constraint (s) | Possible Mitigation Options for Further Detailed Analysis |
|---|---|---|
| 1 | • Spill point at Saint Gerard Majella Catholic School south of Heavitree Dr. | • Spill containment |
| 2 | • Spill point in vicinity of pedestrian bridge north of Sheridan Mall<br>• Spill point in vicinity of Loney Ave. | • Spill containment |
| 3 | • Jane St. bridge overtops<br>• Spill points along Black Creek<br>• Sheridan Mall bridge overtops<br>• Wilson Ave. bridge overtops<br>• Downsview Ave. bridge overtops<br>• Hwy 401 culvert undersized resulting in ponding | • Bridge/culvert conveyance capacity improvements<br>• Channel conveyance capacity improvements<br>• Spill containment<br>• Overland flow improvements |
| 4 | • Hwy 401 culvert undersized resulting in ponding and spill to the Jane St. underpass and across Hwy 401 | • Culvert conveyance capacity improvements<br>• Channel conveyance capacity improvements |
| 5 | • Jane St. bridge south of Hwy 401 overtops<br>• Jane St. bridge north of Hwy 400 overtops<br>• Gordon Mackay Rd. bridge overtops<br>• Spill points along Black Creek | • Bridge conveyance capacity improvements<br>• Channel conveyance capacity improvements<br>• Spill containment<br>• Overland flow improvements |
| 6 | • Spill point at Black Cr. Dr. near Maple Leaf Dr.<br>• Maple Leaf Dr. bridge overtops<br>• Spill points at Lawrence Ave. and Black Creek Dr.<br>• Lawrence Ave. bridge overtops | • Bridge conveyance capacity improvements<br>• Channel conveyance capacity improvements<br>• Spill containment |

# 9.0 REFERENCES


- Aquafor Beech, *Humber River Watershed Hydrology Update*, November 2002.
- Civica, *Humber River Hydrology Update*, June 2015).
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- J.F. Sabourin, *Humber River Floodplain Map Sheets 7, 8 and 9*, 2005.
- TRCA, HEC-RAS Model for the Humber River, February 2011.


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Respectfully Submitted,

**VALDOR ENGINEERING INC.**

LICENSED PROFESSIONAL ENGINEER
W. L. P. COFFEY
PROVINCE OF ONTARIO

**Abdul Baten**, M.Sc., E.I.T.
Water Resources Analyst

**Bill Coffey**, M.Sc., P.Eng.
Head of Water Resources

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