



Final Report
Town of New Tecumseth
Drainage Master Plan Phase 1 - Flood Risk Assessment

Prepared for:
The Corporation of the Town of New Tecumseth

Prepared by:
Matrix Solutions Inc.

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Guelph, Ontario

Unit 7B, 650 Woodlawn Rd. W
Guelph, ON, Canada N1K 1B8
T 519.772.3777 F 226.314.1908
www.matrix-solutions.com

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Kelly Molnar, P.Eng.
Water Resources Engineer



reviewed by
Karen Hofbauer, M.A.Sc., P.Eng.
Senior Water Resources Engineer

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Contributors

Name	Job Title	Role
Kelly Molnar, P.Eng.	Water Resource Engineer	Primary author
Karen Hofbauer, M.A.Sc., P.Eng.	Senior Water Resources Engineer	Senior reviewer
Tim Martin, E.I.T.	Water Resources Engineer-in-training	Two-dimensional hydraulic model developments
Ziyang Zhang, M.Sc., E.I.T.	Water Resources Engineer-in-training	Flow input preparation and two-dimensional hydraulic model development
Peter Bishop, M.Eng., E.I.T.	Water Resources Engineer-in-training	Flood damage database development and vulnerability analysis
Natalie Burrows, M.A.Sc., P.Eng.	Water Resources Engineer	Two-dimensional hydraulic modelling support

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Abbreviations and Acronyms

1D	one-dimensional
2D	two-dimensional
AAD	average annual damages
AMC	antecedent moisture conditions
CPR	Canadian Pacific Railway
DEM	digital elevation model
DMP	Drainage Master Plan
EA	Environmental Assessment
HEC-RAS	Hydrologic Engineering Center River Analysis System
IDF	intensity-duration-frequency
LiDAR	light detection and ranging
LIO	Land Information Ontario
LSRCA	Lake Simcoe Region Conservation Authority
MNRF	Ministry of Natural Resources and Forestry (previously the Ministry of Natural Resources)
MTO	Ministry of Transportation Ontario
NVCA	Nottawasaga Valley Conservation Authority
PHB	Perron, Hudon, Bélanger Inc.
SCOOP	South Central Ontario Orthophotography Project
SCS	Soil Conservation Service
SSR	South Simcoe Railway
SWM	stormwater management

Glossary of Common Terms

Areal reduction factor: A reduction factor applied to the Hurricane Hazel and the Timmins Storm rainfall amounts when simulating the Regional event in a hydrologic model for large watersheds (areas greater than 25 km²). The areal reduction factor is based on the equivalent circular area of the watershed using the longest flow path length as the diameter (MNR 2002). It is used to account for the fact that the rainfall amounts used for simulating the historic Hurricane Hazel and Timmins events were recorded at specific locations covering a known area.

Attenuation: A term used to describe the reduction in peak flows that occurs when water slows down to fill storage areas, whether natural or constructed, such as lakes, wetlands, topographic depressions, or stormwater management ponds; attenuation due to storage can also occur behind bridges/culverts and railway or roadway embankments.

Average annual damages (AAD): The sum of expected damage costs over a given period divided by the length of the time period. AAD is used to account for the fact that over a long period of time the smaller, more frequent events may cause more damage than large, infrequent events.

Boundary condition: A value or set of values applied at the upstream and/or downstream ends of a hydraulic model to define flow conditions at the boundary.

Breakline: linear features used in the generation of a topographic surface to describe distinct changes in topographic features such as a ridge line (high point), edge of pavement, toe of slope, etc.

Catchment: The area of land that collects surface water and drains to a point of interest such as a catch basin, inlet, or water body as defined by the slope of the terrain. Also referred to as drainage area.

Embankment: a bank of earth or stone constructed above the natural ground surface to carry a road or railway, or to prevent water from spreading beyond desired limits.

Erosion: wearing away of soil caused by moving water.

Hydrograph: A graphical representation of flow over a specified time period.

Hyetograph: A graphical representation of rainfall intensity over a specified time interval.

Level of service: The largest storm event conveyed by the minor system before exceeding its design capacity. This term is often used to describe the relative hydraulic performance of an existing drainage system.

Model domain: The area covered by a hydrologic or hydraulic model.

Obvert: The highest point of the internal surface of a culvert. Referred to as soffit for bridges.

Overtopping: In the context of riverine modelling, overtopping occurs when the water elevation upstream of a bridge, culvert or other crossing exceeds the low point in the road (including railways, trails, etc.) and spills onto or over the road surface.

Quasi-steady state: A method of replicating steady state flow conditions in a model that requires unsteady flow input for the purpose of preventing initial condition model instabilities. Quasi-steady state hydrographs are prepared by gradually increasing flow from zero to the peak and holding constant for the remainder of the simulation.

Riverine flooding: Occurs when the water levels of rivers, streams, and creeks rise and overflow their banks, spilling onto adjacent areas.

Scour: erosion of a riverbed caused by streamflow.

Soffit: The highest point of the underside or bottom surface of a bridge deck. Referred to as obvert for culverts.

Spill: A spill in the context of floodplain delineation is an area where flooding is not physically contained in the channel corridor or floodplain and therefore flows into adjacent watersheds or subwatersheds. Spills can occur naturally (i.e., low channel banks or poorly defined floodplains) or as a result of physical barriers that restrict flow and raise upstream water elevations (i.e., bridge/culvert or channel restrictions, berms, or other barriers). An understanding of spill extents, depths, and flow rates are required to accurately define flood risk for all receiving watercourses.

Steady state: Refers to a condition where flow rates remain fixed over time. Steady state inflows are typically used in 1D HEC-RAS applications which uses the standard step method to solve steady gradually varied flow conditions.

Surcharge: In the context of riverine modelling, surcharge is defined as the water elevation exceeds the obvert/soffit of culvert and bridge crossings. In the context of urban drainage system modelling, surcharge refers to situations where the computed hydraulic grade line (water elevation) is aboveground elevation.

Unsteady state: Flow conditions that change over time and typically consists of a hydrograph with a rising limb, peak flow, and falling limb.

Urban drainage system: The network of minor and major system infrastructure that collects and conveys stormwater runoff from urban developed areas. The urban drainage system design and maintenance is typically the responsibility of municipalities.

Urban flooding: Occurs when the capacity of urban drainage infrastructure (i.e., sewers, road rights-of-way, major system flow paths, etc.) is exceeded leading to flooding of streets, basements, etc.

Watercourse: A natural well-defined depression in the ground in which a flow of water regularly or continuously occurs.

Table of Contents

1	Introduction	1
1.1	Study Area Overview	1
1.2	Study Scope and Approach	3
2	Background Review	3
2.1	Data Collection	3
2.2	Light Detection and Ranging Data Collection	4
2.3	Previously Completed Studies	5
2.3.1	Watershed Hydrology Study	5
2.3.2	Lake Simcoe Region Conservation Authority Hydrology Report	5
2.3.3	Lake Simcoe Region Conservation Authority Hydraulics Report	6
2.3.4	Stormwater Management Reports	6
3	Culvert and Bridge Data	7
4	Hydrology Review	13
4.1	Lake Simcoe Region Conservation Authority Flow Data.....	13
4.2	Nottawasaga Valley Conservation Authority Flow Data	14
4.2.1	Revisions to Nottawasaga Valley Conservation Authority Flows ..	14
4.2.2	Design Event Flows	16
4.2.3	Combined Flow Data from Lake Simcoe Region Conservation Authority and Nottawasaga Valley Conservation Authority	18
4.3	Stormwater Management.....	18
4.4	Quasi-steady Hydrographs	19
4.4.1	Flow Corrections for Quasi-steady Modelling.....	21
5	Hydraulic Model Development	25
5.1	Two-dimensional Hydraulic Modelling Concepts	25
5.2	Mesh Development.....	26
5.2.1	Mesh Resolution	26
5.2.2	Breaklines	27
5.2.3	Obstructions	29
5.3	Bridges and Culverts.....	31
5.3.1	Detailed Bridge and Culvert Approach	33
5.3.2	Cut Bridge and Culvert Approach.....	34
5.3.3	Block Bridge and Culvert Approach	34
5.3.4	Future Recommendations for Cut and Blocked Crossings.....	35
5.4	Boundary Conditions	35
5.4.1	Beeton Flats Boundary Condition.....	36
6	Flood Risk Assessment	37

6.1	Understanding Two-dimensional HEC-RAS Results	39
6.1.1	Modelling Limitation at Treetops Subdivision	39
6.2	Flood Characterization.....	40
6.2.1	Summary of Existing Condition Results	40
6.2.2	Alliston	40
6.2.3	Beeton.....	42
6.2.4	Hydraulic Constraints	46
6.3	Spill Assessment	52
6.3.1	Spill from Upper Nottawasaga River to Boyne River Subwatershed	53
6.3.2	Spill from Beeton Creek to Bailey Creek Subwatershed	55
6.3.3	Spill from Baker Drain to Beeton Creek Subwatershed.....	56
7	Sensitivity Analysis	57
7.1	Climate Change	58
7.2	Future Conditions	61
7.2.1	Future Development at Beeton	62
7.2.2	Upsized Culvert on 20 th Sideroad at 5 th Line.....	64
7.2.3	Potential New Culvert on Sir Frederick Banting Road.....	65
8	Vulnerability Analysis.....	67
8.1	Buildings	69
8.1.1	Flood Damage Database	69
8.1.2	Summary of Required Attributes	70
8.1.3	Flood Depths.....	74
8.1.4	Average Annual Damages	79
8.2	Land Parcels.....	80
8.2.1	Average Annual Damages - Land Parcels	85
8.3	Roads and Rails	87
8.3.1	Average Annual Damages - Roads and Rails.....	90
9	Flood Risk Areas	91
9.1	Culvert Capacity	92
9.1.1	Sir Frederick Banting Road	92
9.1.2	Industrial Parkway and Canadian Pacific Railway.....	94
9.1.3	14 th Line at 20 th Sideroad	95
9.1.4	5 th Line at 20 th Sideroad	96
9.1.5	10 th Line and 10 th Sideroad	97
9.1.6	Tottenham Road at 10 th Line.....	98
9.1.7	7 th Line at Canadian Pacific Railway and South Simcoe Railway .	99
9.1.8	Blocked Culverts	100

9.2	Channel Capacity	101
9.2.1	Alliston Wastewater Treatment Plant	102
9.2.2	Honda Manufacturing Plant.....	103
9.2.3	9 th Line at 15 th Sideroad	104
9.2.4	10 th Line East of Tottenham Road.....	105
9.2.5	6 th Line East of Tottenham Road.....	106
9.2.6	Schomberg River North of Highway 9	107
9.3	Previously Identified At-risk Areas	108
9.3.1	Alliston - Spring Creek	108
9.3.2	Beeton - Hendrie Drain	109
9.3.3	Beeton Flats.....	110
9.3.4	Bailey Creek Swamp.....	111
10	Mitigation Planning.....	112
11	Next Steps	120
11.1	Hydrology Update	121
11.2	Two-dimensional HEC-RAS Model Updates.....	121
11.3	Detailed Flood Mitigation Assessments	122
11.3.1	Tottenham Dam Failure - Hydraulic Review.....	123
11.4	Urban Drainage System Modelling	123
11.5	Climate Change Consideration	124
11.6	Coordination with Other Studies	125
12	Conclusion	125
13	References	126

List of Figures

Figure 1	Study Area	2
Figure 2	Screening of Crossings.....	9
Figure 2.1	Screening of Crossings – Alliston	10
Figure 2.2	Screening of Crossings – Beeton	11
Figure 2.3	Screening of Crossings – Tottenham.....	12
Figure 3	Comparison of Steady-state and Unsteady-state Model Inflow Requirements	22
Figure 4	Example Hydrograph Summation	22
Figure 5	Hydrologic Flow Nodes	24
Figure 6	Example of Various Mesh Resolution in the Two-dimensional HEC-RAS Model.....	27
Figure 7	Example of Breaklines in the Two-dimensional HEC-RAS Model.....	28

Figure 8	Example of Small Buildings Excluded from the Two-dimensional HEC-RAS Model	29
Figure 9	Example of Building Simplification for the Two-dimensional HEC-RAS Model	30
Figure 10	Example of Detailed Bridge	34
Figure 11	Existing Regional Event Flooding at Canadian Pacific Railway in Alliston	41
Figure 12	Existing 100-year Event Flooding on Hendrie Drain in Beeton	42
Figure 13	Existing Regional Event Model Results in Tottenham	43
Figure 14	Example of Berms along Baker Drain East Branch	46
Figure 15	Upper Nottawasaga River and Boyne River Subwatersheds (100-year Event)	54
Figure 16	Beeton Creek to Bailey Creek Subwatershed (Regional Event)	56
Figure 17	Baker Drain to Beeton Creek Spill (Regional Event).....	57
Figure 18	Conceptual Illustration of Climate Change Adjusted Rainfall and Resulting Runoff	59
Figure 19	Climate Change Adjusted Flow Example.....	61
Figure 20	Future Development at Beeton	63
Figure 21	Comparison of Existing and Future Condition Regional Event in Beeton	64
Figure 22	Comparison of Existing and Future Condition Regional Event on 20 th Sideroad at 5 th Line.....	65
Figure 23	Comparison of Existing and Future Condition 100-year Event on Sir Frederick Banting.....	66
Figure 24	Overview of Flood Damage Components	68
Figure 25	Schematic of Residential Building with Flooding above First-floor Elevation	76
Figure 26	Schematic of Residential Building with Flooding below First-floor Elevation	76
Figure 27	Schematic of Non-residential and Apartment Buildings with Flooding above First-floor Elevation	77
Figure 28	Direct Damage Costs (Buildings) Probability Distribution	80
Figure 29	Land Use	82
Figure 30	Land Parcel Inundation Frequency	86
Figure 31	Sir Frederick Banting Road Flood Risk Area (Regional Event).....	93
Figure 32	Industrial Parkway and Canadian Pacific Railway Flood Risk Area (Regional Event).....	94
Figure 33	14 th Line at 20 th Sideroad Flood Risk Area (Regional Event).....	95
Figure 34	5 th Line at 20 th Sideroad Flood Risk Area (Regional Event).....	96
Figure 35	10 th Line at 10 th Sideroad Flood Risk Area (Regional Event).....	97

Figure 36	Tottenham Road at 10 th Line Flood Risk Area (Regional Event)	98
Figure 37	7 th Line at Canadian Pacific Railway and South Simcoe Railway Flood Risk Area (5-year Event).....	100
Figure 38	Alliston Wastewater Treatment Plant Flood Risk Area (Regional Event)	102
Figure 39	Honda Manufacturing Plant Flood Risk Area (Regional Event)	103
Figure 40	9 th Line at 15 th Sideroad Flood Risk Area (Regional Event).....	104
Figure 41	10 th Line East of Tottenham Road Flood Risk Area (Regional Event) ...	105
Figure 42	6 th Line East of Tottenham Road Flood Risk Area (Regional Event)	106
Figure 43	Schomberg River North of Highway 9 Flood Risk Area (Regional Event)	107
Figure 44	Alliston - Spring Creek Flood Risk Area (Regional Event)	109
Figure 45	Beeton - Hendrie Drain Flood Risk Area (Regional Event)	110
Figure 46	Beeton Flats Flood Risk Area (Regional Event).....	111
Figure 47	Bailey Creek Swamp Flood Risk Area (Regional Event).....	112

List of Tables

Table 1	Summary of Crossing Inventory.....	7
Table 2	Nottawasaga Valley Conservation Authority Flow Anomalies	15
Table 3	Example Design Event Ratios for Nottawasaga Valley Conservation Authority Jurisdiction.....	17
Table 4	Manning’s Roughness Values	31
Table 5	Bridge and Culvert Modelling	32
Table 6	Regional Storm Volume Check	37
Table 7	Flood Risk Criteria	38
Table 8	Railway Crossings with Backwater	44
Table 9	Bridge and Culvert Capacity	47
Table 10	Summary of Detailed Bridge and Culvert Hydraulic Performance	51
Table 11	Summary of Spill Assessment	53
Table 12	Modelled Spill Rates from Upper Nottawasaga River to Boyne River Subwatershed.....	53
Table 13	Building Database Attributes.....	70
Table 14	Summary of Building Type and Class	71
Table 15	First Floor Elevation Estimates	73
Table 16	Number of Flood Vulnerable Buildings - Existing Condition.....	75
Table 17	Building Damage Costs	78
Table 18	Inundated Area per Total Land Use Type for Modelled Events	83
Table 19	Number of Flood Vulnerable Land Parcels by Flood Event	84
Table 20	Flood Risk at Roads.....	88

Table 21	Land Information Ontario Road Classes	89
Table 22	Flood Risk at Provincial Highways	89
Table 23	Flood Risk at Railways.....	90
Table 24	Average Annual Risk to Roads and Rails	91
Table 25	Flood Risk due to Culvert Capacity Limitations.....	92
Table 26	Flood Risk due to Channel Capacity Limitations.....	101
Table 27	Flood Risk at Previously Identified Locations.....	108
Table 28	Flood Risk Areas Mitigation Plan	115
Table 29	Recommendations for Ditching.....	120

Appendices

Appendix A	Record of Received Data
Appendix B	Drainage Master Plan - Phase 1 - LiDAR Survey Technical Report (PHB 2020)
Appendix C	Complete Inventory of Bridges and Culverts
Appendix D	Matrix Field Inventory Sheets
Appendix E	HEC-RAS Flow Input
Appendix F	Bridge and Culvert Data for Detailed Modelling Approach
Appendix G	Existing Condition Results
Appendix G1	Flood Risk Assessment Mapping
Appendix G2	Longitudinal Profile Plots
Appendix G3	Bridge and Culvert Capacity Details
Appendix H	Sensitivity Analysis Input and Results
Appendix H1	Climate Change Adjusted Flows
Appendix H2	Future Condition Flows at Beeton
Appendix H3	Future Condition Culvert Dimensions
Appendix H4	Future Condition Flood Risk Assessment Mapping
Appendix I	Building Classes and Depth-Damage Curves
Appendix J	Building Database Attributes
Appendix K	Building Database
Appendix L	Vulnerability Analysis Detailed Output
Appendix L1	Building Flood Damages (digital)
Appendix L2	Land Parcels Inundated Area
Appendix L3	Land Parcels Risk Level
Appendix L4	Building Flood Damages (digital)
Appendix L5	Public Infrastructure Vulnerability Maps (digital)

1 Introduction

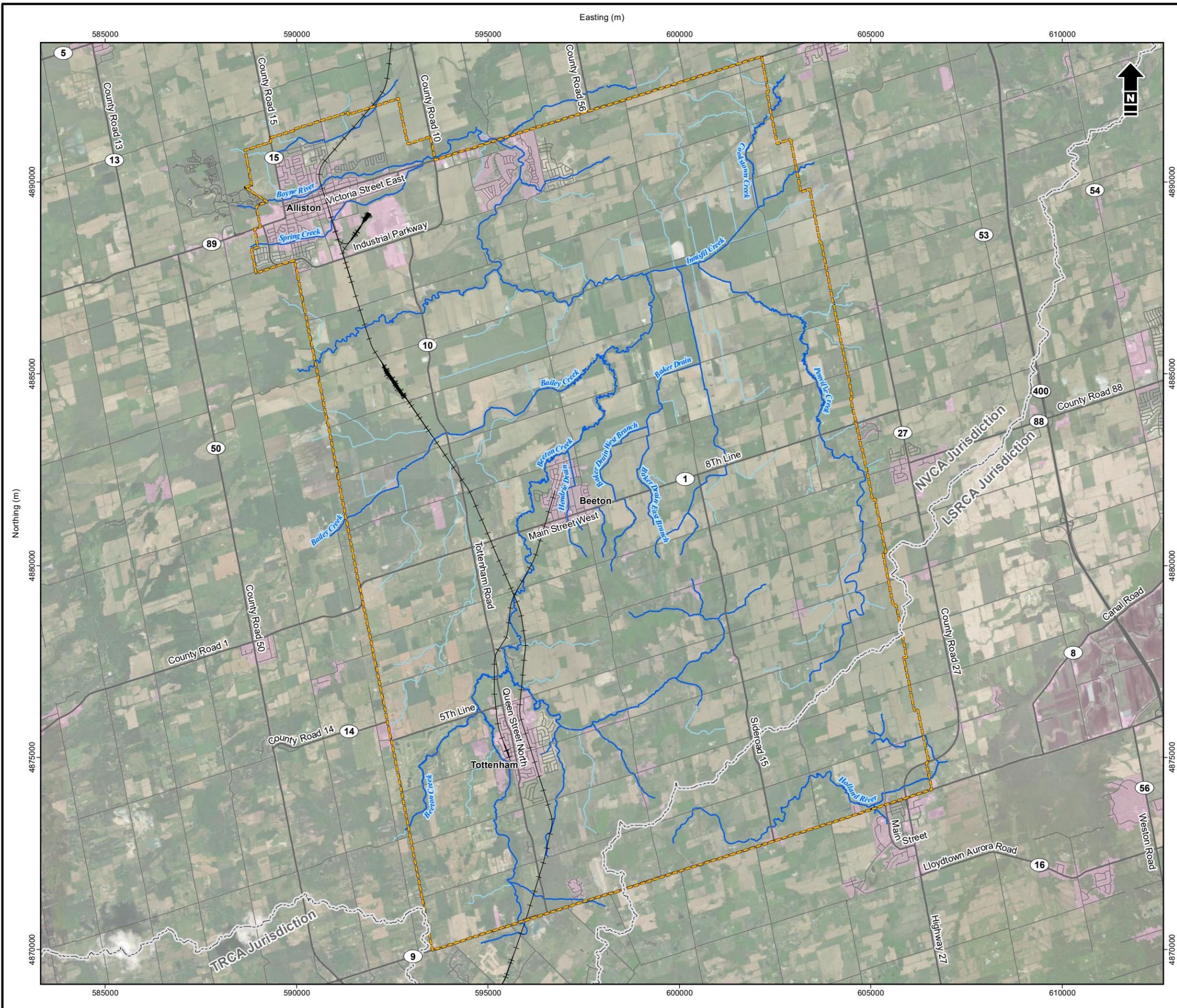
Matrix Solutions Inc. (Matrix) was retained by the Town of New Tecumseth (the Town) to complete Phase 1 of a Town-wide Drainage Master Plan (DMP). The June 23, 2017, major rainfall event caused portions of the community of Beeton to be severely flooded, particularly in the northeast quadrant. This storm was equivalent to a 100-year design storm and more than 100 mm of rainfall was recorded in less than 7 hours. The storm event caused flooding of homes and damage to Town infrastructure such as roads, road embankments, bridges, and culverts. Many residents had to be evacuated from their homes. In addition, significant agricultural losses were reported due to extensive field flooding.

Climate change is predicted to increase the frequency of severe storm events and therefore many municipalities are striving to gain a complete understanding of flood risk in their jurisdictions. The Town *Development Charges Background Study* (Hemson 2018) recommended that a Town-wide DMP be completed as part of the growth-related capital program. With approximately 40,000 residents and forecasted growth, the Town intends to develop a DMP for its jurisdiction to plan and implement future drainage improvements in a comprehensive fashion. The Town is undertaking the DMP in multiple phases. Phase 1, the focus of the current project, is partly funded by the National Disaster Mitigation Program.

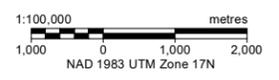
1.1 Study Area Overview

The Town is a thriving municipality with a mix of rural, agricultural, and natural land uses and three settlement areas: Alliston, Beeton, and Tottenham. It is located west of Highway 400 in the County of Simcoe, Ontario and is serviced by highways 9 and 89, and county roads 10 and 50.

The Town's jurisdiction includes many kilometers of open channels, the majority of which are within the Nottawasaga River watershed under the jurisdiction of the Nottawasaga Valley Conservation Authority (NVCA). A small portion of the southeast corner of the study area is within the Holland River watershed in the Lake Simcoe Region Conservation Authority (LSRCA) jurisdiction. The major rivers within the study area include Boyne River, Bailey Creek, Beeton Creek, Penville Creek, Innisfil Creek, Sheldon Creek, and Holland River. The study area is shown on Figure 1.



- Study Area and Municipal Boundary
- Conservation Authority boundary
- Built-up Area
- Railway
- Highway
- Road
- Watercourse**
 - Main Branch
 - Tributary



Reference: Contains information licensed under the Open Government Licence - Ontario. Imagery Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community



The Corporation of the Town of New Tecumseth
Drainage Master Plan Phase 1

Study Area

Date:	July 2020	Project:	28997	Submitter:	K. Molnar	Reviewer:	K. Hofbauer
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1.2 Study Scope and Approach

Phase 1 of the DMP focuses on the major overland system and watercourses and includes the following scope of work:

- collecting light detection and ranging (LiDAR) survey data to generate a detailed digital elevation model (DEM) of current topography within the Town's jurisdiction plus a 500 m buffer
- obtaining and reviewing available background data including previously completed models, reports, drawings, photographs, and GIS data
- developing a two-dimensional (2D) HEC-RAS hydraulic model to establish a comprehensive understanding of existing drainage deficiencies within the Town through flood risk characterization
- completing a sensitivity analysis to review the resiliency of the Town's drainage network to climate change and future developments
- preparing a flood damage assessment database to identify flood vulnerable areas (land parcels and roads) and estimate associated damage costs for flood-prone buildings within the study area
- providing recommendations for subsequent phases of the DMP and future studies

The objective of this study was to identify existing areas at risk to flooding within the Town's jurisdiction for the purpose of identifying opportunities for future flood mitigation assessments. To achieve this objective, a 2D HEC-RAS hydraulic model was developed and was used to simulate a variety of flow events ranging from the 5-year design storm to the Regional storm. The results of the hydraulic modelling presented in this report are not intended to update or replace the current Regulatory floodplain extents in NVCA or LSRCA watersheds. Any current or future development application will still refer to previously defined floodplain extents available from NVCA and/or LSRCA as appropriate.

2 Background Review

2.1 Data Collection

Background data listed as part of the request for proposal, as well as additional data, was requested and reviewed by Matrix to verify the quality and completeness of

available data and to identify data gaps. The data received was summarized and reviewed. A complete record of received data is provided in Appendix A.

- land use data covering the Town's jurisdiction to apply Manning's n to the 2D model
- hydrology data (MacLaren 1988) including design storm peak flows and flow node locations
- existing LSRCA Visual OTTHYMO hydrology model, and HEC-RAS hydraulic model
- existing NVCA hydrologic watershed subcatchment boundaries
- existing NVCA HEC-RAS models and shapefiles of cross-section locations, watercourse centerline, and estimated Regulatory flood extent
- 2018 Municipal Structure Inventory and Inspection
- culvert and bridge locations
- Canadian Pacific Railway (CPR) culvert and bridge dimensions
- various subdivision/development stormwater management (SWM) reports
- records of previous flooding near 5th Line and 20th Sideroad including a hydraulic analysis of the culverts in the area
- record of previous flooding and channel maintenance works on Hendrie Drain in Beeton
- GIS data from Land Information Ontario (LIO), including road centrelines, railways, watercourses, and 2013 South Central Ontario Orthophotography Project (SCOOP) DEM

2.2 Light Detection and Ranging Data Collection

Matrix subcontracted Perron, Hudon, Bélanger Inc. (PHB) to collect and provide the LiDAR information for the Town in accordance with the requirements provided in the *Federal Airborne LiDAR Data Acquisition Guideline* (Government of Canada 2018). The data was collected during one flight mission completed on November 23, 2019, under a clear sky, with leaf-off conditions for vegetation, and low flow conditions. The ground surface was free of snow cover with the exception of potential piles in parking lots that were removed during the classification process where they were deemed significant.

Additional details of the LiDAR data collection, processing, and list of deliverables are provided in *Drainage Master Plan - Phase 1 - LiDAR Survey Technical Report* (PHB 2020). Further discussion on how the topographic surface was applied in the 2D HEC-RAS model is discussed in Section 5.1. PHB provided the following files PHB for use in the hydraulic modelling:

- **1 m grid hydro-flattened DEM:** hydro-flattening is a post-processing method applied to DEMs to ensure that water surfaces appear as flat between banks and are not

increasing in the downstream direction. Hydro-flattening is recommended for use in floodplain mapping and is therefore relevant to this study (Government of Canada 2018).

- **Watercourse bank lines:** these were provided in the form of three-dimensional (3D) breaklines that represent the edge of bank elevation where abrupt change in topographic elevation is created by channel banks. The use of breaklines are discussed in further detail in Section 5.2.2.

2.3 Previously Completed Studies

The following subsections summarize previous studies that were reviewed for this project.

2.3.1 Watershed Hydrology Study

The *Watershed Hydrology Study for Nottawasaga, Pretty and Batteaux Rivers, Black Ash, Silver and Sturgeon Creeks* completed by MacLaren Plansearch Inc. (MacLaren) was used to extract peak flows and flow node locations (MacLaren 1988). The MacLaren hydrology study assessed both present and future conditions of land use. Most of the Town is within the Nottawasaga Valley watershed; therefore, the peak flows provided in the MacLaren hydrology study were used to provide inflow data for this study. The inflows were included in a shapefile provided by NVCA.

The peak flows available from NVCA are over 30 years old; however, there has not been significant development in the headwaters of the Upper Nottawasaga River and Innisfil Creek subwatersheds and therefore this data is considered suitable for use in modelling at this stage. That being said, updating peak flows within NVCA's watercourses would be beneficial to ensure the Town has the most up-to-date and detailed understanding of flood risk in its jurisdiction. This likely requires a full hydrologic study and should be coordinated with NVCA in subsequent studies. Further discussion is provided in Section 11.

2.3.2 Lake Simcoe Region Conservation Authority Hydrology Report

The *Hydrology Report for the West Holland River, East Holland River and Maskinonge River Wetlands* by Cumming Cockburn Ltd. (CCL; CCL 2005a) was provided at the beginning of the project following the request to LSRCA. The CCL hydrology study assessed both existing and future land use conditions. The southeast quadrant of New Tecumseth township is within the West Holland River watershed; therefore, the Visual

OTTHYMO model prepared during the CCL hydrology study was used to extract flow node locations and peak flow data for this study. Model output files and project reports were used to prepare the inflow data in this area.

2.3.3 Lake Simcoe Region Conservation Authority Hydraulics Report

The *Hydraulics Report for the West Holland River, East Holland River and Maskinonge River Wetlands* (CCL 2005b) was completed in conjunction with the hydrology study (CCL 2005a). The CCL hydraulic study included the development of a one-dimensional (1D) HEC-RAS model to establish flood elevations along the selected river reaches. Structure data (i.e., bridges and culverts), and flow node locations from the HEC-RAS model were used in this study.

2.3.4 Stormwater Management Reports

A number of residential developments were constructed in the study area in recent decades and others have been designed but not yet approved for construction. SWM reports provided for these developments in Alliston, Beeton, and Tottenham are listed in Appendix A. The reports were reviewed to determine the nature of work that was completed or proposed (i.e., culvert upgrades, changes in flow patterns, etc.). The following outlines how each of the report types were incorporated into the study.

- **Subdivision developments:** subdivision developments typically include provision of storm sewers, SWM features, and other urban drainage infrastructure. The scope of Phase 1 of the DMP focuses on riverine flooding and does not include modelling urban infrastructure. In addition, the flow inputs applied to the model are based on large catchments suitable for a flood risk assessment, not an urban drainage assessment. At the request of the Town, four future condition developments in Beeton are included in the model using refined flow data available in the relevant reports to review the sensitivity of this area to these developments (refer to Sections 4.2.1.1 and 7.2.1).
- **SWM:** similar to urban developments, SWM features are urban drainage infrastructure components whose function cannot be accurately represented in a riverine model such as HEC-RAS. These components should be modelled in subsequent phases of the DMP using software that is consistent with standard practice for urban modelling (e.g., PCSWMM).
- **Channel work:** any channel maintenance (cleanout) or realignment work completed prior to initiating this project was captured in the LiDAR data collected by PHB in November 2019. Watercourse realignment reports were reviewed to confirm new

structure details as required. Areas that were identified as cleaned out in the past were considered during the review of flood results.

3 Culvert and Bridge Data

Existing culvert and bridge data were provided by the Town during the background review. Major watercourses were also overlaid with the hydrologic flow nodes and public roads and railways to identify any additional crossings. The total number of documented crossings is 226 and their locations are displayed on Figure 2 for the entire study area. Larger scale maps focussed on each settlement area are provided in Figure 2.1 (Alliston), Figure 2.2 (Beeton), and Figure 2.3 (Tottenham). The culvert data is summarized in Table 1.

Table 1 Summary of Crossing Inventory

Screening	Description	Count ⁽¹⁾
Data Provided by Town	Bridges and culverts (over 3 m) on roads within the Town's jurisdiction	88
Data Provided by CPR	Bridges and culverts on CPR crossings (labelled as MSI_2, MSI_3, MSI_19, MSI_24, MSI_28, MSI_34, MSI_40, MSI_51, MSI_61, MSI_63). Note survey was completed by Matrix Solutions Inc. at some of these structures and are therefore included in the counts below.	5
Data Provided by NVCA and LSRCA	Bridges and culverts available in existing HEC-RAS models. Also includes a new 2.4 m diameter corrugated steel pipe at the NVCA Vienneau ice dam structure on Beeton Creek ⁽²⁾ .	25
No data provided	Modelled as block or cut and therefore no data was required.	6
Matrix Solutions Inc. Survey October 2019		
Survey Priority 1	Crossings within or near residential areas	32
Survey Priority 2	Crossings near residential areas but appear to be hydraulically insignificant (i.e., larger bridge structures that are less likely to impact hydraulics)	8
Survey Priority 3	Crossings located within rural parts of the study area	39

Screening	Description	Count ⁽¹⁾
Matrix Solutions Inc. Additional Survey December 2019		
Surveyed	Additional crossings that were not previously identified as hydraulically significant	23
Total		226

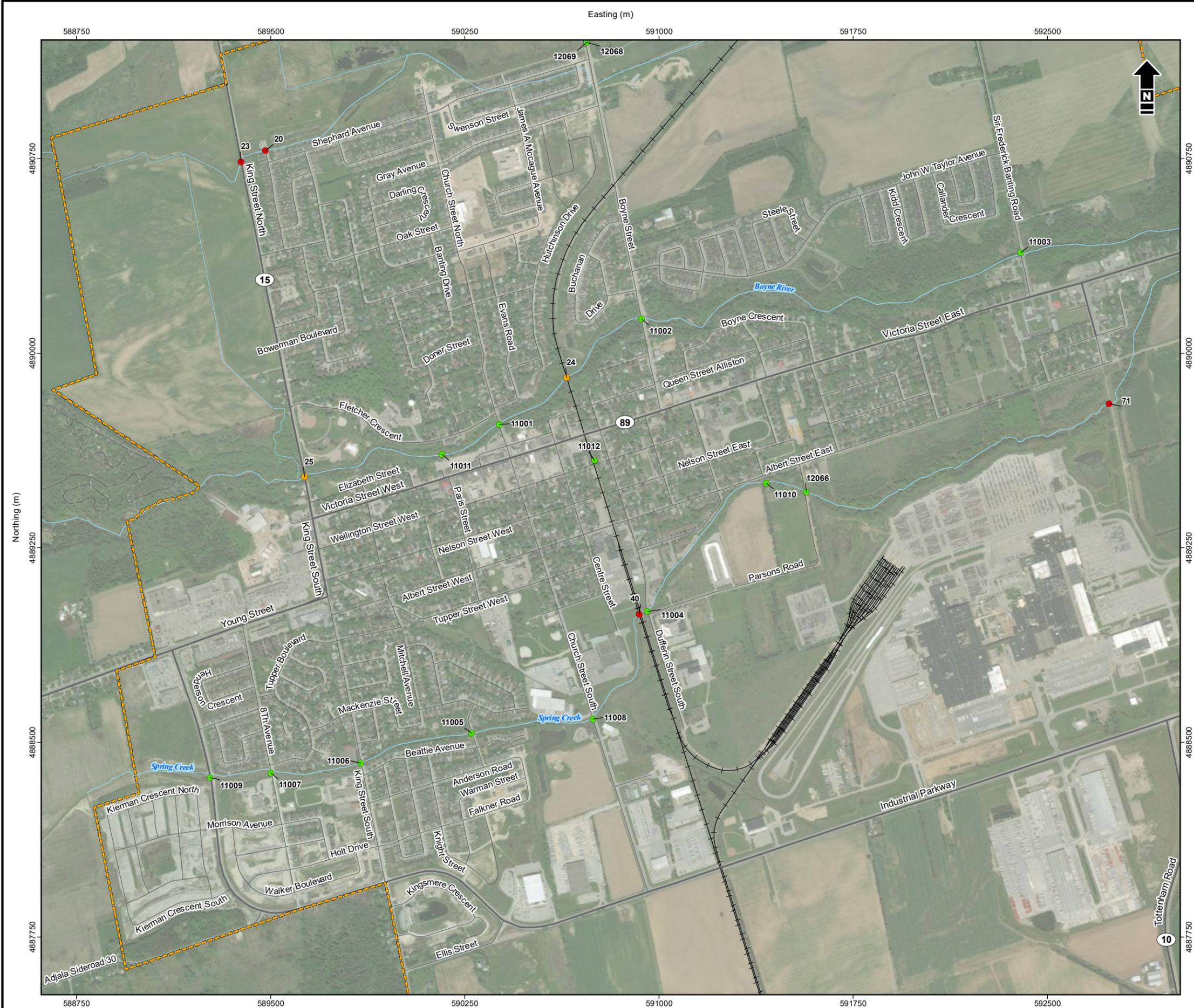
(1) Some of the crossings provided in existing datasets were also surveyed by Matrix and are included in the survey counts.

(2) *Vienneau Ice Dam Structure Preliminary Design, Town of New Tecumseth (GSS 2019).*

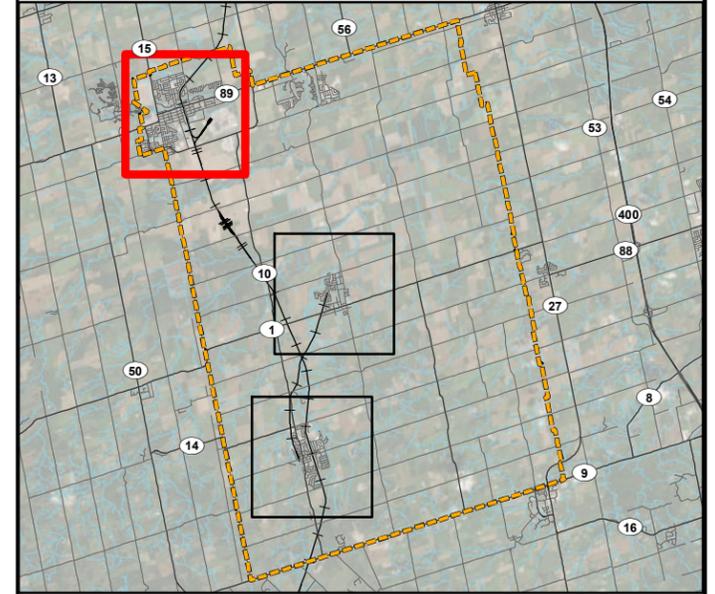
- The Town – the Town of New Tecumseth
- CPR – Canadian Pacific Railway
- NVCA – Nottawasaga Valley Conservation Authority
- LSRCA – Lake Simcoe Region Conservation Authority

Matrix completed topographic survey of crossings between October 22 and 24, 2019. A second survey was completed between December 10 and 16, 2019, to collect data for crossings that were not previously identified as being hydraulically significant and to fill gaps in the provided datasets. The raw survey data is included in the structure inventory sheets provided in Appendix C. A complete inventory of all structures in the study area including data from all available sources (i.e., the Town, CPR, Matrix survey, existing models, etc.) is provided digitally in Appendix D. The following data was collected during the surveys:

- invert and obvert elevation at the upstream and downstream ends of each crossing
- road elevation at each crossing
- photographs of the upstream and downstream ends of each crossing
- documentation of structure type, material, number of cells, inlet type, height/diameter and span, railing height (if present), whether there are piers, etc. as required for the hydraulic model



- Study Area and Municipal Boundary
 - Major Watercourse
 - Highway
 - Road
 - Railway
- Matrix Culvert Survey (Nov 2019)**
- Priority 1: Residential Areas – hydraulically significant
 - Priority 2: Residential Areas – large bridges, less significant
 - Priority 3: Rural Areas
- Matrix Culvert Field Inventory**
- Data Provided by Town of New Tecumseth
 - Data Provided by CPR
 - Data Provided by NVCA
 - Additional Matrix Culvert Survey (Dec 2019)



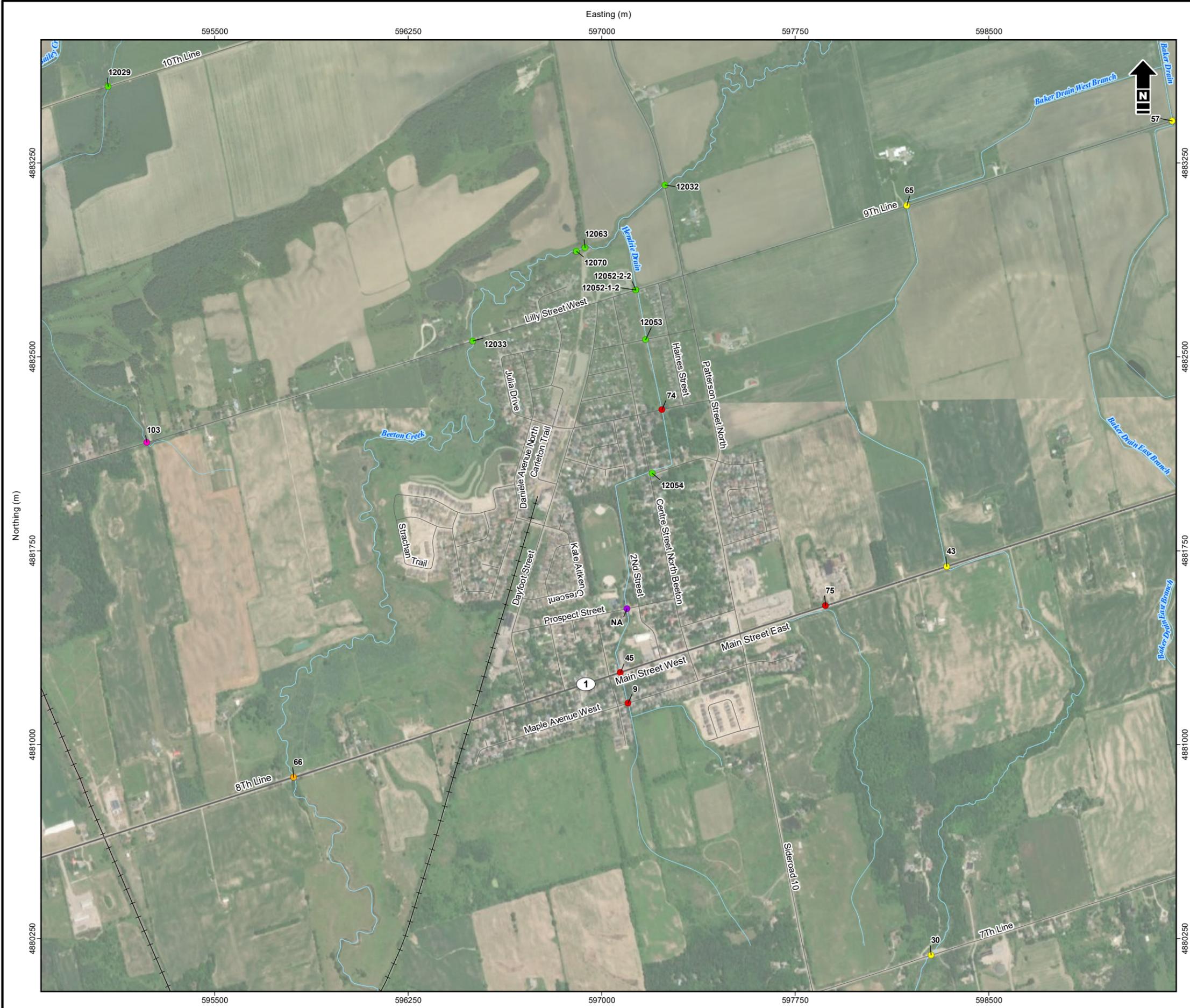
Matrix Solutions Inc.
ENVIRONMENT & ENGINEERING

The Corporation of the Town of New Tecumseth
Drainage Master Plan Phase 1

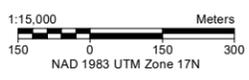
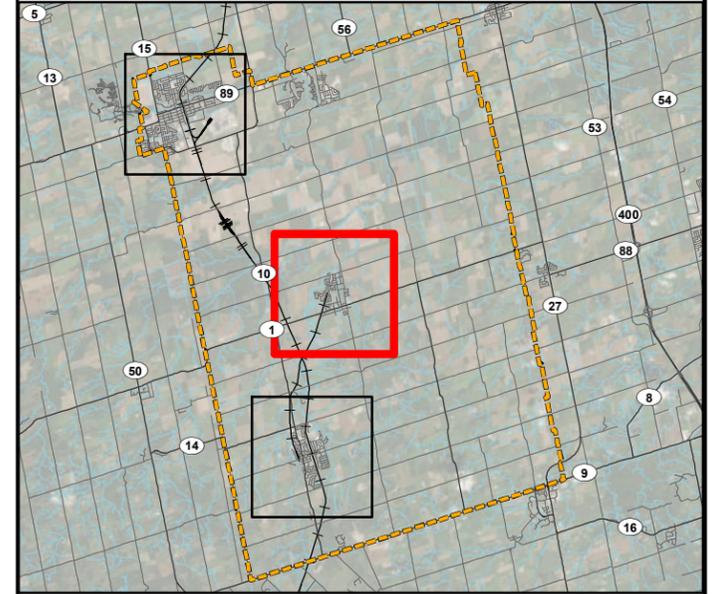
Screening of Crossings - Alliston

Date: July 2020	Project: 28997	Submitter: T. Martin	Reviewer: K. Molnar
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- Study Area and Municipal Boundary
 - Major Watercourse
 - Highway
 - Road
 - Railway
- Matrix Culvert Survey (Nov 2019)**
- Priority 1: Residential Areas – hydraulically significant
 - Priority 2: Residential Areas – large bridges, less significant
 - Priority 3: Rural Areas
- Matrix Culvert Field Inventory**
- Data Provided by Town of New Tecumseth
 - Data Provided by CPR
 - Data Provided by NVCA
 - Additional Matrix Culvert Survey (Dec 2019)



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Screening of Crossings - Beeton

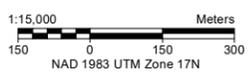
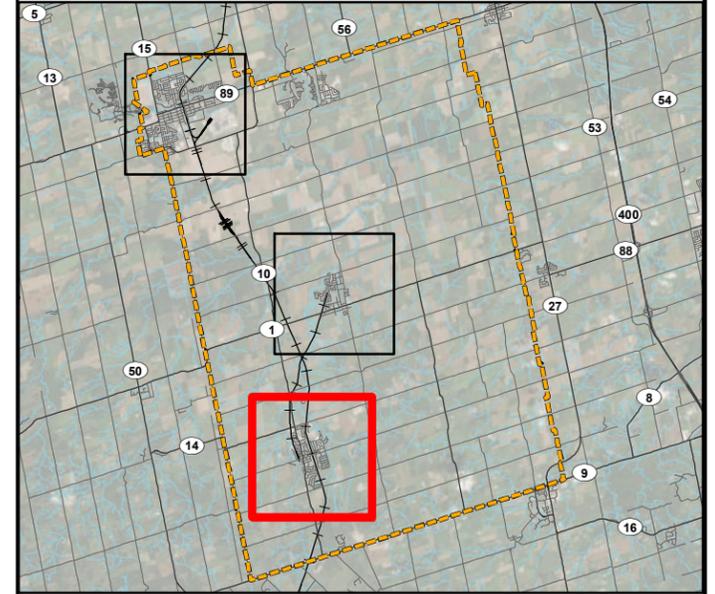
Date:	July 2020	Project:	28997	Submitter:	T. Martin	Reviewer:	K. Molnar
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- Study Area and Municipal Boundary
 - Major Watercourse
 - Highway
 - Road
 - Railway
- Matrix Culvert Survey (Nov 2019)**
- Priority 1: Residential Areas – hydraulically significant
 - Priority 2: Residential Areas – large bridges, less significant
 - Priority 3: Rural Areas
- Matrix Culvert Field Inventory**
- Data Provided by Town of New Tecumseth
 - Data Provided by CPR
 - Data Provided by NVCA
 - Additional Matrix Culvert Survey (Dec 2019)



Reference: Contains information licensed under the Open Government Licence – Ontario. Imagery Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community



The Corporation of the Town of New Tecumseth
Drainage Master Plan Phase 1

Screening of Crossings - Tottenham

Date:	July 2020	Project:	28997	Submitter:	T. Martin	Reviewer:	K. Molnar
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4 Hydrology Review

A review of the existing hydrologic modelling of the Nottawasaga River (NVCA jurisdiction) and West Holland River (LSRCA jurisdiction) through the study area was conducted as part of this project. The West Holland River watershed hydrology model was updated in 2005 and therefore is relatively current and appropriate for use in this project (CCL 2005a). The Nottawasaga River hydrology model was developed in 1988 (MacLaren 1988). While this study is over 30 years old, there has not been significant development in the headwaters of the Upper Nottawasaga River and Innisfil Creek subwatersheds and therefore this data is considered suitable for use in modelling at this stage. The following section provides details on how peak flows from each jurisdiction were incorporated into this study.

4.1 Lake Simcoe Region Conservation Authority Flow Data

Peak flows for the West Holland River were obtained from the existing HEC-RAS model provided by LSRCA. The peak flows in this model originated from a calibrated Visual OTTHYMO Version 2.0 (VO2) model prepared for the *Hydrologic and Hydraulic Modelling for the West Holland River, East Holland River and Maskinonge River Watersheds* report (CCL 2005a).

The flow events applied to LSRCA watercourses are based on future condition land use incorporated into the VO2 hydrology model, which is standard practice for floodplain mapping in Ontario (MNR 2002). The design flows available in this model include the 5-year, 10-year, 25-year, 50-year, and 100-year events and are based on a 12-hour Soil Conservation Service (SCS) Type II rainfall distribution.

In addition to the design events, Regional storm flows from the previous LSRCA model (CCL 2005b) were utilized for this project. In the West Holland River watershed, the Regional storm event is Hurricane Hazel. The Hurricane Hazel historic major storm event of 1954 occurred over a 48-hour period. Rainfall was recorded at Snelgrove, just north of Brampton; 73 mm of rain fell during the first 36 hours of the event, and an additional 212 mm of rain was recorded during the final 12 hours.

Hydrologic modelling of the Hurricane Hazel event typically consists of the final 12 hours and therefore uses Antecedent Moisture Condition III (AMC III) to represent fully saturated ground conditions resulting from the first 36 hours of rainfall. In addition, the Hurricane Hazel storm event was recorded at a specific location and therefore to maintain consistency with the geographic area over which the rainfall occurred, it should

only be applied to watersheds with areas less than 25 km². To simulate the Regional event in larger basins, an areal reduction factor is applied to the rainfall amounts based on the equivalent circular area of the watershed using the longest flow path length as the diameter (MNR 2002). Areal reduction factors do not apply to the design events.

The flow change locations (flow nodes) that were applied to the 2D HEC-RAS model within LSRCA jurisdiction are shown in Figure 5 in Section 4.4.

4.2 Nottawasaga Valley Conservation Authority Flow Data

Peak flows for the remaining rivers in the hydraulic model domain are within NVCA jurisdiction. At the beginning of the project NVCA provided Matrix with a GIS file of flow change locations that were digitized from the MacLaren (1988) report. The catchments contributing to these flow nodes are quite large and only apply to major watercourses. Therefore, subdividing the flows was required to ensure flows were provided on smaller channels and tributaries within each catchment. This also avoids overloading the 2D HEC-RAS model with large inflows at the flow input locations.

A significant amount of effort would be required to subdivide drainage catchments to a level that would be appropriate for the 2D HEC-RAS model. The NVCA's Generic Regulations HEC-RAS model includes flows that are based on the *Watershed Hydrology Study for Nottawasaga, Pretty and Batteaux Rivers, Black Ash, Silver and Sturgeon Creeks* report (MacLaren 1988) and were subdivided using a linear interpolation approach. The Regional storm peak flows from the NVCA's existing HEC-RAS model were used as the inputs for the 2D HEC-RAS model.

4.2.1 Revisions to Nottawasaga Valley Conservation Authority Flows

4.2.1.1 Updated Flows at Beeton

Within the Town, the major storm event of June 2017 had the largest impact on the community of Beeton causing significant widespread flooding. Detailed studies and alterations to drainage patterns have been completed in the Beeton area since the MacLaren (1988) hydrology study. As a result, updates to the flow data provided by NVCA was required on Hendrie Drain (which flows through Beeton) and Baker Drain (which flows along the eastern edge of Beeton).

In consultation with NVCA, flows provided in the recent detailed studies (N.H.D. 2013, C.F. Crozier 2016) were used for this project because they provide a more refined and current understanding of flood conditions in this area of concern. This also enables

modelling four proposed development areas in Beeton (Sorbara, Flato North, Flato South, and Oxnard developments) as requested by the Town. The future conditions scenario was prepared as part of the sensitivity analysis discussed in Section 7.2.1.

4.2.1.2 Additional Flow Corrections

In a few locations, flow anomalies were identified in the data provided by NVCA. These locations are summarized in Table 2 including the adopted method to address each of the anomalies. Some small upstream tributary flows were unreasonably large compared to adjacent tributaries or were missing from the HEC-RAS models. To determine the flow inputs for these tributaries in the 2D HEC-RAS model Matrix subtracted the total confluence flow in the NVCA's HEC-RAS model from the upstream flow on the main branch. The difference between these values is considered to be the tributary inflow and was applied at the upstream end of these tributaries. This approach and the updated flow values were discussed with and accepted by NVCA for use in this project. Note that the model results using updated flows are intended to inform the Town of existing flood risk and do not replace the current NVCA Regulatory floodlines.

Table 2 Nottawasaga Valley Conservation Authority Flow Anomalies

Location	Identified Issue	Method to Address
Bailey Creek, XS 13717.62	XS 13717.62 spans multiple tributaries to Bailey Creek	Distributed flow across the three Bailey Creek tributaries
Bailey Tributary A4, XS 1756.863	Tributary flow too high ⁽¹⁾	Calculated new tributary flow ⁽²⁾
Bailey Tributary A5, XS 247.3252	Tributary flow too high	Calculated new tributary flow
Bailey Tributary A6, XS 280.0543	Tributary flow too high	Calculated new tributary flow
Bailey Tributary C3, XS 5692.097	Tributary flow too high	Calculated new tributary flow
Beeton Tributary A1, XS 1500.71	Tributary flow too high	Calculated new tributary flow
Beeton Trib. A6A2, XS 174.3327	Tributary flow too high	Calculated new tributary flow
Beeton Tributary B2, XS 1793.471	Tributary flow too high	Calculated new tributary flow
Beeton Tributary A11, XS 236.3763	Tributary flow too high	Calculated new tributary flow

Location	Identified Issue	Method to Address
Beeton Tributary A10, XS 664.7734	Tributary flow too high	Calculated new tributary flow
Beeton Tributary B1, XS 697.0068	Tributary flow too high	Calculated new tributary flow
Beeton Trib. A4A2, XS 798.1252	Tributary flow too high	Calculated new tributary flow
Beeton Tributary 11, XS 844.1422	Tributary flow too high	Calculated new tributary flow
Mid Nottawasaga Tributary (Three locations)	Tributary and flows not available from Nottawasaga Valley Conservation Authority models	Approximate Regional flow based on MacLaren flows for Spring Creek catchment, which has a similar drainage area and land use
Penville Tributary C, XS 2059.03	Tributary flow too high	Calculated new tributary flow

- (1) Tributary flow appears too high relative to similar tributaries.
- (2) Calculated tributary flow as total confluence flow minus upstream main branch flow.

Throughout the study area, additional flow corrections were required to account for the difference in modelling technique. Specifically, the flow input methods for traditional 1D steady state hydraulic modelling (i.e., the data source from NVCA) and the 2D quasi-steady hydraulic modelling being completed in this study are different. Details on the purpose and application of these corrections are provided in Section 4.4.1.

4.2.2 Design Event Flows

The existing NVCA HEC-RAS model includes Regional storm flows only. In the Nottawasaga River watershed, the Regional storm event is the Timmins Storm, which occurred in 1961 in Timmins, Ontario and produced 193 mm of rainfall in a 12-hour period. Similar to the Hurricane Hazel storm, the Timmins Storm is to be applied to basins smaller than 25 km² and an areal reduction factor is applied to the Timmins rainfall amounts for drainage areas larger than 25 km² based on the equivalent circular area method.

Since the NVCA’s existing HEC-RAS model only includes the Regional storm, a method for calculating design event peak flows was required. As previously mentioned, use of the MacLaren design event peak flows directly would overload the model due to the large size of the MacLaren subcatchments. More importantly, without applying flow at each modelled junction, the smaller tributaries would appear dry in the model and would therefore not accurately reflect flow conditions in the upstream areas.

An approach for applying design storm flows at the flow change locations within NVCA jurisdiction was developed in consultation with NVCA. Using the peak flows provided in the 1988 MacLaren report, the ratio between the Regional storm peak flow and each design event peak flow was calculated. An example is illustrated in Table 3 for catchment 205 as indicated on Figure 1.1 of the MacLaren (1988) report. In this example, the ratios for each design event were applied to all subdivided flow nodes that fall within catchment 205 as delineated by MacLaren (1988). A record of the design event ratios and flow input values used throughout the 2D HEC-RAS model is provided in Appendix F.

Table 3 Example Design Event Ratios for Nottawasaga Valley Conservation Authority Jurisdiction

Storm Event	Total Peak Flow (m ³ /s) ⁽¹⁾	Ratio to Regional Storm
5-year	20.4	0.17
10-year	24.7	0.20
20-year ⁽²⁾	29.3	0.24
50-year	35.5	0.29
100-year	40.2	0.33
Regional (Timmins)	123.4	1.00

(1) *Watershed Hydrology Study for Nottawasaga, Pretty and Batteaux Rivers, Black Ash, Silver and Sturgeon Creeks* (MacLaren 1988)

(2) The return period provided in the MacLaren (1988) report reflects standard practice at the time. More recent peak flows estimates use the 25-year return period.

One thing to note is that the Regional storm flows provided in the MacLaren (1988) hydrology report include areal reduction factors for basins larger than 25 km². Typically, areal reduction factors are not applied to design events. However, considering that the areal reduction factors are applied to the rainfall input to the hydrologic model and not the resulting peak flow output, it would be difficult to quantify the impact that the areal reduction factor had on the resulting output without rerunning the NVCA’s hydrology model. For the purpose of this study it was determined that the design events would be developed using the ratios that incorporate areal reduction factors. This will result in

slightly lower peak flow for the design events but was determined to be appropriate for the screening level assessment completed at this time. In future studies, when updated hydrology data is available for NVCA watercourses, it is recommended that the peak flows applied in the 2D HEC-RAS model be updated.

4.2.3 Combined Flow Data from Lake Simcoe Region Conservation Authority and Nottawasaga Valley Conservation Authority

There were a few differences between the NVCA and LSRCA flow events available for inputs to the model. These inconsistencies and the approach used to address them are described below.

- NVCA uses the Timmins Storm as the Regional event, while LSRCA uses Hurricane Hazel. Therefore, the Regional storm run includes a combination of these two historic events applied to the relevant watercourses. The LSRCA and NVCA tributaries do not interact hydraulically and therefore combining these flow events was considered reasonable.
- The design event return frequencies used by LSRCA and NVCA are different due to the era in which they were each prepared. In the past, calculating the 20-year return period was standard practice, while more recent flow estimates in Ontario use the 25-year return period. The MacLaren flows provided by NVCA include a 20-year return period event, while LSRCA events used a 25-year return period. Additionally, the updated local data in Beeton also used the 25-year return period event. Similar to the Regional event, a combination of design event return periods was applied to the model simulation. Since NVCA's jurisdiction covers the majority of the study area, the model simulation file was labelled as the 20-year event even though 25-year return period flows were applied to LSRCA's watercourses and in Beeton.

4.3 Stormwater Management

For floodplain delineation purposes, “stormwater management facilities may not be used to provide any reduction in flood flows” (p.18a, MNR 2002). SWM facilities are not typically designed to contain flows from the Regional storm event. Furthermore, the Ministry of Natural Resources and Forestry (MNR; previously the Ministry of Natural Resources [MNR]) policy states that wherever possible hydraulic modelling for floodplain delineation be completed using a steady-state hydraulic modelling approach which does not consider attenuation effects due to storage or reverse flow conditions (MNR 2002). While this is not a floodplain mapping study, adopting the approach

outlined by the MNRF will provide a conservative estimate of flood risk in the study area and therefore was adopted for this project.

Sometimes SWM facilities are considered in models of the smaller design events for flood risk studies. However, including SWM facilities would be a component of the hydrology portion of the study which should be a component of future phases of the DMP. SWM facilities have little impact on the hydraulic component of a flood risk study, particularly when using a steady-state or quasi-steady-state approach. Hydrology data for this study area is simply not refined enough to enable SWM assessment at this time.

The Town has expressed interest in the existing level of service of local drainage infrastructure (storm sewers, etc.), and the benefits of future SWM and other improvements. In order to adequately address these types of questions, a detailed urban drainage assessment is required. It is recommended that Phase 2 of the DMP include an integrated 1D-2D (sewer and overland) drainage study including urban hydrology for areas of interest.

4.4 Quasi-steady Hydrographs

The method for incorporating flow input to a hydraulic model depends on the type of modelling being completed (steady vs. unsteady) and the anticipated hydraulic conditions within the study area. The four general types of flow analysis are described below:

- **One-dimensional steady flow:** typically used to calculate flood elevations in situations where peak flows are not largely influenced by storage attenuation, the channels and reaches are well-defined and known in advance, and watercourses are not subject to reverse flow conditions. This method consists of applying single peak flow values to flow change locations in a 1D hydraulic model (i.e., a hydraulic model in which geometry is represented by cross-sections along each river reach). Due to the large storage area formed by Beeton Flats (refer to Section 5.5.1), and the wide-flat nature of the study area, 1D steady flow modelling is not ideal for assessing flood risk here. However, steady state hydraulic modelling is the recommended approach for floodplain delineation in Ontario (MNR 2002) and as such the available NVCA and LSRCA HEC-RAS models were prepared in this manner.
- **One-dimensional unsteady flow:** ideal for calculating flood elevations in open, well-defined channels or reaches with significant storage, reverse flow, or subject to rapidly varying flow conditions. This method consists of applying time-series hydrographs to flow input locations in a 1D hydraulic model. The flat nature of the New

Tecumseth study area makes it difficult to define all channels and overland flow paths in the study area with certainty and therefore a model developed in this manner may not accurately reflect flood conditions. For this reason, 1D unsteady flow modelling is not the most appropriate method for assessing flood risk in the study area.

- **Two-dimensional steady flow:** applicable to watercourses in wide, flat floodplains where flow may be travelling in two or more directions, is disconnected from the main river (i.e., split flow), or where flow patterns are complex, not known in advance, and/or difficult to visualize. Two-dimensional models are constructed of a grid or mesh to define topography and steady flows are input as single flow values to particular locations within the model domain. A 2D steady flow application can be used to replicate the recommendations of MNRF standard practice in complex hydraulic systems. However, applying steady-state flows in a 2D hydraulic model tends to create computational issues in the model because water is introduced instantaneously at the start of the simulation and is therefore not ideal for large flow events such as those simulated in this study. This is especially true in large study areas where mesh resolution and computational time step must be balanced with file sizes and simulation times (refer to Section 5.1).
- **Two-dimensional unsteady flow:** applicable to watercourses in wide, flat floodplains where flow may be travelling in multiple directions, or is disconnected from the main river, and where storage attenuation largely influences peak flows. Unsteady flow inputs are applied as hydrographs to particular locations within the model domain to replicate realistic results in areas with complex flow patterns.

The HEC-RAS models provided by NVCA and LSCRA were prepared using a 1D steady approach and therefore include single peak flow values at each flow change location. To be consistent with recommendations in the *Technical Guide – River and Stream Systems: Flooding Hazard Limit* (MNR 2002) and current MNRF policy for Regulatory floodplain delineation, a steady-state approach was adopted for this study as it provides a conservative estimate of flood conditions. However, hydraulic modelling using 2D solvers, such as the 2D HEC-RAS model developed for this project, is designed for unsteady flow analysis which means that the model requires time series-based inflow hydrograph input as opposed to a single value.

To replicate the steady-state hydraulic modelling regime, quasi-steady hydrographs were developed for the Regional storm event whereby the flow is gradually increased to reach the peak flow values and was held constant until steady state was achieved throughout the study area. Using this approach reduces potential instabilities or errors caused by the instantaneous inflow of water at the beginning of the simulation.

A quasi-steady hydrograph for the Regional flow at each of the 142 flow nodes in the model domain was incorporated into the 2D HEC-RAS model. For the design event flows, Matrix used the “multiplier” feature available in the flow input menu of HEC-RAS. This multiplier gets applied to every value in the Regional storm quasi-steady hydrograph which creates quasi-steady inputs for the various design flow events. The multipliers applied in the model are consistent with the ratios of Regional storm to each design flow discussed in Section 4.2.2. The existing LSRCA HEC-RAS model provided flow values for the design events; however, to maintain consistency in the flow input approach, the ratio of the Regional to design event flows was calculated for each LSRCA flow node and entered into the corresponding multiplier field.

4.4.1 Flow Corrections for Quasi-steady Modelling

In steady-state hydraulic modelling, flow inputs are required at the upstream end of each branch and at every confluence to represent the combined flow at that location (tributary flow plus main branch flow). In unsteady and quasi-steady hydraulic modelling, flow inputs are applied to the upstream end of each branch as well, but in this case the model is capable of combining the flows itself and therefore inputting total flows at confluence locations is not required.

A generalized version of this concept is demonstrated in Figure 3 using a schematic example of a tributary entering a main branch. In the steady-state model example (left figure), flows are applied at the upstream ends and the confluence. The confluence flow represents the combined flows from each branch as calculated by a hydrology model that accounts for flow attenuation within the system.

In the unsteady (and quasi-steady) state example (right figure), the flows are applied to the upstream end of each branch only. Flows at the confluence are calculated internally by the model to be the combined flows in each branch and no flow change input is required at the confluence.

The example flow values provided in Figure 3 demonstrates the fact that the peak flows in each tributary to a channel typically do not sum directly. The summation of flows occurs through the superposition of hydrographs such that the total flow at the confluence accounts for differences in the time to peak of the two watercourses. This means that the peak flow downstream of the confluence can be slightly less than a direct sum of the peak flows from upstream. The summation can occur in the hydrology model (required in the case of steady and quasi-steady simulations) or hydraulic model (possible in unsteady simulations). A sample hydrograph summation following the previous example is provided in Figure 4.

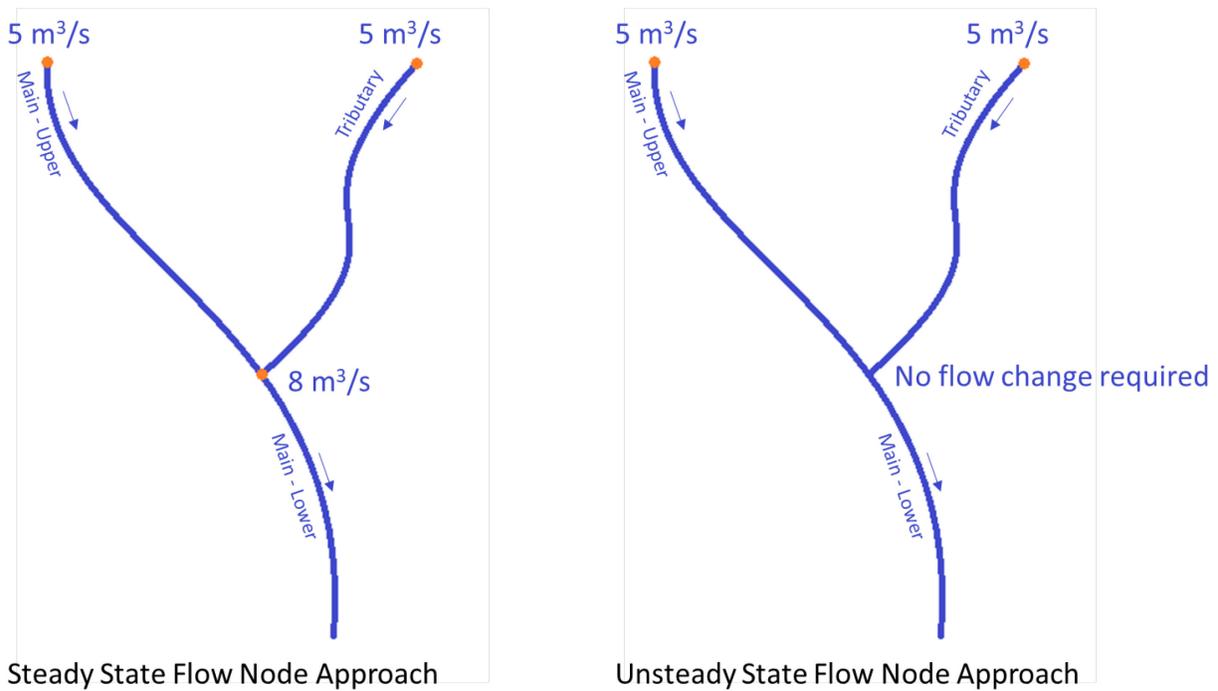


Figure 3 Comparison of Steady-state and Unsteady-state Model Inflow Requirements

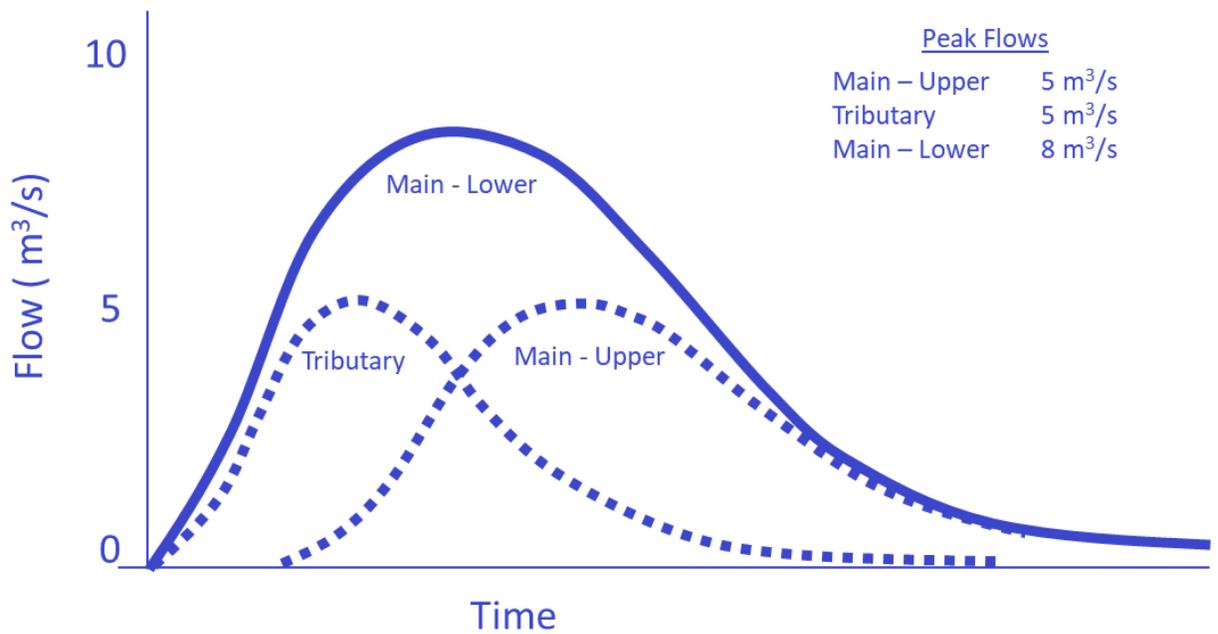
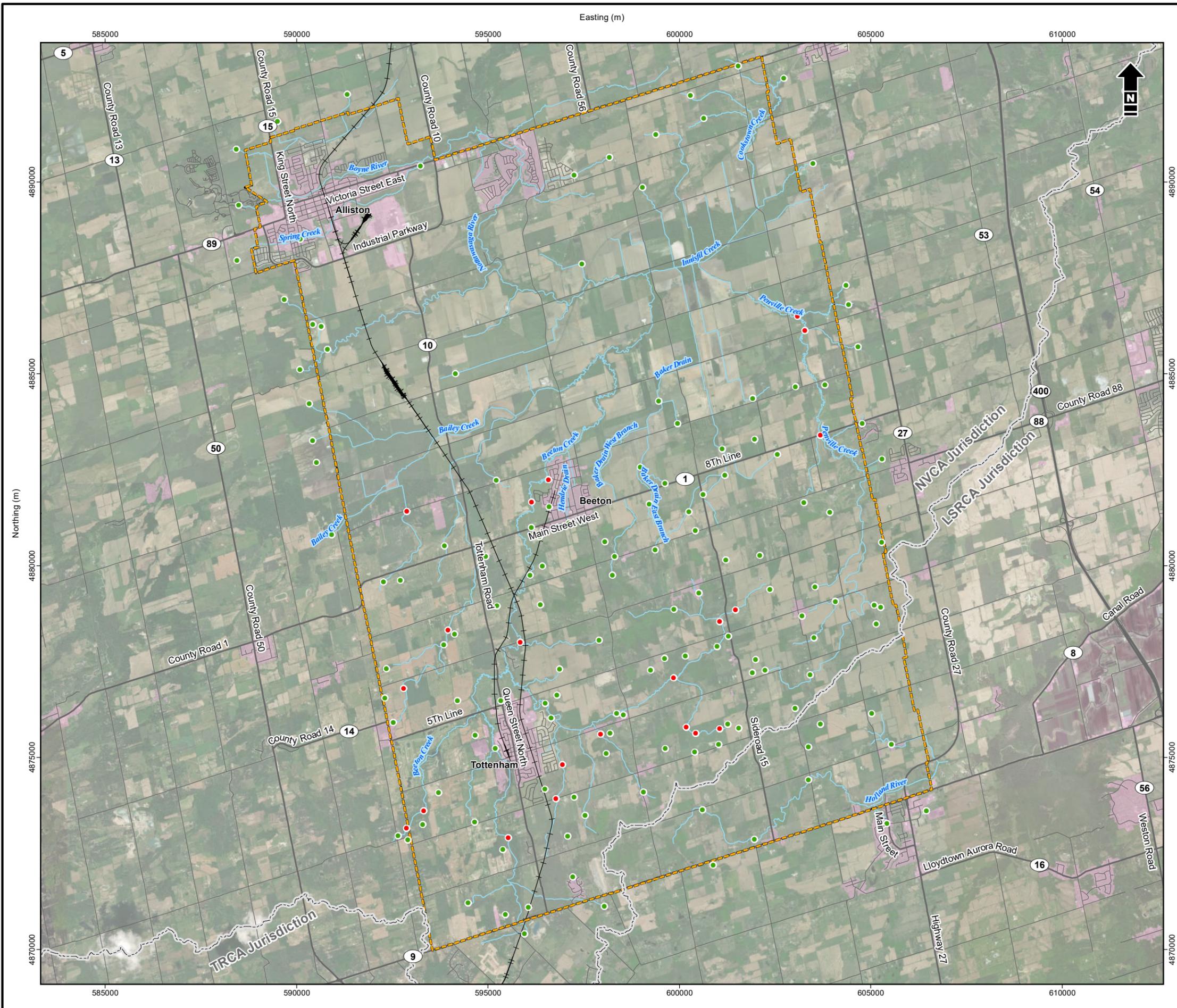


Figure 4 Example Hydrograph Summation

Quasi-steady-state hydraulic models are intended to mimic the effects of steady-state models in that they do not account for difference in timing of peak flows and therefore flow corrections are required at confluences to ensure the summation of flows does not produce an unrealistic flow value. In locations as exemplified in Figure 4 where incoming tributary peak flows in the quasi-steady model summed to a higher value than the NVCA peak flow downstream of the confluence, a negative quasi-steady inflow was applied at the confluence to reduce the peak flow and to ensure specified flows are maintained downstream of the confluence. This flow correction approach was applied in consultation with NVCA. Continuing the same example, a negative quasi-steady hydrograph of $2 \text{ m}^3/\text{s}$ would be applied downstream of the confluence to bring the total flow at the confluence back down to $8 \text{ m}^3/\text{s}$. Figure 5 indicates the flow change locations applied in the 2D HEC-RAS model including flow inputs (142 locations) and the quasi-steady flow corrections (20 locations) to enable the quasi-steady modelling technique.



- Study Area and Municipal Boundary
- Conservation Authority boundary
- Built-up Area
- Railway
- Highway
- Road
- Flow Input Node
- Flow Correction Node
- Tributary



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The Corporation of the Town of New Tecumseth
Drainage Master Plan Phase 1

Hydrologic Flow Nodes

Date: July 2020 Project: 28997 Submitter: K. Molnar Reviewer: K. Hofbauer

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5 Hydraulic Model Development

The 2D HEC-RAS model development included the following general tasks further discussed in the subsections below.

- generating a mesh of 2D flow area cells of the surface topography to calculate water elevations within the study area; the mesh includes buildings and other features such as roads and watercourses
- assigning surface roughness values (Manning's n) for various land use types based on information provided by the Town; this is required to ensure that friction losses due to different surface roughness is accounted for in the flow calculations
- incorporating bridges and culverts into the model to accurately represent flood conditions in the study area; due to the size of the study area a screening approach was applied to determine the most suitable method for modelling each structure
- assigning boundary conditions to the model; this includes inflows and appropriate outflow conditions at the model boundaries

5.1 Two-dimensional Hydraulic Modelling Concepts

The HEC-RAS model developed for this project uses a full 2D approach in which the topography in the study area including all watercourses and overbank areas (floodplains, overland flow paths, etc.) are represented by a mesh of 2D cells. During the simulation, computations occur at each cell allowing water to flow from one cell to the next along the path of least resistance. Flow exchange occurs across the face of each cell (referred to as the cell face), which is the line between two adjacent cells. The flow exchange between adjacent cells is based on an elevation to flow relationship computed by HEC-RAS along the cell face based on the detailed terrain underneath.

Cell size plays a large role in model detail, computational effort, and stability; therefore, optimizing mesh resolution is a key step in 2D modelling. The mesh cells need to be small enough to capture detailed topographic features but large enough that the model is manageable to use. Since computations occur at each cell in the model, increasing the number of cells increases the number of computations that occur at each modelled timestep. This impacts the duration of the simulation and size of the modelling files and resulting output. In addition, the software developer recommends that the mesh should not include more than one million cells (USACE 2016). Representing complex features in the mesh (i.e., tightly spaced small buildings, buildings with irregular shapes, etc.) can complicate the mesh development process as the model attempts to create many

small mesh elements to represent fine details, and therefore, simplifications to input files are often required.

Another key component in 2D modelling is selecting an appropriate computational time step to ensure the model is stable for all simulated events. The computational time step is the incremental change in time for which the model equations are being solved to calculate water surface elevations at cells. The computational time step must be less than the time it takes for water to travel through each cell, otherwise the model may not be able to come to a solution causing oscillations in the resulting water levels. In addition, the time step should not be too small as this will not only increase simulation time and file sizes but can also lead to instabilities if too much water is trying to pass through a cell in too short of a time step.

5.2 Mesh Development

5.2.1 Mesh Resolution

One benefit of HEC-RAS over other 2D modelling programs is that the 2D cells in HEC-RAS maintain the underlying ground elevations within each cell. Many other modelling programs are limited in that each cell is assumed to have a flat bottom represented by a single average elevation within that cell. This means that in order to represent significant changes in topography or areas with fine details, a fine mesh of small cells would be required. In HEC-RAS the flow transfer is based on a pre-processed elevation to flow relationship; cells do not have to have one constant elevation. This approach allows for the use of larger computational cells without losing details of the underlying terrain that govern the movement of flow (USACE 2016). This is extremely beneficial for large study areas such as in this study.

A constant mesh resolution across the entire study area was not feasible as it would exceed the recommended maximum number of 2D cells the program can handle. Matrix reviewed existing hydraulic information to determine appropriate mesh resolutions drawing upon previous experience. The various mesh resolutions were applied as follows based on land use and likelihood of inundation (refer to Figure 6):

1. A 10 m x 10 m mesh resolution was applied to the urban settlement areas of Alliston, Beeton, and Tottenham. The extents of these settlement areas were delineated using aerial imagery. This higher resolution allows for capturing detailed changes in topography and key features such as tightly spaced buildings, roadways, curbs, and sidewalks, etc. to provide an accurate and reliable representation of overland flooding in the developed areas of interest.

2. A 20 m x 20 m mesh resolution was applied to rural areas that are expected to be inundated, based on the existing NVCA and LSRCA floodlines, plus a 200 m buffer. This resolution allows for representing topography in enough detail in areas where flooding is expected while still allowing for representation of buildings and roads. Buildings and other obstructions in rural areas are typically more spaced apart than urban areas and therefore a coarser mesh is appropriate.
3. A 100 m x 100 m mesh resolution was applied to rural areas that are not expected to be inundated based on the existing NVCA and LSRCA floodlines, plus a 200 m buffer. Fine detail in these areas is not required since they will not be flooded.

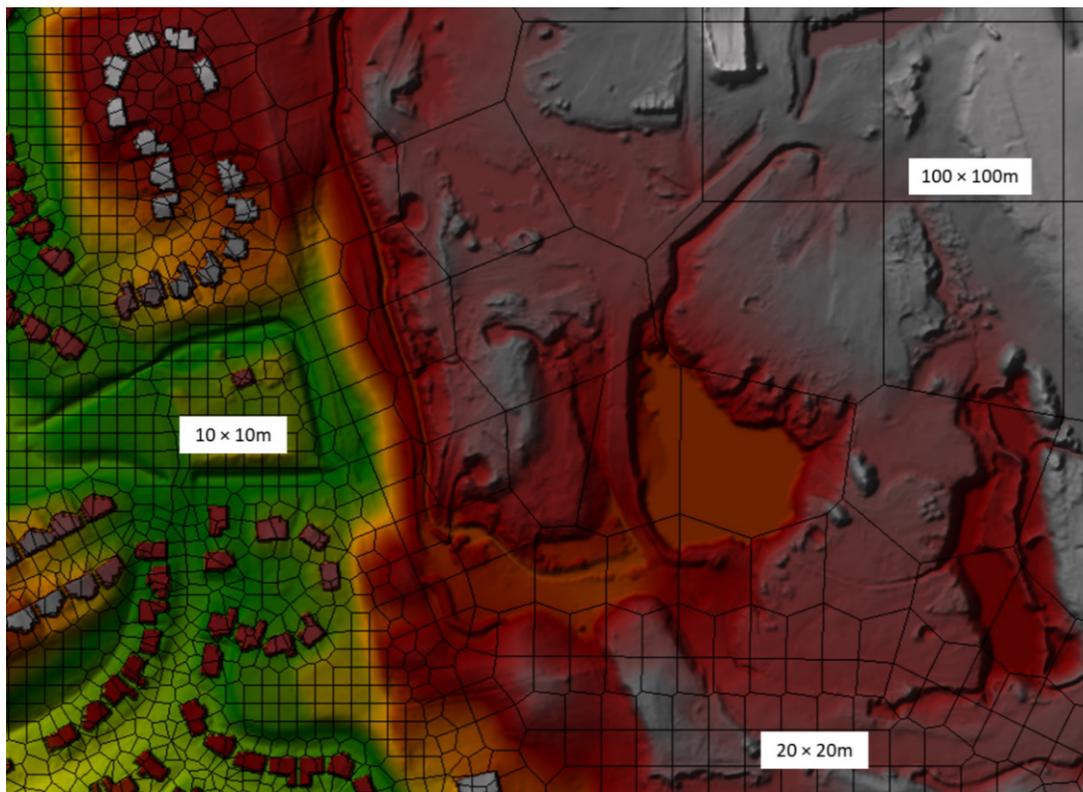


Figure 6 Example of Various Mesh Resolution in the Two-dimensional HEC-RAS Model

5.2.2 Breaklines

Breaklines are lines that are used to connect distinct topographic features such as a ridge line (high point), edge of pavement, toe of a slope, centreline of a road, or flow line of a ditch or stream. Breaklines were added to the 2D HEC-RAS model to ensure significant topographic features or changes in elevation (i.e., roads, railways, buildings, riverbanks, etc.) are appropriately represented by the mesh. Breaklines were also used

to split the model into areas with different mesh resolutions. An example of the breaklines in the 2D HEC-RAS model is shown in Figure 7.

Matrix used shapefiles of roads, railways, watercourse centrelines, and water body outlines collected from LIO during the background review as breaklines in the model. The water body shapefile obtained from LIO includes riverbanks on larger watercourses, such as the Nottawasaga River and Bailey Creek through Alliston and was used as the breaklines for these large watercourses. Riverbank lines were not available for smaller watercourses and instead, the LIO watercourse centreline shapefile was used as the breaklines. This ensured the low point along the river is represented in the mesh.

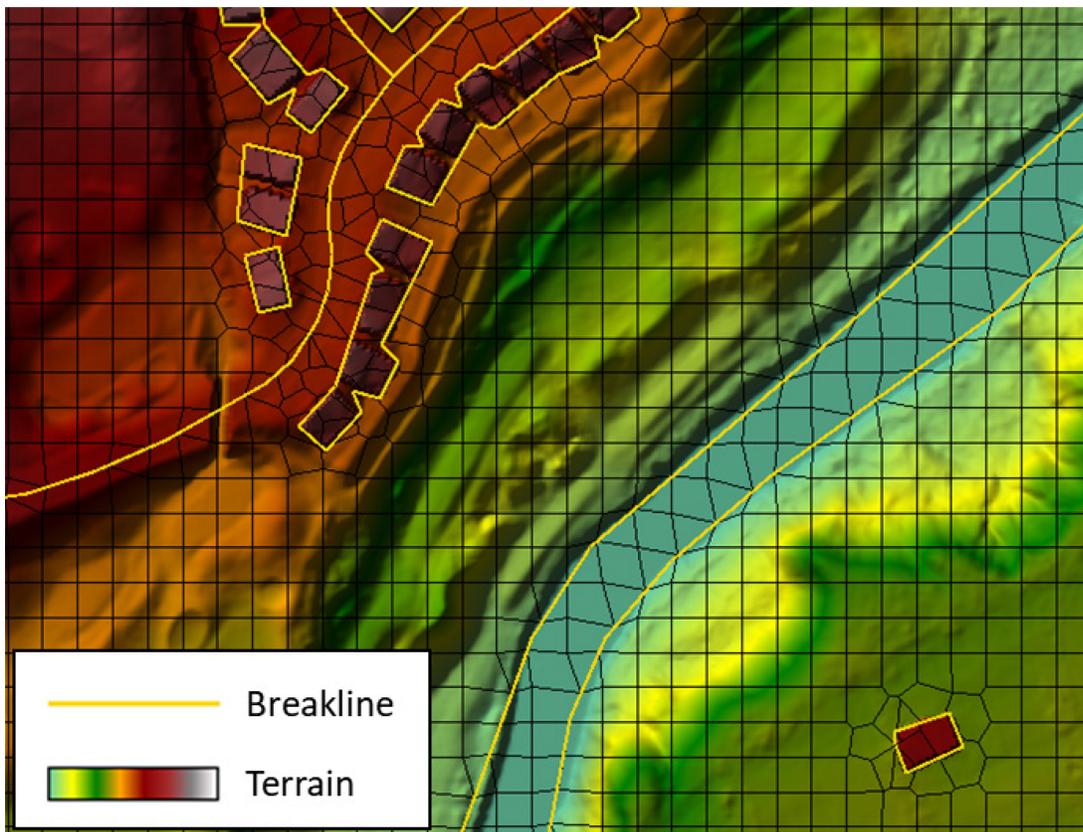


Figure 7 Example of Breaklines in the Two-dimensional HEC-RAS Model

PHB provided 3D breaklines that were created during the LiDAR production process to define the edge of all watercourse features within the model domain. However, the 2D mesh development was initiated prior to LiDAR data availability and an approach using data from LIO (riverbanks and centrelines) was already implemented as described above. Matrix reviewed the 3D breaklines provided by PHB following the delivery of the LiDAR data. Generally speaking, the breaklines developed by Matrix from the LIO data

were similar to the PHB breaklines for the purposes of this model development. The main difference between these datasets is that the 3D breaklines produced by PHB included riverbank lines for all watercourses including those that are very small. Inclusion of these breaklines for very small channels would create errors in the mesh development process as the distance between the riverbanks on these small watercourses is less than the proposed 10×10 m mesh resolution on watercourses. Following this review, Matrix determined the breaklines developed from the LIO datasets were appropriate to use in this modelling for Phase 1 of the DMP. However, Matrix recommends that the model developed in Phase 2 use the refined breaklines available from PHB.

5.2.3 Obstructions

Buildings were added to the terrain to simulate the blockage that buildings in the floodplain represent to flow conveyance; all buildings within Alliston, Beeton, and Tottenham were included in the terrain. Before adjusting the terrain, Matrix first filtered out small buildings with an area ≤ 20 m² as illustrated in Figure 8. These small buildings generally include external sheds which are not considered to be significant blockages and as such, do not need to be included in a flood model of this nature.

In addition, buildings in rural areas that are not expected to be inundated during the modelled storm events were not incorporated into the mesh as they would not impact model results. These buildings would not be well represented in the large mesh resolution applied in these areas (refer to Section 5.1).

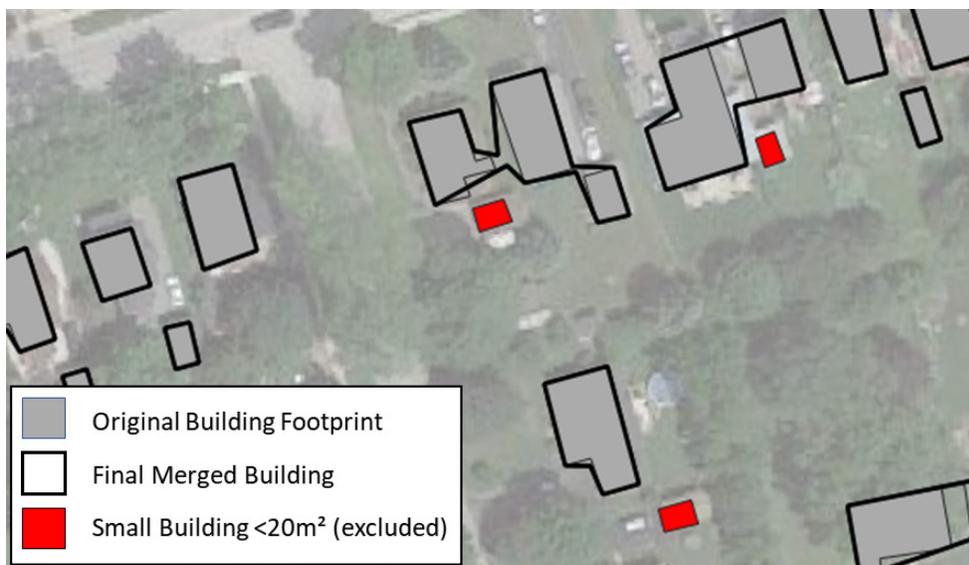


Figure 8 Example of Small Buildings Excluded from the Two-dimensional HEC-RAS Model

Our final processing step before incorporating buildings into the terrain was to simplify and merge the building polygons as shown in Figure 9. The building polygons provided to Matrix were digitized by the Town using aerial photography. These polygons are rather complex as they include irregular roof shapes, chimneys, garages, and/or porch overhangs and were therefore simplified to remove these details and simplify the mesh development. To avoid small openings between buildings that are tightly spaced, Matrix merged all buildings that are within 5 m of each other. From a hydraulics perspective, flow between buildings is considered ineffective and therefore assuming blockage between closely spaced buildings is a reasonable assumption.

The surface elevations within the final merged building footprints were raised from the DEM using the breaklines to ensure that these areas would block flow and remain dry during the model simulations.

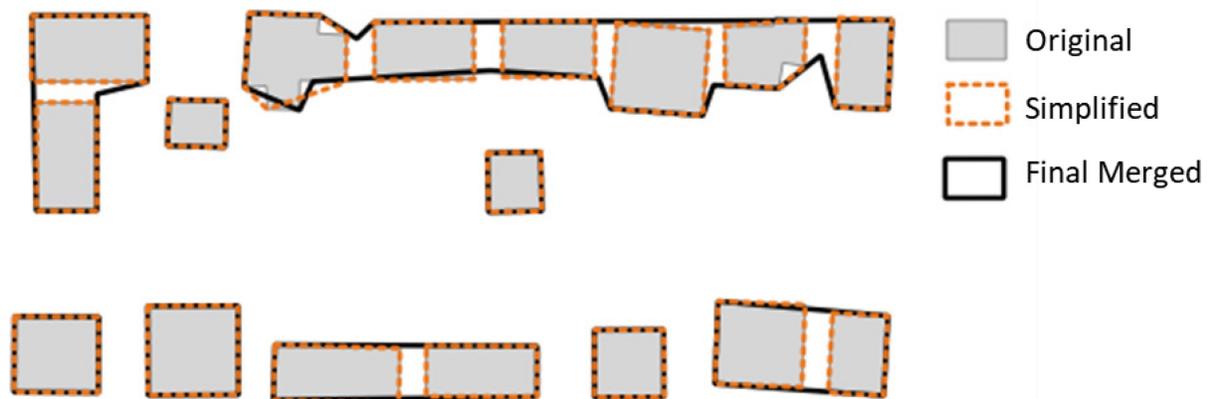


Figure 9 Example of Building Simplification for the Two-dimensional HEC-RAS Model

1.1 Surface Roughness

Land use shapefiles from the Town’s 2019 Official Plan (Town of New Tecumseth 2019) were provided to Matrix to apply surface roughness values (Manning’s n) to the model. The land use types within the Town’s jurisdiction are outlined in Table 4. Each land use type was assigned a land cover category in accordance with the *NVCA Natural Hazards Technical Guide*

(NVCA 2013). The Treetops subdivision area east of Alliston and the Canadiana Golf Course were not included in the Town’s Official Plan and therefore these were manually delineated by Matrix using aerial imagery and added to the land use shapefile. The 3D breakline file provided by PHB representing riverbanks classified during the LiDAR

production was used to apply Manning’s n values to the channels within the model domain.

Table 4 Manning’s Roughness Values

Land Use Type (Town of New Tecumseth Official Plan)	Cover (Nottawasaga Valley Conservation Authority)	Assigned Manning’s n
Environmental Protection	Woods	0.08
Agriculture	Meadows	0.07
Rural		0.07
Golf	Lawns	0.05
Urban Open Space		0.05
Urban Residential		0.05
Downtown Core Transitional		0.05
Open Water	Natural Channel	0.035
Corridor Commercial	Armourstone	0.025
Institutional	(Asphalt)	0.025
Downtown Core Commercial		0.025
Employment Area 1		0.025
Employment Area 2		0.025
Industrial		0.025
Major Commercial		0.025
Rail		0.03
Road		0.025

5.3 Bridges and Culverts

The hydraulic modelling completed for this study used a full 2D approach in HEC-RAS meaning that all drainage features in the topographic surface ranging from major rivers and creeks to ditches, gutters, and overland flow paths are represented in a 2D mesh of varying resolution across the entire study area. To ensure the 2D HEC-RAS model appropriately replicates flood conditions throughout the study area, an understanding of the hydraulic impact of each culvert and bridge was required.

Modelling culverts using a full 2D approach is different than typical 1D steady state HEC-RAS modelling and required a critical evaluation of each bridge and culvert. In a full 2D approach, there are no modelled river reaches on which to add a crossing. In addition, current limitations on 2D HEC-RAS modelling do not specifically support

modelling bridges in the same manner as a 1D model. This is because the pressure flow and road overflow calculations require 1D cross-sections. Therefore, any bridges that may experience surcharge were represented as a culvert. This is consistent with typical modelling practice (USACE 2016).

As previously mentioned, a total of 226 bridges and culverts were identified within the study area. Matrix assessed the hydraulic importance of each crossing to ensure the modelling could be completed without compromising the quality of the output. Matrix considered three approaches to integrate bridges and culverts into the model: detailed, cut, and block. Each of these methods require a different level of effort. Hydraulic considerations and the level of risk associated with modelling each crossing based on an assumption were incorporated into the selection of an appropriate modelling approach. Each of the modelling approaches are provided in the subsections below including specific details as to how they were incorporated into the model. Table 5 summarizes the modelling approach used for crossings. The complete inventory of all bridges and culverts were included in Appendix C. Input data required for the bridges and culvert that were modelled using the detailed approach are provided in Appendix G.

Table 5 Bridge and Culvert Modelling

Modelling Approach	Number of Crossings	Description
Detailed	68	Used for hydraulically significant crossings that will influence hydraulics (i.e., create backwater). These crossings were fully incorporated in the model, complete with culvert/bridge opening and weir to account for overtopping.
Cut	84	Assumes that the bridge deck does not experience pressure flow or road overtopping and therefore is only recommended for large bridge crossings that will cause minimal impacts to water levels upstream. Crossing obstructions (i.e., road decks) were removed from the terrain.
Block	74	Used for small culverts and those not located on major watercourses. This option represents complete culvert blockage and is appropriate for small culverts that provide minimal flow capacity during large events. Road obstructions were blocked to omit the culvert opening.

5.3.1 Detailed Bridge and Culvert Approach

The detailed modelling approach requires the greatest level of effort but provides the most accurate hydraulic modelling and the highest quality output. This method requires similar information as a 1D culvert modelling approach such as field survey to collect physical characteristics of the structures. In a 2D HEC-RAS model, modelled crossings require the following:

- the centreline of the crossing
- details of the crossing geometry (i.e., dimensions, material, entrance type, inverts, etc.)
- the centreline of the roadway weir
- details of the roadway weir (i.e., elevation, width, length, discharge coefficient, etc.)

A total of 68 crossings were modelled in detail. A shapefile of the bridge and culvert centrelines were created connecting upstream and downstream invert locations surveyed by Matrix or by drawing individual centerlines as observed on aerial imagery. In instances where multiple barrels were observed or surveyed, a centreline was included for each barrel. Culvert lengths were not specifically surveyed onsite by Matrix due to safety concerns related to walking through bridges/culverts and measuring across roadways. Therefore, in cases where culvert length was not available from the Town or NVCA datasets, the modelled bridge/culvert length was set to match the drawn centreline.

To calculate weir overflow at crossings, road centrelines were developed using existing road and railway data. Weir geometry was extracted from the LiDAR data in HEC-RAS to represent the road elevations along the centreline. In cases where the road deck was cut out by PHB during the LiDAR production process, Matrix filled these in using adjacent elevations. This was required to adequately represent the flow capacity through the crossing and over the road for overtopping situations. Each filled-in crossing was reviewed to ensure tie-ins were consistent with adjacent surroundings.

Furthermore, rigid barriers such as railings create blockages to flow and it is standard practice to include them when modelling crossings. During the field investigation and background review of crossings, the presence and dimensions of solid steel railings, concrete barriers, and/or handrails were recorded and added to the modelled road deck.

An example of a watercourse crossing modelled in detail in 2D HEC-RAS is shown in Figure 10. This illustrates an instance where the road deck was removed from the terrain in the LiDAR production process but added back to the model with the railing to demonstrate blockages to flow.

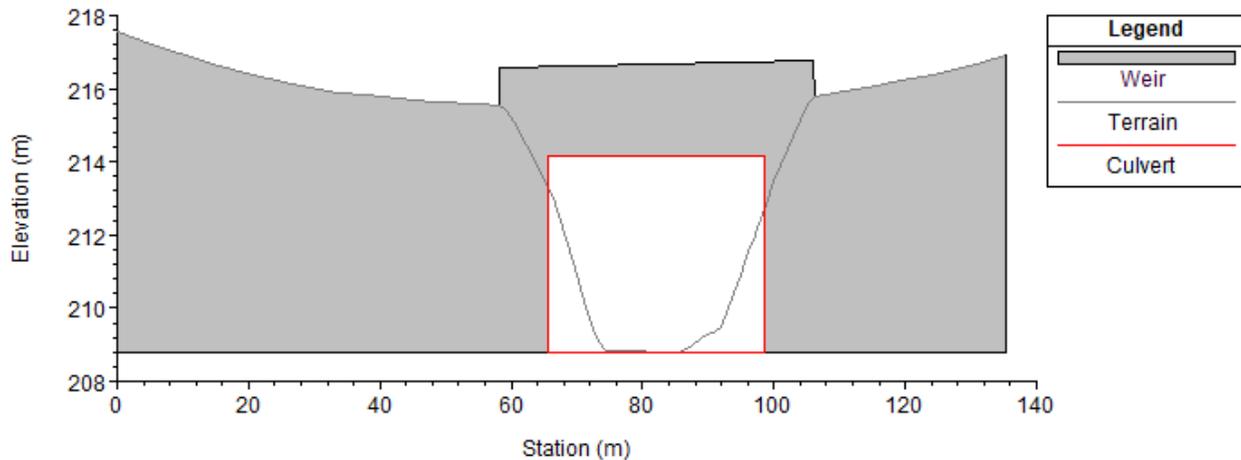


Figure 10 Example of Detailed Bridge

5.3.2 Cut Bridge and Culvert Approach

Large structures that do not create significant blockages to flow were removed from the terrain to simulate these as crossings that do not significantly impact flow conveyance. This approach was only applied in cases where the previously modelled flood elevations (from NVCA or LSRCA as appropriate) were well below the bridge or culvert soffit and/or where the bridge deck was relatively thin thereby not significantly contributing to backwater upstream.

A total of 84 crossings were modelled as cut. Of these, 70 were already cut from the LiDAR-derived DEM by PHB based on the width of the bridge/culvert observed during the LiDAR classification process. The remaining 14 were cut by Matrix using a similar approach based on aerial imagery. Spot checks were completed throughout the model domain to verify that the cutting reflected the bridge width, ensuring an appropriately sized opening was cut from the terrain.

5.3.3 Block Bridge and Culvert Approach

Small crossings that will not provide significant conveyance during flood events (specifically the Regional and 100-year events) were modelled as completely closed. This is a conservative approach and serves as a method to represent debris blockage during large events. A total of 74 crossings were modelled as blocked. Of these, 37 were already blocked in the LiDAR-derived DEM provided by PHB. The remaining 37 were blocked by Matrix using adjacent road elevations. Each blocked crossing was reviewed to ensure tie-ins were consistent with adjacent surroundings.

While this method is appropriately conservative for large storm events, Matrix recognizes that it is likely too conservative for smaller events. For example, it is far too conservative to assume that during a 5-year storm event, a small culvert will not provide any conveyance. This conservative assumption would be further exacerbated by the quasi-steady nature of the modelling. In order to reach steady state, the water elevations upstream of the structure would continue to rise until the structure is overtopped, even though there may not be enough volume of rainfall in a 5-year storm event for this overtopping to occur. For this reason, the 37 crossings that were blocked by Matrix were revised to a cut approach for the 5, 10, and 20-year simulations.

5.3.4 Future Recommendations for Cut and Blocked Crossings

As part of future studies, crossings modelled as cut or blocked in the 2D HEC-RAS model could be revised to be modelled in detail. This could be done in conjunction with the application of updated flows from NVCA or as needed when site-specific studies are initiated.

5.4 Boundary Conditions

The boundary conditions for the 2D HEC-RAS model were assigned based on existing site conditions. In a 2D HEC-RAS model, the edge of the mesh is assumed to be “closed” by default, which means that flow cannot exit the model unintentionally. A closed boundary was used for the majority of the study area. This is consistent with 2D standard modelling practice in which the 2D domain is typically set large enough to fully contain anticipated flood extents.

However, there are instances where fully containing flooding is not possible, such as where watercourses or other drainage features exit the model. In these cases, open boundary conditions are required to allow water to exit the system in an appropriate manner. Otherwise, unrealistic ponding may occur at the edge of the model, which can result in a misrepresentation of flooding. Following review of preliminary model runs, Matrix identified three locations where water was ponding at the edge of the model. In accordance with standard practices, the following three locations were updated to have open boundaries: unnamed tributary north of Wilson Drain in Alliston, the Nottawasaga River, and the West Holland River.

Where available, Regional storm flood elevations from existing HEC-RAS models were used to assign known water surface elevation boundary conditions for these locations. Since NVCA’s previous models only included the Regional event, a normal depth boundary condition was assigned for the design events based on the topographic slope at the boundary to permit water to exit the system naturally instead of ponding at the

edge of the model. The LSRCA HEC-RAS model was used to assign a rating curve of flow and water elevations for the design events and the Regional storm at the West Holland River boundary.

5.4.1 Beeton Flats Boundary Condition

The Beeton Flats area in the northeast portion of the study area has flat topography and contains the confluences of a number of watercourses. As previously mentioned, steady- and quasi-steady-state modelling do not consider attenuation effects due to storage. This means that features which could provide storage of water (i.e., wetlands, ponds, low-lying areas, areas behind large embankments, etc.) are assumed to fill and the time it takes and volume required to do so are not considered. Therefore, when reviewing flood extents based on quasi-steady modelling it is important to consider the total volume of observed flooding compared to the available volume of water in the system for that particular storm event. For instances where potential storage effects are significant, such as in Beeton Flats, the use of steady state calculations without accounting for attenuation can be unrealistically conservative. If the available storage volume is greater than the volume of rainfall for a given storm event, it is unreasonable to expect that the storage area will fill during the simulation and therefore steady state conditions cannot be met. At the end of the simulation if the modelled volume of water in the storage area is significantly greater than the available water in the system from a particular rainfall event, a review of storage attenuation would be required to determine whether steady state is reasonable. Unsteady state modelling using inflow hydrographs is one method to more accurately reflect flood extents under storage conditions. However, unsteady hydrographs were not available for use in this study and therefore alternate methods for confirming that the model reached an appropriate final water level was required as described below.

Matrix reviewed rainfall volumes for each storm event from the Ministry of Transportation (MTO) IDF [intensity-duration-frequency] Curve Lookup tool (MTO 2016). This tool allows for selecting a location on a map from which to extract up to date IDF parameters, rainfall intensities, and rainfall depths. Assuming a rainfall duration of 24 hours, the depths for each design event were extracted and are shown in Table 6. The volume of rainfall contributing to Beeton Flats was then calculated assuming a contributing drainage area of 472.2 km² (MacLaren 1988). The runoff volume was calculated assuming 75% of rainfall generates runoff and makes it to the Beeton Flats ponding area. The 75% runoff ratio is a best estimate at this time to represent infiltration, wetting loss, and other storage within the upstream watershed.

Next, to determine the available storage volume in Beeton Flats, an elevation-volume relationship was calculated in AutoCAD® using the LiDAR topography. Using this relationship, the anticipated water surface elevation was computed based on the estimated runoff volume for each design event. This water surface elevation was applied as a boundary condition rating curve at the eastern edge of Beeton Flats.

Table 6 Regional Storm Volume Check

Design Event	Rainfall Depth (mm) ⁽¹⁾	Rainfall Volume (m ³) ⁽²⁾	Estimated Runoff Volume (m ³) ⁽³⁾	Water Surface Elevation (m)
5-year	75.0	35,415,000	26,561,250	213.114
10-year	86.9	41,034,180	30,775,635	213.355
25-year	102.3	48,306,060	36,229,545	213.667
50-year	113.5	53,594,700	40,196,025	213.894
100-year	124.7	58,883,340	44,162,505	214.101

(1) Obtained from Ministry of Transportation Ontario IDF

[intensity-duration-frequency] Curve Online Lookup Tool (MTO 2016)

(2) Assuming a contributing drainage area to Beeton Flats of 472.2 km² (MacLaren 1988)

(3) Estimated runoff volume contributing to Beeton Flats assuming 75% of rainfall generates runoff

Following model simulations Matrix reviewed the resulting flow hydrographs at Beeton Flats in five locations (four inflow locations and one outflow location) and confirmed that the watercourses flowing into and out of the storage area had reached steady state. Matrix therefore determined that the boundary conditions applied at Beeton Flats appropriately represent the expected flooding conditions at the edge of the model as well as the features flowing into and out of Beeton Flats and results can be used with confidence.

6 Flood Risk Assessment

The 2D HEC-RAS model was run to establish existing flood conditions over a range of flow events including the 5-year, 10-year, 20-year, 50-year, 100-year, and Regional storms. Flood risk assessment map sets are provided in Appendix G1 and include two mapping products consisting of the following:

- Flood Inundation Mapping: 1) flood inundation extent, 2) maximum flood elevation

- Flood Hazard Mapping: 3) maximum depth, 4) maximum velocity, 5) maximum depth × velocity, 6) overall risk based on MNRFR risk categories

The flood inundation mapping was developed from the 2D HEC-RAS model results. The flood inundation extents is a line depicting the maximum lateral limits of flooding, while the maximum flood elevations illustrate a grid of maximum water elevations in each cell for each modelled event. Since the objective of this study is to review flood risk, and not to redefine the regulated floodplain, the flood inundation maps are for the purpose of understanding the resulting flood risk for the current study only and will not influence NVCA or LSRCA Regulatory mapping. Any development considerations will still refer to the Regulatory floodplain mapping available from NVCA and LSRCA for their respective watersheds.

The flood hazard mapping was developed with consideration of three risk factors: depth, velocity, and depth × velocity. The risk mapping criteria provided in Table 7 are based on current NVCA guidelines which refer to MNRFR criteria considering the risk of flooding as a threat to life based on average characteristics of a healthy adult (height, weight, etc.). These flood risk criteria were used to develop the flood risk mapping presented as Sheet 6 in each of the map sets in Appendix G1.

- **Low risk** includes areas that are inundated but where vehicular and pedestrian access and egress are still feasible.
- **Medium risk** areas do not permit vehicular access and egress due to water depths, but pedestrian access and egress by a healthy adult is possible.
- **High risk** areas do not facilitate safe access of any kind.

Table 7 Flood Risk Criteria

Risk Level	Low	Medium	High ⁽¹⁾
Depth	≤0.3 m	>0.3 m and ≤0.8 m	>0.8 m
Velocity	≤1.7 m/s	≤1.7 m/s	>1.7 m/s
Depth × Velocity	≤0.4 m ² /s	≤0.4 m ² /s	>0.4 m ² /s

(1) Exceedance of any one of the criteria results in high risk.

6.1 Understanding Two-dimensional HEC-RAS Results

The 2D HEC-RAS model creates spatial output covering the model domain and is capable of outputting resulting depth, velocity, depth × velocity, and water surface elevation, among other parameters. In a typical quasi-steady state simulation, the final time step represents steady state conditions, the time at which all depressions and storage areas are filled, flow has reached all the boundaries, and has stabilized at its maximum rate. The final model time step was used to obtain maximum depth, velocity, depth × velocity, and water surface elevation.

The modelling completed for the scope of this study is focussed on the riverine system; it does not include the urban drainage system and therefore the conveyance capacity of catch basins and storm sewers, storage provided by SWM facilities, local overland drainage paths, and other urban drainage components are not part of the 2D HEC-RAS modelling. As such, the modelled results presented in this report may not reflect the complete picture of flooding conditions in urban areas. These areas should be studied in more detail in the subsequent Phase 2 of the DMP study.

6.1.1 Modelling Limitation at Treetops Subdivision

The existing condition model results are not stable in the Treetops subdivision east of Alliston. Oscillations in flow are shown along residential streets, and the model results indicate that flooding occurs from the adjacent drainage channel that was constructed through the development. Several attempts were made to improve model stability including refining the mesh, incorporating more breaklines, and adjusting the computational time step. Despite these efforts, the model remained unstable in this area. Oscillations in water elevations continue to occur here and therefore the results are not considered to be accurate.

Due to the model size and the large cells required to cover the flat rural nature of the majority of the study area, the time step could not be reduced to a value that was low enough to achieve stability in the Treetops subdivision. After numerous iterations, Matrix concluded that the mesh size and details in the Treetops subdivision area were too different from the rest of the study area to be appropriately represented using the same model simulation parameters. Therefore, Matrix recommends that the Treetops subdivision area be considered in the urban drainage system modelling (e.g., PCSWMM) that will be completed during Phase 2 of the DMP.

6.2 Flood Characterization

Matrix completed characterization of existing riverine flood risk in the study area. Riverine flooding occurs when water levels of rivers, streams, and creeks rise and overflow their banks, spilling onto adjacent areas. Conservation authorities are responsible for determining the hazard from riverine flooding, while design and assessment of right-of-way conveyance and major overland flow paths fall within municipal responsibilities. The flood characterization considered the following potential sources and causes of riverine flooding:

- structure capacity issues (bridges and culverts)
- channel capacity issues (areas with constrictions, low points in banks, and lack of floodplain connectivity)
- backwater conditions

6.2.1 Summary of Existing Condition Results

Aside from the three urban settlement areas of Alliston, Beeton, and Tottenham, the study area is dominated by rural and agricultural land uses and a generally flat landscape. The lowlands of Beeton Flats in the northeastern portion of the study area create a large wetland area, also referred to as the Innisfil Creek Swamp. Model results for all storm events indicate that Beeton Flats creates a large inundated area categorized as a high flood risk due to the depths of ponding.

Similarly, along the western edge of the study area is an area referred to as the Bailey Creek Swamp. Model results in this area indicate widespread flooding starting at the 5-year modelled event; however, the flooded areas do not become high risk (due to depth) until the 100-year modelled event.

6.2.2 Alliston

There are three watercourses that flow through Alliston: Wilson Drain in the north; Boyne River through the central area; and Spring Creek to the south. There is some low- to medium-risk flooding along Wilson Drain upstream (west) of King Street North during all modelled storm events. As well, a large inundated area upstream of the CPR is shown to occur during the modelled Regional event, likely caused by backwater from the CPR culvert as shown in Figure 11 (refer to Ex1.1 to Ex1.6 in Appendix G1 for larger context area). Urban drainage improvements and development occurred here in response to flooding issues along Wilson Drain (Stantec 2008). However, given that the modelling for this project focussed on the riverine system, Matrix recommends that

urban drainage system improvements in this area be incorporated into a detailed urban drainage model during Phase 2 of the DMP.

No significant high-risk flood areas were observed outside the natural floodplain extents along Boyne River. One exception to this is at the Alliston Wastewater Treatment Plant located on the south bank of Boyne River just upstream of Sir Frederick Banting Road. This location was identified as a high flood risk area and is discussed in more detail in Section 9.2.1.

Flooding along Spring Creek is generally contained within the floodplain for all modelled events with the exception of the reach between King Street South and Dufferin Street South where flooding extends beyond the floodplain into residential neighbourhoods. NVCA identified a large number of flood vulnerable buildings within Alliston adjacent to Spring Creek and therefore was identified as an area of concern (NVCA 2018). The results of the current study do not replicate this level of vulnerability. This area was identified as a flood risk area and is discussed in more detail in Section 9.3.1.

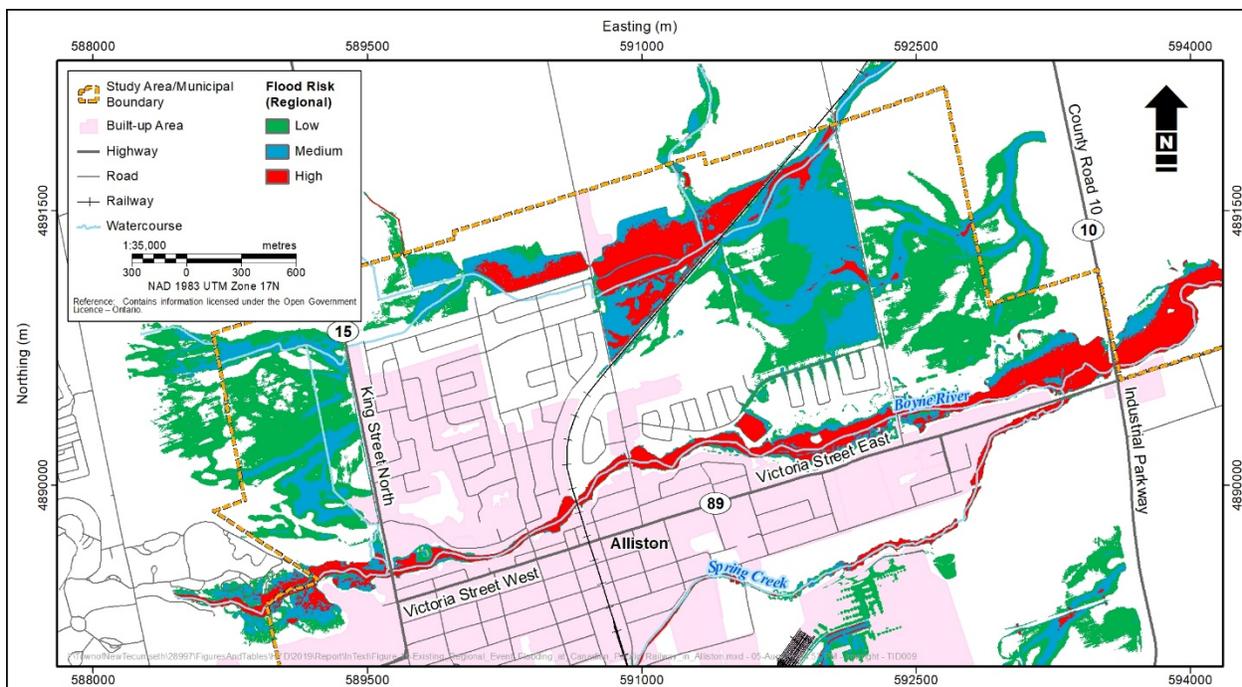


Figure 11 Existing Regional Event Flooding at Canadian Pacific Railway in Alliston

Widespread flooding with some areas of high risk is also shown south of Industrial Parkway as a result of riverine flow from Upper Nottawasaga Creek extending beyond its floodplain (refer to flood risk area discussions in Section 9.1.2 and 9.2.2). Matrix

recommends that urban drainage system components in Alliston be modelled in more detail during Phase 2 of the DMP.

6.2.3 Beeton

Of the three urban settlement areas in the Town's jurisdiction, Beeton was the most impacted by the major storm event of June 2017. Beeton Creek flows from north to south along the western edge of the community of Beeton; Hendrie Drain flows northerly through the centre of the community and joins Beeton Creek just north of Lilly Street East; and Baker Drain flows from south to north along the eastern edge of Beeton, eventually draining to Beeton Flats.

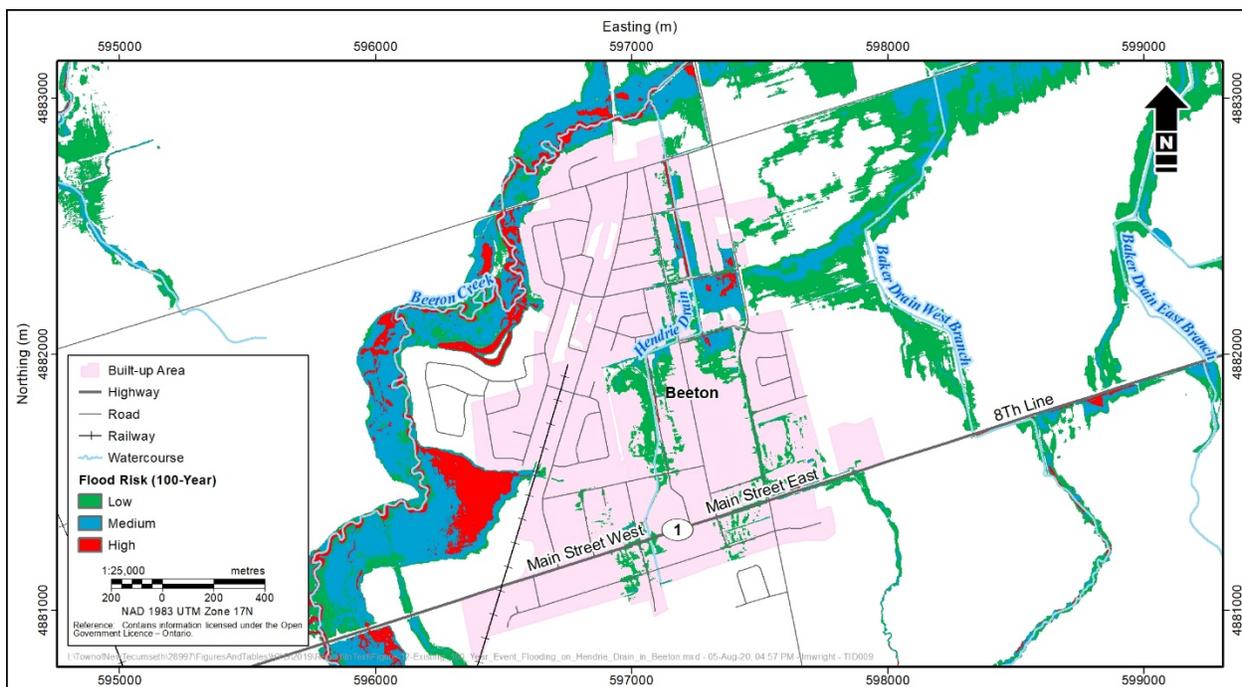


Figure 12 Existing 100-year Event Flooding on Hendrie Drain in Beeton

Significant widespread flooding in Beeton was observed following the June 2017 event. In addition, there have been a number of flooding complaints by residents living near Hendrie Drain and subsequent maintenance and cleanup work were completed to improve drainage (R.J. Burnside 2009).

Flooding is shown through the residential areas of Beeton (refer to Section 9.3.2) starting at the 5-year modelled event with high-risk flood depths occurring starting at the 100-year modelled event as shown in Figure 12 (refer to Ex2.1 to Ex2.6 in Appendix G1 for larger context area). As with Alliston, urban drainage improvements and developments have occurred in Beeton and therefore Matrix recommends that these

urban drainage system components be modelled in more detail using an appropriate model in Phase 2 of the DMP.

6.2.3.1 Tottenham

The community of Tottenham is surrounded by the main branch of Beeton Creek to the west and north and a tributary to Beeton Creek to the east. Tottenham Dam is located along the main branch of Beeton Creek just south of Mill Street/4th Line in Tottenham. The June 2017 storm event caused excessive damage to the emergency spillway of Tottenham Dam including erosion and failure of the concrete and gabion basket retaining wall and therefore repairs were required.

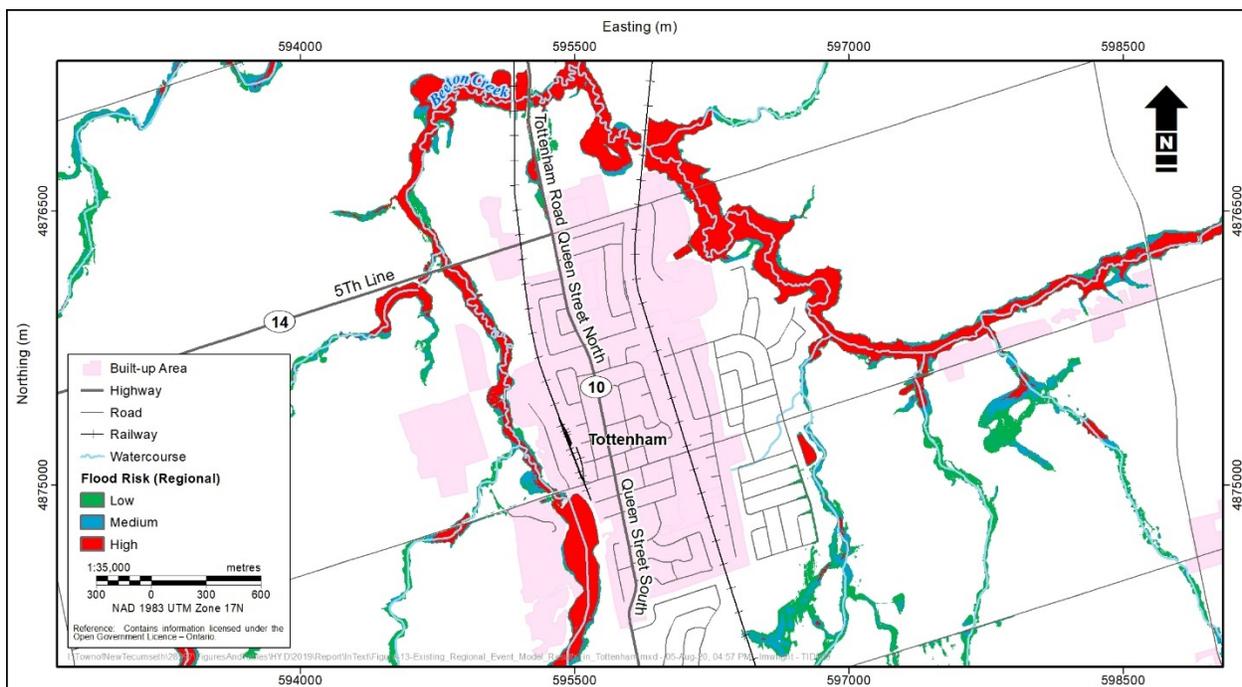


Figure 13 Existing Regional Event Model Results in Tottenham

The modelled results do not indicate flooding through the community of Tottenham in the Regional event (Figure 13); however, the dam failure was not incorporated into the model. To understand the potential flood risk to Tottenham resulting from dam failure, the terrain in the 2D HEC-RAS model could be adjusted in future studies to reflect removal of the dam.

6.2.3.2 Railway Crossings

The CPR and South Simcoe Railway (SSR) lines passing through the study area create barriers to flood flows along Wilson Drain, Upper Nottawasaga Creek, Bailey Creek, and Beeton Creek. In our experience, railway embankments are often found to create

hydraulic restrictions leading to increased flood hazard areas. This is likely due to the era during which the railways were constructed in which there were minimal standards related to culvert design. Additionally, there are potential complications related to mitigation measures on railway lands. Construction of mitigation measures may require disruption to rail service, which is typically unacceptable from the perspective of the railway company. Therefore, it is important that potential flooding issues associated with railways are identified as early as possible to initiate coordination.

The modelled results show backwater at the five railway culverts identified in Table 8. Overtopping was not shown in any of the modelled events at these structures. MSI_41 and MSI_61 indicate surcharge during the Regional event only; therefore, mitigation measures have not been provided as these would meet hydraulic design guidelines for culverts on a railway crossing. MSI_120 surcharges during the 10-year event and contributes high risk flooding on Tottenham Road at 10th Line. Detailed discussion of next steps for flood mitigation at this location is provided in Section 9.1.6.

Table 8 Railway Crossings with Backwater

Structure ID	River	Road Name	Road Class	Storm Event Causing Surcharge	Storm Event Causing Overtopping
MSI_19 ⁽¹⁾	Beeton Tributary	CPR, north of Highway 9	Rail	-	-
MSI_41	Beeton Creek	CPR, north of 5 th Line	Rail	Regional	N/A
MSI_34 ⁽²⁾	Beeton Creek	CPR/SSR, south of 7 th Line	Rail	-	-
MSI_120	Bailey Tributary	CPR, north of 9 th Line	Rail	10-year	N/A
MSI_61	Wilson Drain	CPR, east of Boyne Street	Rail	Regional	N/A

(1) MSI_19 modelled using the “block” approach due to its small size. Surcharge and overtopping info not available.

(2) MSI_34 modelled using the “cut” approach due to its large size. Surcharge and overtopping info not available.

CPR - Canadian Pacific Railway

SSR - South Simcoe Railway

6.2.3.3 Berms Along Channels

As discussed in Section 5.2.2 breaklines were added throughout the model to represent significant features and changes in elevation. During the model development, Matrix identified some watercourses that have berm-like features along the top of bank which separate the watercourses from their floodplains. Consistent with current MNRF practice for floodplain mapping, berms are not considered permanent flood control features meaning they are assumed to fail under large storm events. For this reason, breaklines were not specifically included along berms as they are assumed to fail under the modelled storm events in accordance with the *Technical Guide – River and Stream Systems: Flooding Hazard Limit* (MNR 2002). Not including breaklines on these features prevents cell faces from occurring along them and therefore allows flow to pass through.

Large berms are naturally reflected in the topographic surface due to their size, an example of Baker Drain East Branch is shown in Figure 14. These berms not only restrict channel capacity and isolate the channel from its floodplain, but they also prevent water in the floodplain from re-entering the channel, which leads to isolated pockets of flooding with no outlet. While these berms may have been constructed by local landowners to prevent frequent nuisance flooding, they may exacerbate flood risk in larger storm events, as demonstrated in the example below. Water is shown to be trapped on the west side of the berm and is unable to flow into the channel. Through consultation with the Town, Matrix understands that this area is currently under a development application. It is recommended that the Town require the developer to review the hydraulics in the area in a detailed study. Further discussion of potential next steps for flood mitigation at this location is provided in Section 6.3.3.

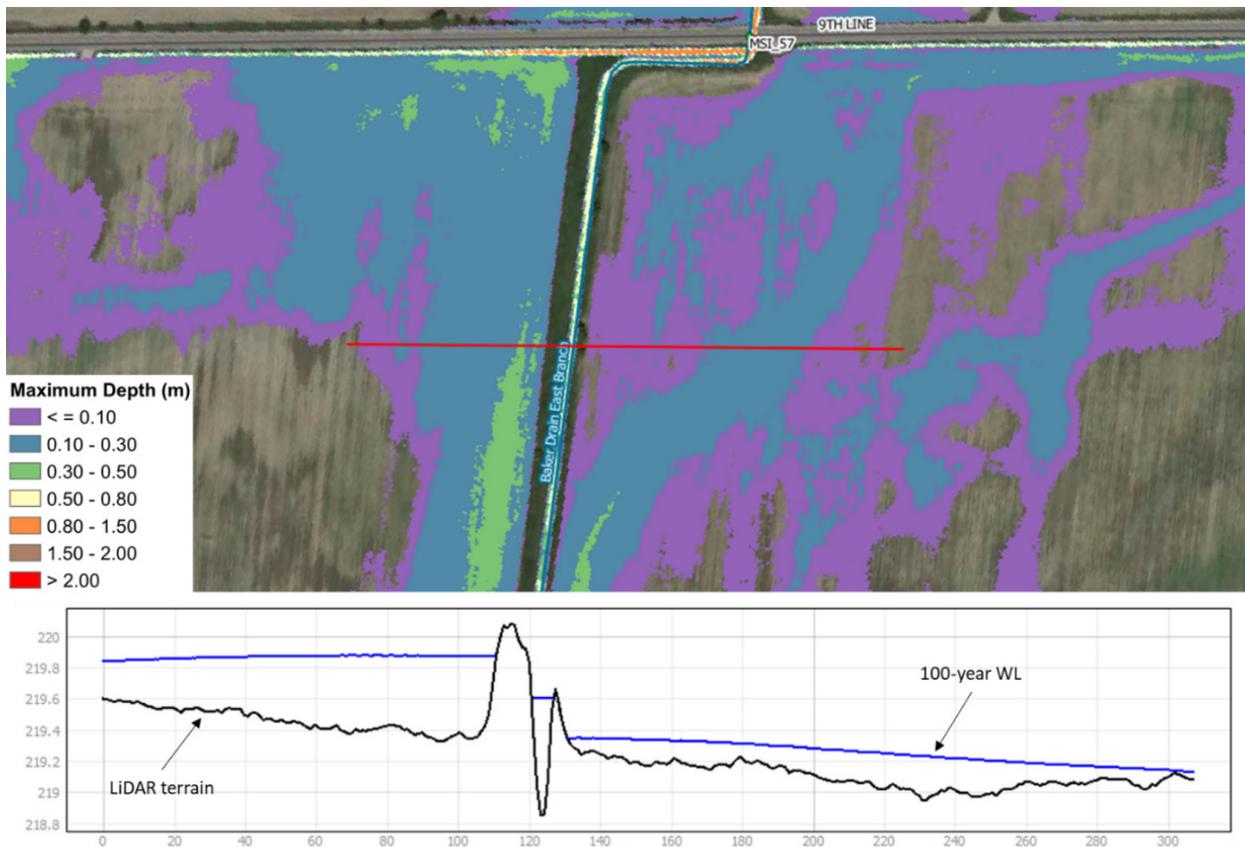


Figure 14 Example of Berms along Baker Drain East Branch

6.2.4 Hydraulic Constraints

Matrix reviewed structure capacity to identify potential sources of riverine flooding within the study area. Table 9 provides details of structure capacities, soffit elevations, and storm events leading to surcharge and overtopping at each structure. A structure is considered surcharged when the water surface elevation reaches the obvert/soffit, while overtopping is when the water surface elevation exceeds the low point of the road. For reference, the Town's design standards specify that all road crossings and driveway culvert be sized to convey the 25-year storm (Town of New Tecumseth 2005).

Note that Table 9 assesses the detailed modelled structures only; the assessment does not include culverts modelled through cut and block techniques. Additionally, some detailed culverts are excluded from the table where the model is not stable and further assessment is recommended (e.g., MSI_72 and MSI_73 in the Treetops neighbourhood).

Table 9 Bridge and Culvert Capacity

Row No	Structure ID	River	Road Name	Road Class	Storm Event Causing Surcharge	Storm Event Causing Overtopping ⁽¹⁾
1	MSI_63	Bailey Creek	CPR	Rail	N/A	N/A
2	MSI_120	Bailey Tributary	CPR	Rail	10-year	N/A
3	12032	Beeton Creek	10 th Sideroad	Collector	N/A	N/A
4	12033	Beeton Creek	9 th Line	Collector	50-year	Regional
5	12039	Beeton Creek	6 th Line	Local/Street	Regional	N/A
6	12041	Beeton Creek	Mill Street	Collector	Regional	Regional
7	12063	Beeton Creek	Old Railway	Rail	5-year	N/A
8	MSI_08	Beeton Creek	SSR	Rail	Regional	N/A
9	MSI_141	Beeton Creek	Pond	N/A	5-year	Regional
10	MSI_47	Beeton Creek	SSR	Rail	N/A	N/A
11	MSI_48	Beeton Creek	Tottenham Road	Arterial	Regional	N/A
12	MSI_62	Beeton Creek	5 th Line	Arterial	Regional	Regional
13	12044	Beeton Tributary	5 th Line	Local/Street	50-year	Regional
14	12045	Beeton Tributary	4 th Line	Local/Street	N/A	N/A
15	MSI_16	Beeton Tributary	Tecumseth Pines Drive	Local/Street	Regional	Regional
16	MSI_41	Beeton Tributary	CPR	Rail	Regional	N/A
17	MSI_50	Beeton Tributary	5 th Line	Arterial	Regional	N/A
18	MSI_55	Beeton Tributary	3 rd Line	Local/Street	N/A	N/A
19	11002	Boyne River	Boyne Street	Local/Street	N/A	N/A
20	11003	Boyne River	Sir Frederick Banting Road	Local/Street	100-year	N/A

Row No	Structure ID	River	Road Name	Road Class	Storm Event Causing Surcharge	Storm Event Causing Overtopping ⁽¹⁾
21	11011	Boyne River	Trail	Trail	N/A	N/A
22	12012	Cookstown Creek	14 th Line	Local/Street	Regional	Regional
23	MSI_132	Cookstown Tributary	14 th Line	Local/Street	5-year	N/A
24	MSI_133	Cookstown Tributary	14 th Line	Local/Street	50-year	Regional
25	MSI_99	Cookstown Tributary	20 th Sideroad	Local/Street	50-year	Regional
26	12053	Hendrie Drain	English Drive	Local/Street	Regional	N/A
27	12054	Hendrie Drain	Centre Street	Local/Street	N/A	N/A
28	12052-1-2	Hendrie Drain	Lilly Street	Collector	Regional	N/A
29	12052-2-2	Hendrie Drain	Lilly Street	Collector	10-year	N/A
30	MSI_22	Hendrie Drain	Prospect Street	Local/Street	5-year	20-year
31	MSI_45	Hendrie Drain	Main Street	Arterial	50-year	20-year**
32	MSI_74	Hendrie Drain	Stewart Street	Local/Street	50-year	N/A
33	MSI_9	Hendrie Drain	Maple Avenue	Local/Street	Regional	N/A
34	MSI_17	Innisfil Tributary	15 th Sideroad	Arterial	Regional	Regional
35	MSI_92	Innisfil Tributary	10 th Sideroad	Collector	50-year	N/A
36	12019	Penville Drain	5 th Line	Local/Street	Regional	N/A
37	MSI_95	Penville Drain	20 th Sideroad	Local/Street	100-year	Regional
38	MSI_94	Penville Tributary	20 th Sideroad	Local/Street	20-year	N/A
39	12050	Schomberg River	20 th Sideroad	Local/Street	Regional	N/A
40	12051	Schomberg River	17 th Sideroad	Local/Street	Regional	N/A
41	MSI_77	Schomberg River	15 th Sideroad	Arterial	Regional	Regional

Row No	Structure ID	River	Road Name	Road Class	Storm Event Causing Surcharge	Storm Event Causing Overtopping ⁽¹⁾
42	MSI_79	Schomberg River	Western Avenue	Arterial	Regional	N/A
43	MSI_68	Schomberg Tributary	Highway 9	Highway	N/A	N/A
44	11004	Spring Creek	Dufferin Street South	Local/Street	100-year	N/A
45	11005	Spring Creek	Beattie Avenue	Local/Street	Regional	Regional*
46	11006	Spring Creek	King Street South	Collector	Regional	Regional
47	11007	Spring Creek	8 th Avenue	Local/Street	Regional	Regional*
48	11008	Spring Creek	Church Street South	Local/Street	N/A	N/A
49	11009	Spring Creek	Industrial Parkway	Arterial	Regional	N/A
50	11010	Spring Creek	Walking Path	Trail	N/A	N/A
51	12066	Spring Creek	Parsons Road	Local/Street	Regional	N/A
52	MSI_40	Spring Creek	CPR	Rail	100-year	N/A
53	MSI_71	Spring Creek	Trail	Trail	Regional	N/A
54	MSI_78	Spring Creek	Victoria Street	Arterial	20-year	Regional*
55	12067	Treetops Tributary	10 th Sideroad	Collector	10-year	Regional
56	MSI_33	Treetops Tributary	10 th Sideroad	Collector	N/A	N/A
57	MSI_44	Treetops Tributary	Sutcliffe Way	Local/Street	Regional	Regional
58	MSI_52B	Treetops Tributary	Riverview Road	Local/Street	20-year	N/A
59	MSI_52C	Treetops Tributary	Riverview Road	Local/Street	N/A	N/A
60	12068	Wilson Drain	Boyne Street	Local/Street	Regional	N/A
61	12069	Wilson Drain	Boyne Street	Local/Street	N/A	N/A
62	MSI_56	Wilson Drain	Sir Frederick Banting Road	Local/Street	5-year	N/A

Row No	Structure ID	River	Road Name	Road Class	Storm Event Causing Surcharge	Storm Event Causing Overtopping ⁽¹⁾
63	MSI_61	Wilson Drain	Rail Line	Rail	Regional	Regional**

(1) In some instances (denoted with a *), the road is overtopped because the water elevation of a storm event is higher than the low point, despite that water level being equal to or lower than the top of road elevation at the structure. Further, in one instance (denoted with a **), the obvert elevation is higher than the low point in the road profile and therefore road overtopping occurs but the structure is not surcharged.

CPR - Canadian Pacific Railway

SSR - South Simcoe Railway

N/A - no surcharge/overtopping observed for any of the modelled events

The assessment of bridge and culvert capacity indicates many bridges and culverts are surcharged during the design storms and over 75% are surcharged during the Regional event. Table 10 summarizes the overall hydraulic performance of detailed structures within the study area. One third of the bridges and culverts modelled using the detailed approach are overtopped during the Regional event.

Two culverts, located along the Hendrie Drain on Prospect Street and Main Street, are overtopped during the 20-year modelled event. The Prospect Street and Second Street crossing (total length: 236 m, flow area: 1.62 m²) and Main Street crossing (length: 135 m long, flow area: 1.77 m²) are both long, buried culverts which are undersized. The Town is aware of this issue and indicated that there were design and constructability constraints related to elevation and available space at these locations. Although they are undersized, these structures are close to meeting the Town's 25-year design standard (Town of New Tecumseth 2005).

Table 10 Summary of Detailed Bridge and Culvert Hydraulic Performance

	5-yea r	10-yea r	20-yea r	50-yea r	100-yea r	Region al
Surcharged Structures (count)	5	8	12	19	23	49
Surcharged Structures (%)	8%	13%	19%	30%	37%	78%
Overtopped Structures (count)	0	0	2	2	2	21
Overtopped Structures (%)	0%	0%	3%	3%	3%	33%

Longitudinal profile plots of several key reaches (Beeton Creek, Bailey Creek, Penville Creek, Schomberg River, Spring Creek, and Boyne River) are presented in Appendix G2. The water surface elevations at bridges and culverts for all modelled events were extracted from the model and are summarized in Appendix G3. The approximate location and height of the bridges and culverts were indicated along each reach. The modelled structure type (detailed, cut, and block) is indicated by colour. These plots provide further insight into which bridges and culverts may be causing backwater.

Bailey Creek: Structures along Bailey Creek are a mix of detailed, cut, and blocked representation. While modelling shows high water elevations at the Tottenham Road and CPR, the Regional water elevations do not approach the top of road elevation for any of the structures and therefore overtopping does not occur.

Beeton Creek: Beeton Creek flows through the urban settlement areas of Tottenham and Beeton and therefore structures along Beeton Creek used a mix of both detailed and cut representation. The Tottenham Dam, characterized by its flat bottom and notable drop at the downstream end, is shown on the Beeton Creek profile at a horizontal distance (x-axis) value of 21,000 m. In Tottenham, Mill Street and 5th Line, the ice structure culverts, and Tecumseth Heights Drive are shown to overtop during the Regional event. The 9th Line bridge in Beeton is shown to surcharge at the 50-year event and overtops during the Regional event. 7th Line is also overtopped during the Regional event.

Boyne River: Structures along Boyne River are a mix of both detailed and cut representation. While an increase in water elevation is observed at the trail crossing upstream of Church Street in Alliston, the Regional water elevation do not approach the top of road elevation for any of the structures and therefore overtopping does not occur.

Nottawasaga Creek: There are no detailed bridges or culverts modelled along the Nottawasaga Creek. All identified structures were modelled as cut. The 13th Line and 14th Line bridges are significantly inundated and overtopped during the Regional event.

Penville Creek: There are no detailed bridges or culverts modelled along the main branch of Penville Creek. All identified bridges and culverts were modelled as either cut or blocked. The 10th Line and 11th Line bridges are overtopped during the Regional event. The bridge at 11th Line is in a backwater condition under the Regional storm due to ponding in Beeton Flats.

Schomberg River: Structures along Schomberg River are a mix of detailed and cut representation. During the Regional event, 15th Sideroad is overtopped, and 17th Sideroad and 20th Sideroad are surcharged.

Spring Creek: All structures along Spring Creek, which flows through Alliston, are represented as detailed structures. Under the Regional event, all structures upstream of Dufferin Street (with the exception of CPR) are at or near overtopping. Backwater starts at the 5-year event at both Parsons Road and the CPR crossing despite the bridges having capacity for much larger storms; this is due to the contraction of the river into the structure. Further downstream, significant backwater is observed at Victoria Street and Tottenham Road for the Regional event as Victoria Street is overtopped.

6.3 Spill Assessment

A floodplain spill is an area where flooding is not physically contained within the channel corridor or its floodplain and therefore flows into adjacent watersheds or subwatersheds.

Spills can occur naturally (i.e., low channel banks or poorly defined floodplains) or as a result of physical barriers that restrict flow and cause elevated water elevations upstream (i.e., bridge/culvert or channel restrictions, berms, or other barriers). An understanding of spill extents, depths, and flow rates are required to accurately define flood risk for all receiving watercourses.

Matrix completed a spill assessment to confirm floodplain connectivity between adjacent watersheds and subwatersheds under various flow events. Three spill areas were identified and are discussed in the following sections. A summary is provided in Table 11.

Table 11 Summary of Spill Assessment

Spill Location	Storm Event at which Spill Occurs
Upper Nottawasaga to Boyne River	5-year
Beeton Creek to Bailey Creek	Regional
Baker Drain to Beeton Creek	Regional

6.3.1 Spill from Upper Nottawasaga River to Boyne River Subwatershed

Spill from the Upper Nottawasaga River into the Boyne River subwatershed occurs during all modelled storm events. The maximum rate of spill for each modelled event is summarized in Table 12.

Table 12 Modelled Spill Rates from Upper Nottawasaga River to Boyne River Subwatershed

Modelled Event	Maximum Spill Rate (m ³ /s)
5-year	0.8
10-year	1.1
20-year	1.3
50-year	1.7
100-year	1.9
Regional	5.4

For lack of readily available data, the subwatershed boundaries considered for the purpose of the spill assessment were obtained from Figure 3.1-1 of the *Integrated Watershed Management Plan Characterization Report* (Ecosystem Recovery Inc. 2018). These were overlaid with the existing condition Regional modelled flood risk results as shown in Figure 15. The spill occurs along the delineated subcatchment boundary in the location identified as point 221. Beyond the 10-year modelled event, flow overtops CPR and continues into the Boyne River watershed.

However, the watershed boundaries do not align with modelled flow path suggesting that the area to the east of CPR may in fact be part of the Middle Nottawasaga River subcatchment. This would mean there is no spill occurring between different watersheds since the Upper Nottawasaga and Middle Nottawasaga subcatchments are part of the same watershed. Matrix suggests that, in consultation with NVCA, the subsequent phase of the DMP should include review of major system subcatchment delineation using the updated LiDAR topography.

Additionally, Matrix recommends reviewing the required capacity for a potential new culvert under CPR along the primary overland flow path south of Industrial Parkway shown on Figure 15 and discussed further in Section 9.1.2. We recognize that construction of a culvert under a railway would require additional study and stakeholder interaction and therefore the culvert was identified at this screening level stage for future consideration by the Town.

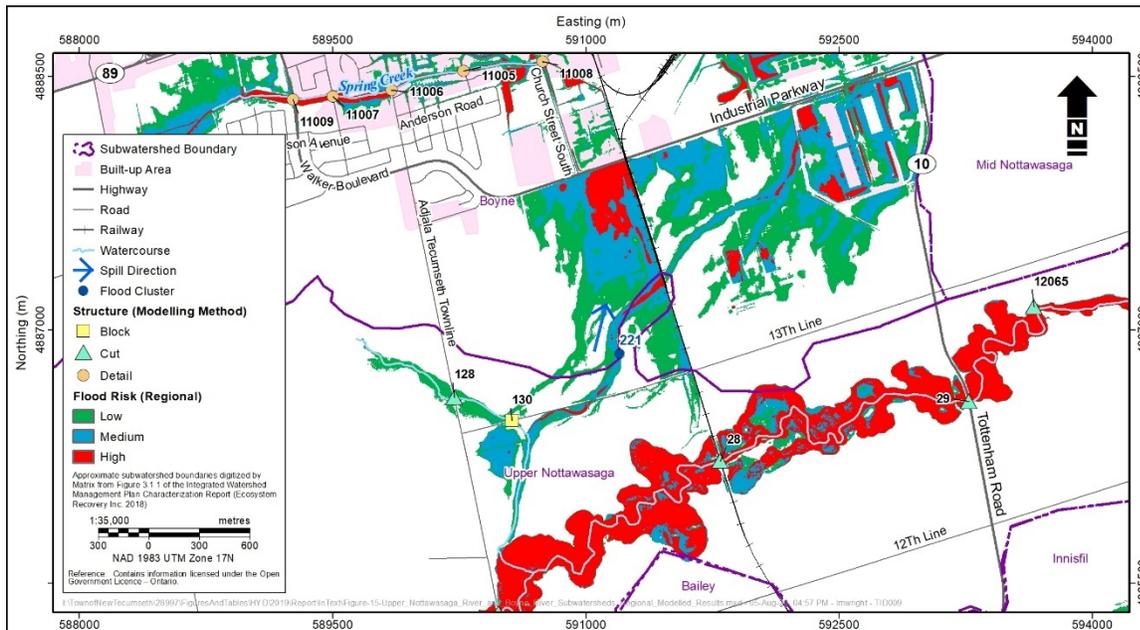


Figure 15 Upper Nottawasaga River and Boyne River Subwatersheds (100-year Event)

6.3.2 Spill from Beeton Creek to Bailey Creek Subwatershed

Spill from Beeton Creek into the Bailey Creek subwatershed was shown during the modelled Regional event only with a maximum flow rate of 44.3 m³/s. Structure 12032 on 10th Sideroad creates backwater during the Regional event to an elevation of approximately 221.0 m on the upstream side. The spill point between Beeton Creek and Bailey Creek between the Trans Canada Trail and 10th Sideroad is at an elevation of approximately 220.7 m; the road elevation at Structure 12032 is approximately 222.8 m, which is higher than the spill point to Bailey Creek. Flows above the spill elevation cross the subwatershed boundary to Bailey Creek at the location identified as point 222 (this occurs during the modelled Regional event only). When backwater occurs at the culvert, water will spill toward Bailey Creek before overtopping Sideroad 10. These spill elevations, in addition to the fact that flow is confined between two embankments, contributes to why the spill rate from Beeton Creek to Bailey Creek is so high.

Upstream of this, Structure 12063 conveys flow under an old portion of the SSR line that was abandoned and now services the Trans Canada Trail. The old bridge was removed and replaced with a new pedestrian bridge at a higher elevation. In this study the hydraulic impact of the pedestrian bridge was assumed to be negligible and therefore was modelled using the “cut” approach. In the future phase of the DMP the model should be updated to model this bridge using the “detail” approach to enable a more accurate understanding of potential spill conditions. The bridge and embankment create backwater forcing water to travel north and eventually overtopping the trail at the 50-year modelled event. This overtopped flow is then confined between the embankments created by the trail and 10th Sideroad.

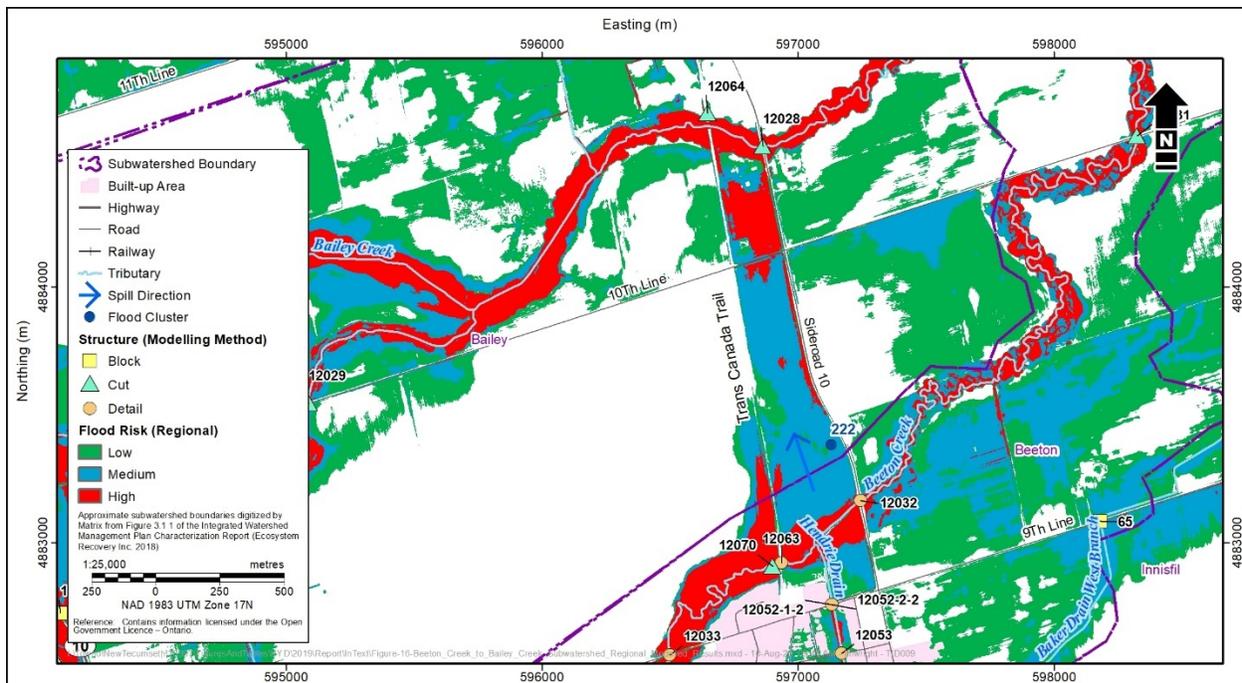


Figure 16 Beeton Creek to Bailey Creek Subwatershed (Regional Event)

The model results indicate that the existing bridges on Bailey Creek (Structures 12064 and 12028) adequately convey the modelled 20-year event and thus meet the Town’s design guidelines. However, they are not adequately sized to convey the additional spill contribution from Beeton Creek, and therefore crossing improvements to reduce water elevations below the spill elevation may be beneficial. Matrix recommends that detailed review of structures 12063 (new pedestrian bridge) and 12032 be completed in subsequent studies including structure capacity improvements, consideration for improved ditching along 10th Sideroad, and potential regrading of the Trans Canada Trail.

6.3.3 Spill from Baker Drain to Beeton Creek Subwatershed

Spill from Baker Drain in the Innisfil Creek subwatershed to the Beeton Creek subwatershed was shown at the modelled Regional event only with a maximum flow rate of 1.3 m³/s. The 2D HEC-RAS model results show that the limited capacity of Baker Drain West Branch immediately downstream of 9th Line is the primary factor contributing to the spill (refer to Figure 17). Matrix recommends that the channel capacity in this location is reviewed in subsequent studies to determine whether capacity improvements would alleviate flooding conditions and mitigate spill from Baker Drain.

Matrix understands that the lands south of 9th Line are currently under a development application. In coordination with the development application it is recommended that the

Town require the developer to complete a detailed study of hydrology and hydraulics in this area. The analysis should also consider the impact of berm removal along the west branch of Baker Drain as mentioned in Section 6.2.1.5 to confirm that development will not increase flooding or the occurrence of spill.

- complete cross-section survey along Baker Drain West Branch to confirm bathymetry
- determine existing condition drainage area and peak flows contributing to Baker Drain
- calculate peak flows under proposed development conditions
- confirm existing hydraulic performance of the Baker Drain West Branch channel and 9th Line crossing (Structure MSI_65) through development of a site-specific hydraulic model or trimming the 2D HEC-RAS model to an appropriate extent
- quantify spill rates to Beeton Creek subwatershed under existing and future development conditions
- review potential flood reduction opportunities such as increasing channel capacity, removal of berms, and restoring floodplain connectivity to determine the impact that changes will have on flooding within the development lands as well as flooding along 9th Line and spill to Beeton Creek

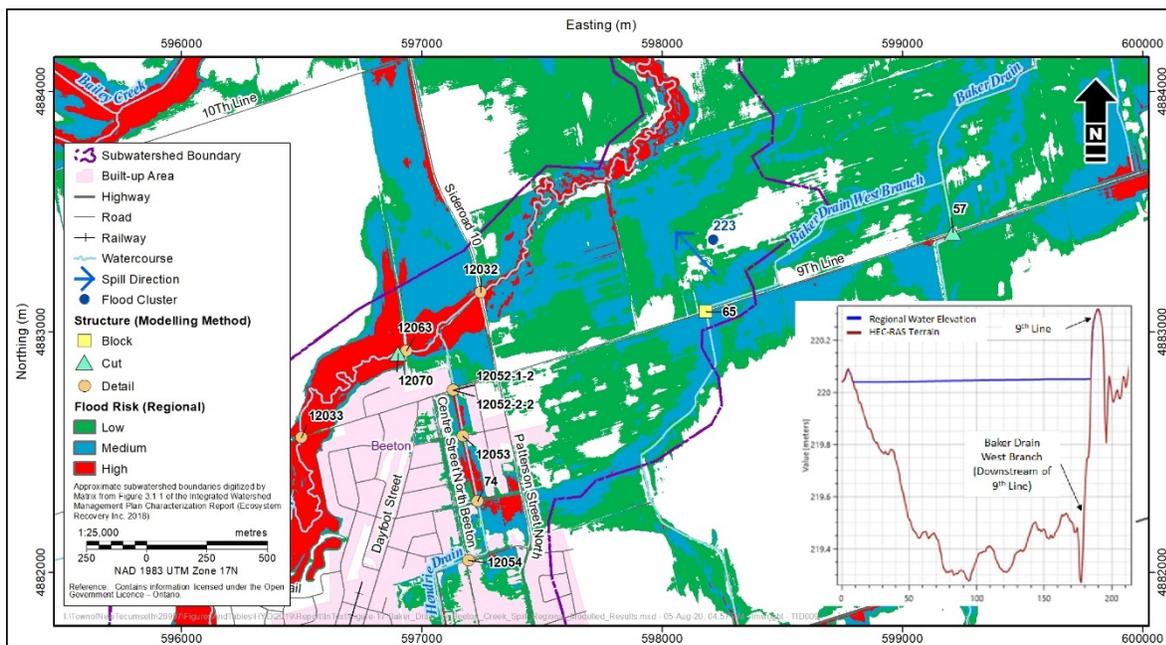


Figure 17 Baker Drain to Beeton Creek Spill (Regional Event)

7 Sensitivity Analysis

Resiliency is an important factor in flood protection, particularly due to the amount of uncertainty in climate as well as limitations of hydrologic and hydraulic modelling. The

scope of work for this study includes two sensitivity analysis scenarios aimed at assessing the impact that climate change and various developments have on the resulting flood extents and flood risk.

7.1 Climate Change

The Government of Ontario has recognized the need to examine current floodplain management methodologies in light of climate change. The 2020 Provincial Policy Statement (MMAH 2020) has articulated this need at the policy level. The MNRF has acknowledged the need to evaluate the 2002 Flood Hazard Technical Guide for municipalities, conservation authorities, and others to integrate consideration of climate change at an operational level (MNRF 2017). Therefore, in addition to typical flood risk assessments, this project included assessment of resiliency to climate change. An approach detailed here was put forward by Matrix to account for climate change in return frequency events to assess flood risk resiliency to climate change within the Town's jurisdiction.

Generally, General Circulation Models (GCMs; i.e., the global climate change models) are used for predicting future temperature forecasts, but they typically do a poor job of predicting event-scale rainfall. There are applications for predicting future intensity-duration-frequency (IDF) curves based on these GCMs, however, the GCMs and their inherent assumptions remain a large source of uncertainty.

The Clausius-Clapeyron relationship states that with increased air temperature there is an increased capacity for water vapour in the air. This relationship can be used to determine future rainfall potential. High intensity summer storm cells do not retain moisture in the atmosphere, but rather precipitate out all the available water vapour (Allen and Ingram 2002; Westra et al. 2014). Therefore, for consideration of design storms, a direct relationship between temperature and rainfall was established to be a 7% increase in rainfall per degree Celsius (Panthou et al. 2014). Using the GCM-predicted temperature increase for Southern Ontario (to 2050) of 3.1°C, this translates to a 22% increase in design event rainfall.

To determine the impact that increased rainfall would have on the resulting runoff, rerunning hydrologic models would be required. Considering the scope of this project and the lack of appropriate hydrologic models in this area, the climate change scenario was based on the results of previous studies. For rural catchment areas located in Southern Ontario, much like the current study area, the 22% increase in rainfall can be translated into 33% increase in runoff.

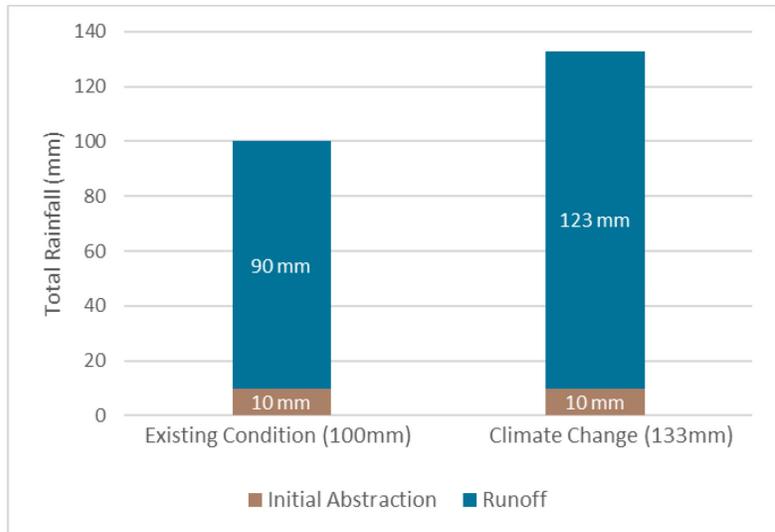


Figure 18 Conceptual Illustration of Climate Change Adjusted Rainfall and Resulting Runoff

The increase in runoff is greater than the increase in rainfall because the catchment initial abstraction does not change between the two climate change scenarios. Initial abstraction is a hydrologic parameter that accounts for the amount of water that can be stored in the soil and vegetation during a storm before runoff occurs. Initial abstraction values are fully utilized before runoff occurs; therefore, a larger portion of the rainfall is converted to runoff during the climate change event. This concept is illustrated in Figure 18.

Figure 19 shows an example location (Innisfil Creek at the eastern edge of the study limit) where flows were increased by 33% to account for climate change. These examples demonstrate that climate change will increase the frequency of each design event. For example, the existing 10-year flow has a 1 in 10 (or 10%) probability of occurring in a given year. The climate-adjusted 10-year flow increased to 29.0 m³/s which falls between the existing 20-year and 50-year return periods and would therefore have somewhere between a 1 in 20 (or 5%) and a 1 in 50 (or 2%) chance of occurring in any given year. Conversely, under existing climate conditions, a flow event with a value of 29.0 m³/s is understood to have a 2% to 5% chance of occurring in any given year; however, under the climate-adjusted scenario, this flow value is understood to have a larger probability, 10%, of occurring in a given year. This means is that the likelihood of each storm occurring has increased due to climate change. The corresponding return period for climate change flows beyond the existing 100-year flow is not provided due to relative uncertainty of extrapolation. However, this trend

continues throughout the set of return frequency events (refer to Appendix H1). For this reason, the smaller return frequency events were not rerun using the climate-adjusted flows as the results would be very similar to the risk level shown in runs already completed.

This project also includes simulating Regional storm peak flows (Timmins Storm applied in NVCA jurisdiction and Hurricane Hazel applied in LSRCA jurisdiction). The Timmins Storm and Hurricane Hazel are historical storms of record that were not derived based on statistical analyses and therefore the Regional flow was not adjusted for climate change. MNR guidelines indicate that the Regulatory flood limit is defined by the larger of the Regional storm or the 100-year inundation boundary (MNR 2002).

In some places, climate change may result in the 100-year event becoming the new Regulatory event. An assessment of the climate-adjusted 100-year +33% scenario results (herein referred to as the climate change scenario) throughout the study area shows that this condition could occur within the study area.

The climate change scenario was simulated in the 2D HEC-RAS model to confirm the flood extents relative to the to Regional event. Review of the results shows that this variability occurs throughout the Town's jurisdiction. The climate-adjusted 100-year flood levels remain lower than the Regional event flood levels throughout most of the study area. However, the climate change scenario produces flows higher than the Regional event on the Boyne River at the western limit of the study area resulting in slightly higher flood levels from the western limit of the study area to County Road 10, at the confluence with Spring Creek. Matrix completed a thorough review of the inundation boundary resulting from the climate change scenario. The increased risk is minimal in extent and magnitude. No additional buildings are at risk as a result of this scenario.

Existing Return Period	Existing Flow	Climate Change Adjusted Flow	Corresponding Return Period of Climate Change Adjusted Flow
5-year	17.6 m ³ /s	23.4 m ³ /s	15 years
10-year	21.8 m ³ /s	29.0 m ³ /s	30 years
20-year	26.0 m ³ /s	34.6 m ³ /s	75 years
50-year	31.8 m ³ /s	42.3 m ³ /s	-
100-year	36.5 m ³ /s	48.5 m ³ /s	-

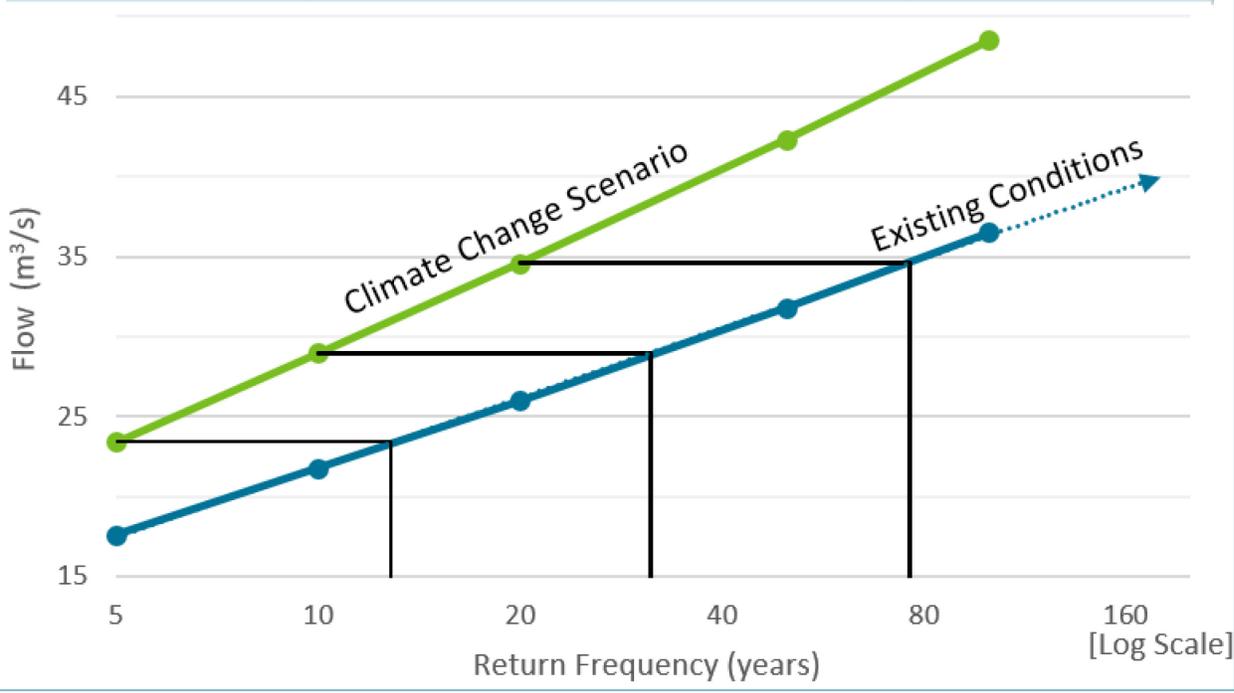


Figure 19 Climate Change Adjusted Flow Example

In theory, the climate change assessment results indicate that the 100-year +33% inundation boundary could become the Regulatory limit in the Town’s jurisdiction if the current climate change predictions occur as detailed above. However, prior to creating new policy around this prediction, a comprehensive review and update of the hydrologic inputs to this model for existing conditions should be completed. When hydrologic data is revised, the climate change assessment should be updated accordingly.

7.2 Future Conditions

The future conditions scenario includes the following changes, which are described in further detail below:

- Future developments at Beeton including updated flows and changes in land use
- Upsized culvert on 20th Sideroad just south of 5th Line

- Potential new culvert on Sir Frederick Banting Road north of Highway 89

7.2.1 Future Development at Beeton

To understand the impacts that proposed development will have on the flood conditions in Beeton, the Town requested that the sensitivity analysis include a future condition scenario to reflect the four proposed development areas in Beeton (Sorbara, Flato North, Flato South, and Oxnard developments) as shown on Figure 20.

As part of the Sorbara and Flato South developments, a flow diversion is proposed to reroute design event flows from Hendrie Drain toward Beeton Creek with base flows continuing to flow to Hendrie Drain (approximate location of diversion shown for to illustrate for schematic purpose only). Future condition flow data was obtained from the *Master Environmental / Servicing Report* (N.H.D. 2013) and the revised HEC-RAS flows are provided in Appendix H2. The Manning's n was also updated to reflect the change in land use that will occur at the four subdivision developments from the existing rural and/or agricultural land cover (Manning's n of 0.07) to the future urban residential land use (Manning's n of 0.05).

The flow diversion reduces flooding extents through Beeton as shown in Figure 21 for the modelled Regional event (similar figures comparing the remaining modelled events are in Appendix H4). Areas shown in red on the figure indicate those that were flooded in existing conditions but were reduced under future conditions. A more detailed understanding would likely be shown using an urban model that is capable of incorporating the storm sewers, SWM components, as well as the overland drainage system present in this area. Matrix recommends that Beeton be modelled in detail including all existing and future potential developments during Phase 2 of the DMP.



Figure 20 Future Development at Beeton

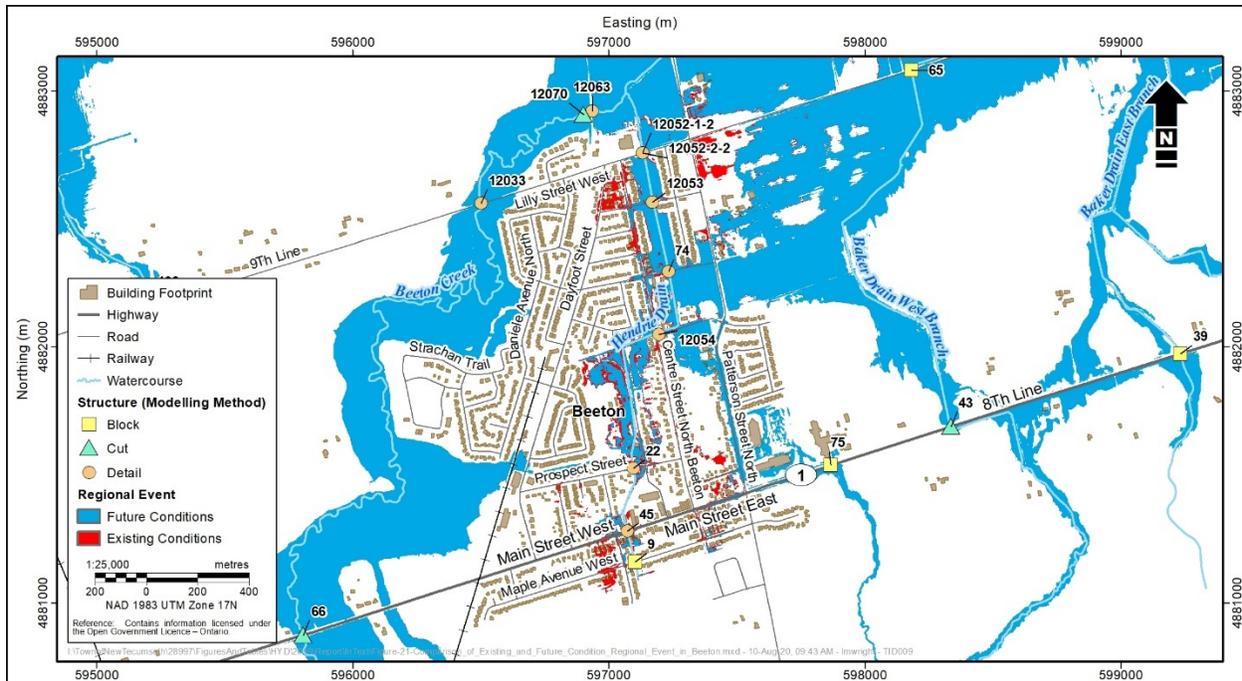


Figure 21 Comparison of Existing and Future Condition Regional Event in Beeton

7.2.2 Upsized Culvert on 20th Sideroad at 5th Line

The Town received flooding complaints near the intersection of 5th Line and 20th Sideroad after the June 2017 major rain event. The complaint was explored by Town staff with an external drainage consultant and a detailed analysis and report were completed (R. J. Burnside 2018). The analysis included hydrologic and hydraulic modelling of the existing 2.21 m span x 1.60 m rise corrugated steel pipe (CSP) arch culvert crossing 20 Sideroad on Penville Municipal Drain. The R.J. Burnside analysis suggests that the culvert has capacity to convey its assumed 10-year design flow for a local road; however, the provided freeboard (0.26 m) is slightly less than the 0.3 m requirement (MTO 2008). The analysis showed overtopping at a flow of 8.38 m³/s, which is between the 10-year and 25-year flows as calculated by R.J. Burnside (2018). This flow value falls between the 100-year (3.78 m³/s) and Regional (11.13 m³/s) flows as modelled by Matrix at this location. R.J. Burnside (2018) did not provide sufficient information for Matrix to reconcile the differences in these flows.

Recently, the resident complained of flooding again in the same area following a rain event in January 2020. The Town forwarded the complaint to Matrix to be reviewed as part of this flood risk assessment study and to understand potential future mitigation actions. Therefore, the culverts in this area were modelled in detail in the existing condition 2D HEC-RAS model.

The existing condition 2D HEC-RAS modelling completed by Matrix show overtopping starting at the Regional event, which confirms the estimated flow capacity of the 20 Sideroad culvert calculated by R.J. Burnside (between 3.78 m³/s and 11.13 m³/s). As part of the future condition assessment, the 20 Sideroad culvert (MSI_95) was upsized to a 3.60 m × 1.80 m rise concrete box culvert (refer to Appendix H3 for model details). The proposed culvert reduces flood elevations by up to 1.2 m during the modelled Regional event (flow of 11.13 m³/s) as shown in Figure 22 (similar figures comparing the remaining modelled events are in Appendix H4). If the return frequency of this flow as calculated by R.J. Burnside is correct, the replacement culvert modelled by Matrix will adequately meet the hydraulic performance standards and effectively mitigate flooding at this location for the range of modelled events. However, given the discrepancy in flow frequency at this location, further review of hydrology including IDF parameters is recommended during design before the Town proceeds with culvert replacement in the future.

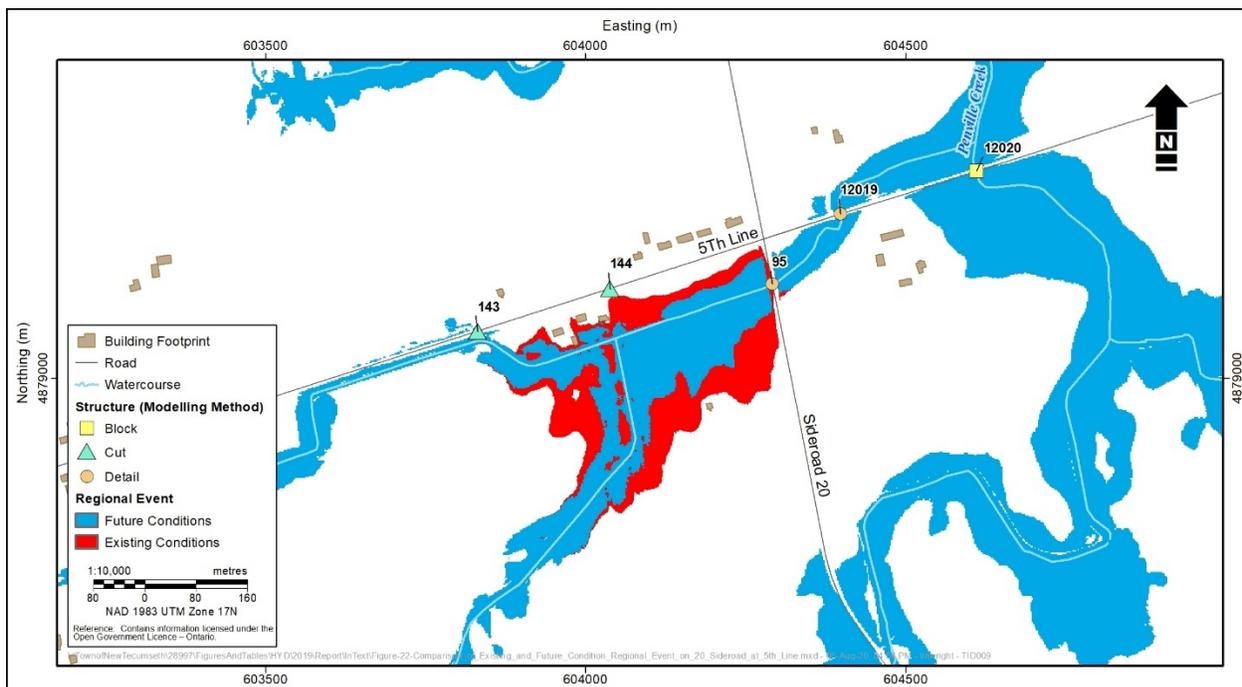


Figure 22 Comparison of Existing and Future Condition Regional Event on 20th Sideroad at 5th Line

7.2.3 Potential New Culvert on Sir Frederick Banting Road

The existing condition model results show flooding starting at the 20-year event at the Banting homestead on Sir Frederick Banting Road. A culvert was not identified by Matrix in this location during the background review or subsequent review of aerial imagery and therefore was not included in the existing condition model. While modelling

of mitigation recommendations is not part of the scope of this study, a potential new culvert was incorporated into the future condition scenario to demonstrate possible applications of the 2D HEC-RAS model as a future screening tool and to assess the reduction in flood risk that may be realized if a culvert was constructed here. The future condition scenario includes a 2.1 m span x 1.4 m rise concrete box culvert (MSI_146_Proposed) on Sir Frederick Banting Road approximately 950 m north of Highway 89. The modelled culvert details are provided in Appendix H3 and results from the 100-year event are shown in Figure 23.

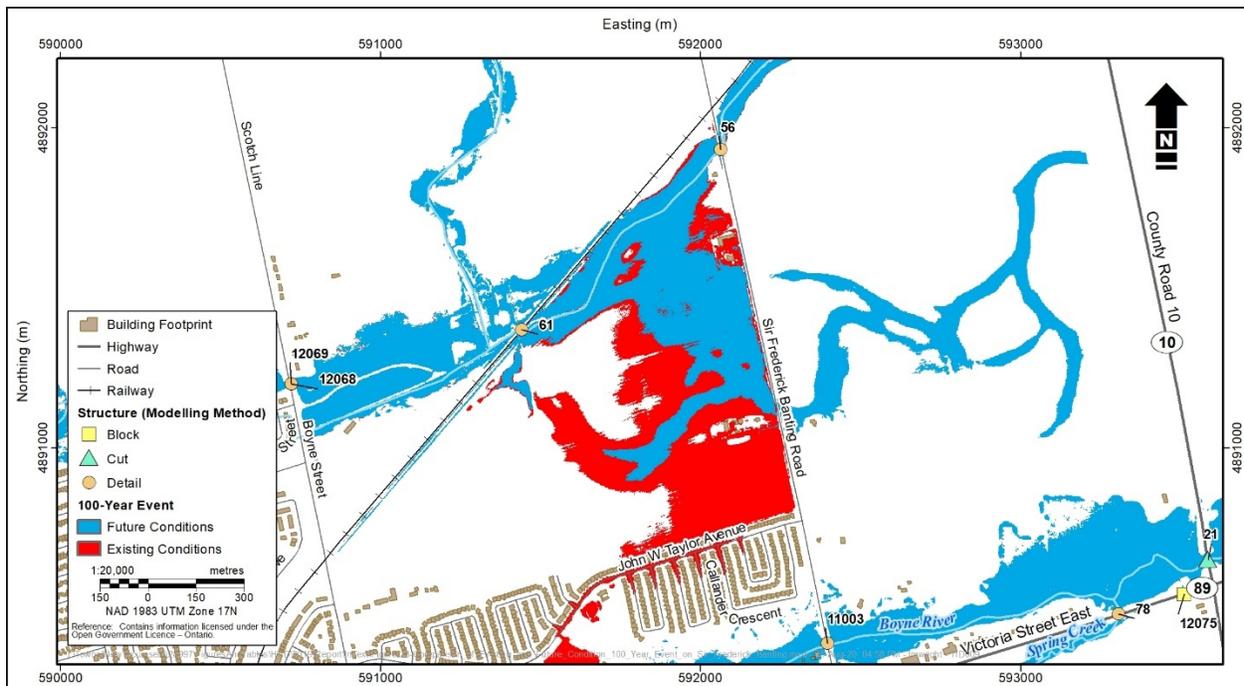


Figure 23 Comparison of Existing and Future Condition 100-year Event on Sir Frederick Banting

Results show significant reduction in flooding during all events except the Regional event. The Regional event overtops the road under existing and future condition scenarios and minimal changes in elevations were shown. The results of the modelling completed for this study demonstrate that there will be no increase in flood risk to downstream properties as a result of the new culvert during the Regional event. The flooding into the residential area on John W. Taylor Avenue shown during the existing condition 50-year event was also mitigated up to the 100-year event. Further assessment of culvert requirements should be reviewed in subsequent studies if the Town wishes to pursue installing a culvert here.

Matrix recommends that future drainage system needs, such as addition of a culvert, are coordinated with future road improvement studies in this area, particularly if road widening or other construction is proposed. There are opportunities to align the design and construction of multiple Town infrastructure projects to minimize disruptions and similar road works. Sir Frederick Banting Road at the Victoria Street intersection was found to have a poor flow of traffic, particularly for southbound travellers (Ainley & Associates 2012). Based on the results of a traffic signal analysis, a traffic signal is not warranted at this intersection and therefore the traffic study recommended that monitoring continue at the intersection. However, widening or other opportunities should also be considered in conjunction with flood risk mitigation.

8 Vulnerability Analysis

The flood damage assessment includes review of modelled flood depth, velocity, and frequency of inundation to provide an understanding of the existing vulnerability to private and public assets. The analyses discussed herein quantify the frequency and severity of flooding at buildings, land parcels (properties), roads, railways, and bridges/culverts to assist the Town and its residents in preparing for and responding to potential future floods. Where feasible, we have quantified the costs related to potential damage for planning and budgeting purposes.

As shown in Figure 24, flood damage encompasses both tangible and intangible damages. Intangible damages include social or emotional hardships suffered by those impacted by floods and therefore cannot be quantified. The tangible, or financial, damages include direct and indirect costs. Direct costs comprise structural damages to buildings, bridges and roadways, as well as damage to items within and around flooded buildings. Indirect costs include lost business, accommodations due to resident displacement, cleanup costs within buildings, as well as debris and refuse cleanup. Indirect costs are hard to quantify; they are typically calculated as a percentage of direct costs. Previous studies from across Canada have estimated indirect costs ranging from 10% to 45% of direct costs (Agra Earth and IBI 1998, IBI 2015, IBI and ECOS 1984, 1982, Kates 1965, Nichols 1979). In similar studies, Matrix generally recommends that the median of the percentages of previous studies be used, which corresponds to 15% for residential buildings, 35% for industrial buildings, 34% for institutional buildings. However, due to the uncertainty related to indirect and intangible costs, the damages calculated within this study include only the tangible direct costs. For the purpose of this report, the term “damage” is used to represent these financial damage costs.

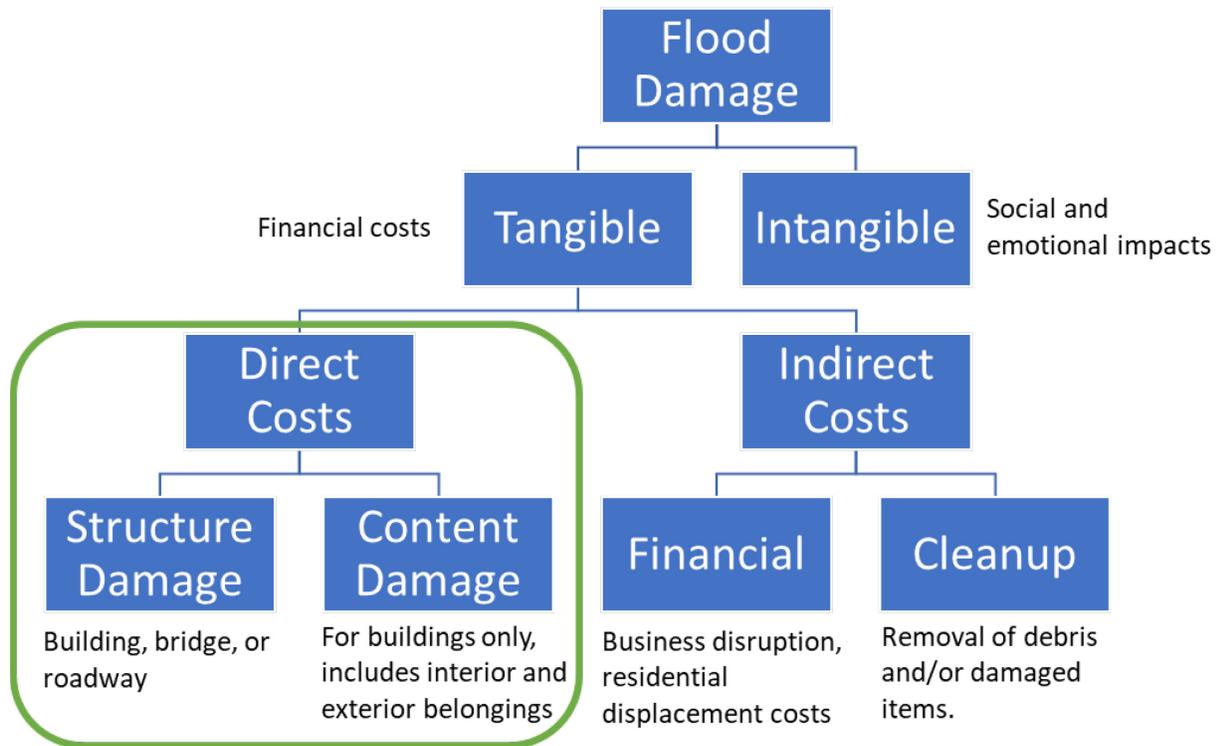


Figure 24 Overview of Flood Damage Components

The Town intends to use the results of the vulnerability analyses to develop plans for preparing for and responding to future potential floods. These plans may include prioritizing future studies and flood remediation efforts for flood risk areas subject to frequent damages or developing flood forecasting and warning systems for flood risk areas. The flood vulnerability analysis should be updated as aspects of the hydraulic model are updated (i.e., updated peak flows, refined bridges/culvert modelling approach, etc.) and as flood mitigation works are implemented, the vulnerability analysis should be updated.

Plans such as these are much easier to manage if they are consistent year after year, especially for budgeting purposes. While flood damages are not annually consistent due to year to year variations in weather, the average of flood damages over a number of events can be calculated to provide guidance on the long-term probability of flooding and the associated damages. The average annual damage (AAD) cost due to flooding was computed by plotting the total damage costs against probability of a given event occurring (e.g., the 5-year event has a 1-in-5 or 20% probability of occurring in any given year) and then computing the area under the curve. There is a high level of uncertainty in applying a return period and annual probability of occurrence to the Regional events, as they are historical storms of record and likely the highest recorded

in a given area. Therefore, the Regional event results were not used in the AAD calculations.

8.1 Buildings

Flood depth-damage curves were used to relate the flood depth at buildings to dollars of damage. The currently accepted curves in Canada were developed in Calgary following the July 2013 flood event (IBI 2015). Toronto and Region Conservation Authority recently had these curves translated to reflect Ontario costs (e.g., based on repair material and contractor rates) and updated to 2016 dollars. Matrix recognizes that this report was completed in 2020 and there has been a 7.89% increase in the price of consumer goods since 2016 due to inflation (Bank of Canada 2020). However, to avoid updating the depth-damage curves the analysis was completed using 2016 dollars. The depth-damage curves consist of two components: structural damages and content damages. The structural component includes repairs to the building structure and building components that are considered part of the house (i.e., not taken if an individual is moving) such as the furnace, hot water heater, wall-to-wall carpeting, etc. The content component includes the internal building contents and external items such as lawn furniture, garages, sheds, etc. The depth-damage curves used for the project are presented in Appendix I.

8.1.1 Flood Damage Database

The first step in estimating flood damage costs at buildings is developing and populating a database of relevant building assets within the study area. Any building that was within or partially within (touching) the modelled flood inundation extents was included in the flood damage database. The flood damage database therefore includes 630 buildings that lie partially or completely within the modelled flood extents from the 2D HEC-RAS model. Note that the buildings database prepared with this study was developed using the buildings shapefile provided to Matrix by the Town in November 2019. During the field inspection for building verification, it was noted that some buildings in new residential developments were not captured in the shapefile (in Tottenham, on and around Turner Driver south of Weaver Terrace; and in Alliston, on and around Michaelis Street). Due to time restrictions on the project, these additional buildings were not captured in this assessment. Matrix recommends that the Town keep an up to date record of new buildings in its jurisdiction to facilitate future flood damage assessment needs.

8.1.2 Summary of Required Attributes

The following sections describe how the building attributes were gathered. Table 13 summarizes the required attributes. A detailed explanation of each attribute, its use, and its data gap filling method is presented in Appendix J.

Table 13 Building Database Attributes

Attribute	Source
Building Type	Town GIS data and confirmed/revised through visual inspection
Interior Area	Polygon area from Town GIS data reduced by attached garages where identified in visual inspection and roof overhang factor
Number of Storeys	Visual inspection
Ground Elevation	Minimum, mean, and maximum ground elevations at building from LiDAR
First floor Elevation	Visual inspection (based on number of stairs to front door) and ground elevation
Lowest Opening Elevation	Assumed equal to minimum LiDAR elevation at building

the Town - the Town of New Tecumseth

LiDAR - light detection and ranging

Building attributes were gathered from a combination of GIS data provided by the Town and visual inspection as appropriate; many of the attributes required further visual inspection. Matrix completed these visual inspections through a combination of publicly available photos (i.e., Google Street View) and field inspection through to March 2020. Google Street View is not available on all roads within the Town's jurisdiction, building attributes may be obscured (e.g., stairs obscured by hedge), and the photography is not always clear (e.g., when the building is too far from road). Field inspections, while normally the most conclusive, may be complicated by temporary obstructions (e.g., parked vehicles, foliage, snow piles). For each building, the best available method was used for visual inspections. Of the 630 buildings in the flood damage database, field inspection was completed for 336 buildings, and Google Street View and/or aerial imagery was used for 294 buildings. The database includes a record of the visual inspection method for each building.

Of the 336 buildings for which a field inspection was attempted, 162 buildings were inaccessible or partially obstructed and therefore some assumptions were required and are identified as such in the database (i.e., each attribute is recorded as “inspection” or “assumed” depending on how the data was collected). Where needed, conservative assumptions were made to fill data gaps in attributes. For example, if stairs leading up to the first floor were not clearly visible, Matrix assumed that no stairs were present; this is a conservative assumption for the first floor elevation as it assumes first floor elevation is equal to ground elevation and is therefore more susceptible to flooding. As another example, if Matrix was unable to count the number of attached garages, it was assumed there were none. Assuming no garage means a greater living space area is calculated and provides a conservative estimate of damage costs. Matrix acknowledges that if assumptions are too conservative, the flood damage costs may be overestimated. Therefore, by recording parameters that were assumed in the database, the Town or future users of the database can more easily update using refined data as it becomes available.

8.1.2.1 Building Type

Each building type was assigned to appropriate structure and content depth-damage curves to facilitate damage cost calculations. These curves allow the depth of flooding at each building to be associated with a damage cost per unit area. Using the building type provided in the Town’s shapefile, Matrix classified each building as outlined in Table 14 to align with available structure and content depth-damage curves.

Some minor updates were made to building types in the Town’s shapefile based on the visual inspection.

Table 14 Summary of Building Type and Class

Building Type ⁽¹⁾	Structure Class	Content Class
Accommodation	Hotel/Motel	Hotels
Commercial	Office/Retail	*refer to list below
Community Centre	Institutional	Institutional
Cultural	Institutional	Institutional
Educational	Institutional	Institutional
Farm	Industrial/Warehouse	Warehouse/Industrial
Golf Course	Institutional	Institutional
Industrial	Industrial/Warehouse	Warehouse/Industrial
Infrastructure	Industrial/Warehouse	Warehouse/Industrial
Recreation	Institutional	Institutional

Building Type ⁽¹⁾	Structure Class	Content Class
General	Residential	Residential
Other/Null	Individually assessed	

(1) Building type as identified in Town’s database.

Each commercial building was visually inspected by Matrix to determine the most appropriate content depth-damage curve from the available options below.

- Shoes
- Clothing
- Stereos/TV/Electronics
- Paper Products
- Hardware/Carpet
- Miscellaneous Retail
- Furniture/Appliances
- Groceries
- Drugs
- Auto
- Hotels
- Restaurants
- Personal Service
- Financial
- Theatres

There are multiple depth-damage curves available for residential buildings based on the wide variations within residential housing. Residential buildings were classified according to their structure type (mobile home, apartment building, and standard home), size and quality of living space, and number of storeys, details of which were provided in Appendix J. Each residential building was visually inspected to confirm the applicable residential structure and content depth-damages curves (refer to Appendix I).

8.1.2.2 Interior Area

The interior area of a building is a critical attribute for the flood damage database. This building attribute can be acquired from the Municipal Property Assessment Corporation; however, gathering and processing this data requires a significant level of effort (NRCAN 2017) that is not warranted for a broadscale study like this. For the purposes of this project, the building outline area in the Town’s GIS data was used as a surrogate for the interior area of a single floor. A single storey of the internal area is used for calculating damage costs from the depth-damage curves. As a result of this process, townhouses, and other linked buildings are considered a single building in the GIS base data.

8.1.2.3 Number of Storeys

For residential buildings, the number of storeys is used to estimate the total living space area in order to determine its content class. Number of storeys was determined through visual inspection.

8.1.2.4 Ground Elevation

The minimum, mean, and maximum ground elevation was sampled from the LiDAR dataset, within a 1 m buffer of each building. Ground elevation is an important value needed to estimate the first-floor elevation and flood depths.

8.1.2.5 First Floor Elevation

Depth-damage curves for buildings are based on flood depth relative to the first floor of each building. While finished floor elevations are typically included in building designs, these attributes are typically not carried through to municipal databases. First-floor elevations were required for the flood damage assessment and therefore were estimated through visual inspection based on the number of stairs leading to the front door and a representative ground elevation at the location of the stairs. Refer to Table 15 for examples of how first floor elevations were estimated for various building types.

Table 15 First Floor Elevation Estimates

Example	Description
	<p>Stairs were counted from the front of this flat lot.</p> <p>First floor elevation is represented by the mean LiDAR elevation at the building plus the stair height.</p>
	<p>Stairs were counted from the low point at the front of the house.</p> <p>First floor elevation is represented by the minimum LiDAR elevation at the building plus the stair height.</p>

Example	Description
	<p>The stairs were counted from the high point at the front of the building as the lot drops off to the rear of the building.</p> <p>First floor elevation is represented by the maximum LiDAR elevation at the building plus the stair height.</p>

LiDAR - light detection and ranging

8.1.2.6 Lowest Opening Elevation

In addition to the first-floor elevation, the lowest opening (window well, basement door, etc.) for each building is required to determine the minimum water elevation that will cause flooding. This is a difficult attribute to identify for each building because lowest openings can often be at the rear of a house. Detailed data collection for this attribute would require survey equipment and permission to enter at each property. As such, it has become common practice to assume the lowest opening is equal to the lowest elevation captured by the LiDAR adjacent to the building and was carried for this project. Note that during the visual inspections Matrix did not identify any buildings that have underground parking in the flooded areas within the study area.

8.1.3 Flood Depths

Flood depths for each modelled storm event were assigned to each building for the existing condition, future condition, and climate change scenarios. The maximum water surface elevation within 1 m of each building was entered into the database for each modelled event. Buildings with water surface elevations above the lowest opening elevation are considered inundated (flooded). Table 16 lists the number of inundated buildings categorized by type for existing conditions. The table excludes non-residential buildings which have a water elevation lower than first floor elevation, since basement damage costs are not calculated in these cases (described further below). Table 16 also excludes some residential buildings that are in the database because they show flooding within the 1 m buffer but are located on a steep slope. In these cases, the flood elevation was below the lowest opening, and therefore these buildings are not considered flood vulnerable.

Table 16 Number of Flood Vulnerable Buildings - Existing Condition

Building Type ⁽¹⁾	Modelled Event					
	5-year	10-year	20-year	50-year	100-year	Regional ⁽³⁾
General	38	54	84	112	137	388
Accommodation	0	0	0	0	0	2
Commercial	1	1	1	1	1	1
Community Centre	0	0	0	1	1	3
Educational	0	0	0	0	0	1
Farm	20	28	35	50	64	146
Golf Course	0	0	0	0	0	1
Industrial	2	2	2	2	2	9
Industrial (Honda Plant) ⁽²⁾	0	0	0	0	0	1
Infrastructure	1	2	3	4	4	10
Recreation	0	0	1	1	4	8
Total	62	87	126	171	213	570

(1) Building type as identified in Town's database.

(2) The Honda manufacturing plant in Alliston is reported separately because it consists of three connected buildings with a large combined footprint area (214,400 m²) and therefore has the potential to significantly skew the calculated flood damage costs for industrial buildings.

(3) The climate change scenario (100-year + 33%) produced lower (but similar) flood elevations to the Regional event throughout most of the study area (exception discussed in Section 7.1). Matrix thoroughly reviewed the inundation boundary resulting from the climate change scenario and the increased risk compared to the Regional flood is minimal in extent and magnitude. No additional buildings are at risk as a result of the climate change scenario and therefore the reported Regional values reported can be assumed to apply as a conservative estimate of impacted buildings during the climate change event.

For illustrative purposes, two possible cases for calculating damage costs at residential buildings are shown in the schematics below. Figure 25 illustrates the case where the water surface elevation is above the first-floor elevation of a residential building (i.e., the

flood depth relative to the first floor is >0 m). This scenario would induce damage to both the basement and first floor.

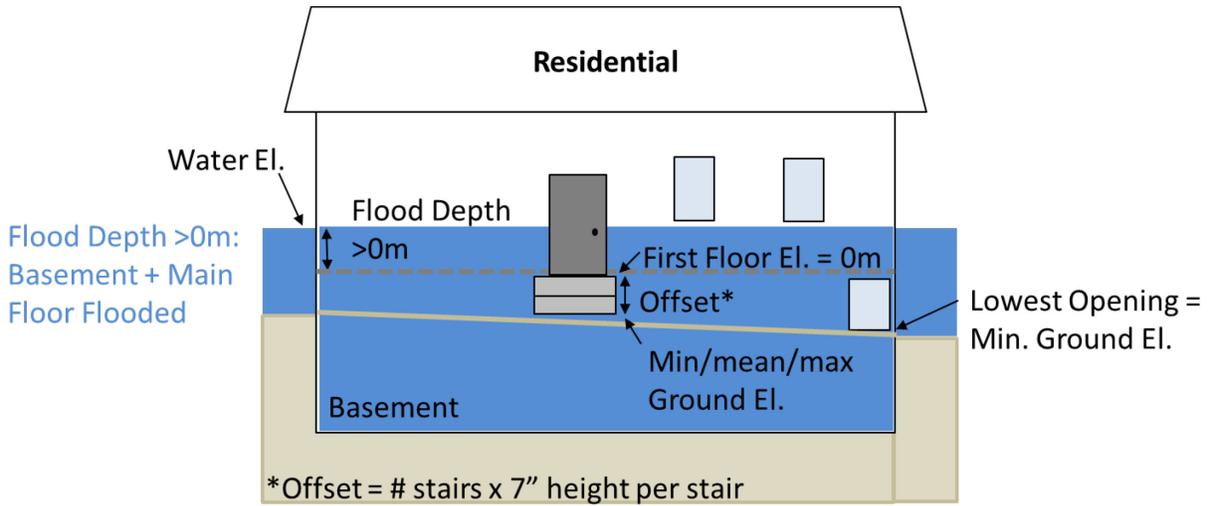


Figure 25 Schematic of Residential Building with Flooding above First-floor Elevation

Figure 26 illustrates the case for residential buildings where the water elevation is below the first-floor elevation but still above the lowest opening elevation. In this case, the flood depth relative to the first-floor elevation is <0 m, but water would be permitted to enter the basement via the lowest opening and therefore this scenario would have damages to the basement only.

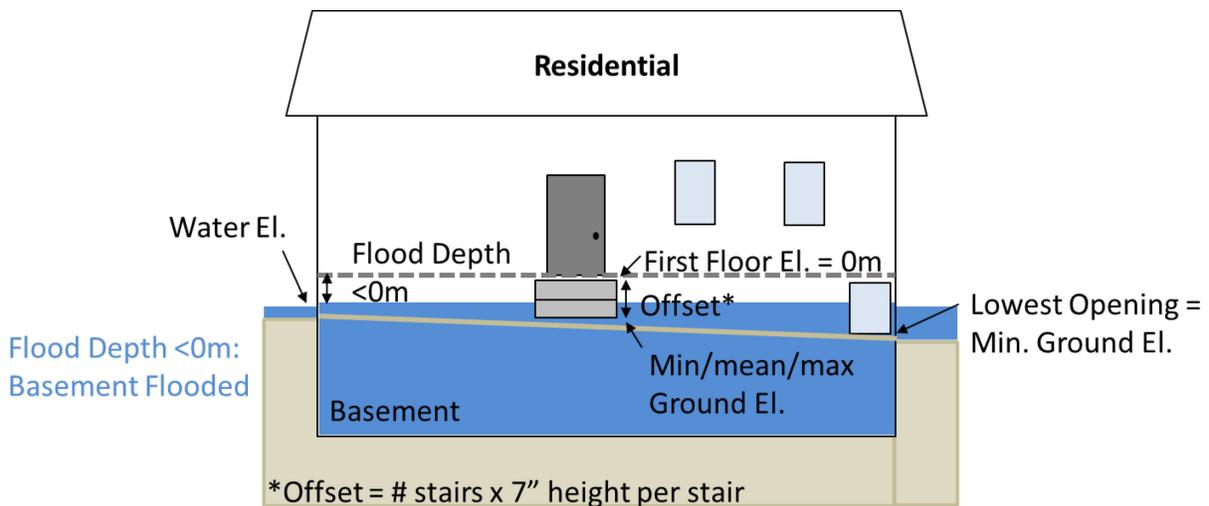


Figure 26 Schematic of Residential Building with Flooding below First-floor Elevation

For non-residential and apartment buildings (Figure 27), the currently accepted depth-damage curves do not consider basement damage costs. Consequently, there is only one case for the computation of flood damages and that is when the modelled flood elevation exceeds the first-floor elevation.

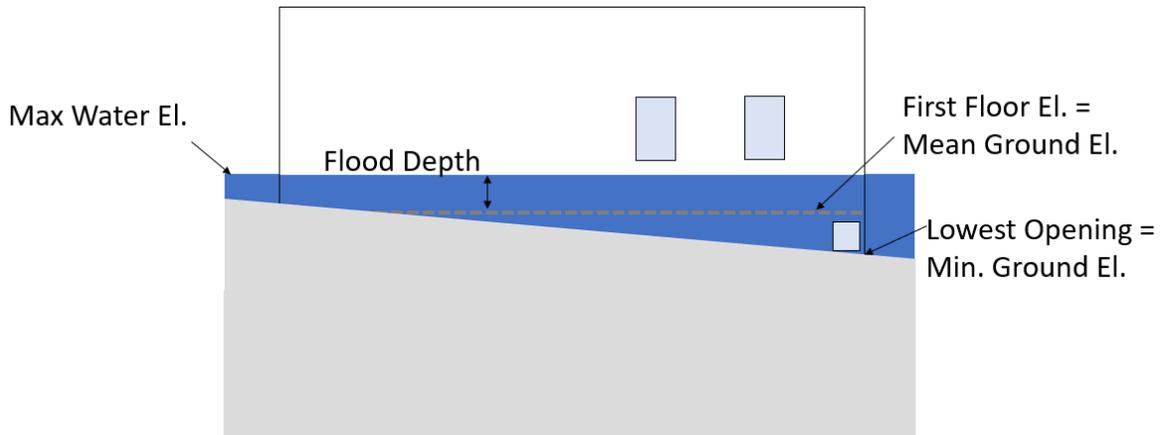


Figure 27 Schematic of Non-residential and Apartment Buildings with Flooding above First-floor Elevation

1.1.1 Flood Damage Costs

Using the calculated flood depths and the properties of each building, flood damage costs were calculated for each modelled event. These damage costs are presented in Table 17. A detailed breakdown of flood damage costs is included in Appendix L.

Implementation of flood mitigation measures would reduce the damage costs reported in Table 17. Detailed modelling of mitigation measures would be required to understand the true extent of the cost and impact/benefit of implementation. An assessment of potential flood mitigation is recommended in future phases of the study.

Table 17 Building Damage Costs

Building Type ⁽¹⁾	Flood Damage Cost at Buildings ⁽²⁾					
	5-year	10-year	20-year	50-year	100-year	Regional ⁽³⁾
General	\$3,240,504	\$5,279,207	\$8,026,702	\$12,181,258	\$14,693,245	\$48,872,457
Accommodation	\$0	\$0	\$0	\$0	\$0	\$2,877,971
Commercial	\$655,667	\$656,326	\$656,985	\$657,079	\$674,775	\$690,639
Community Centre	\$0	\$0	\$0	\$7,538	\$17,817	\$386,564
Educational	\$0	\$0	\$0	\$0	\$0	\$35,018
Farm	\$3,531,972	\$5,120,550	\$7,914,479	\$14,179,222	\$19,104,713	\$45,256,464
Golf Course	\$0	\$0	\$0	\$0	\$0	\$1,950
Industrial	\$116,900	\$119,110	\$119,847	\$160,257	\$163,013	\$15,149,269
Industrial (Honda Plant) ⁽⁴⁾	\$0	\$0	\$0	\$0	\$0	\$267,682,245
Infrastructure	\$1,325	\$4,897	\$84,883	\$343,302	\$480,067	\$708,854
Recreation	\$0	\$0	\$5,999	\$7,278	\$60,486	\$182,377
Total	\$7,546,369	\$11,180,090	\$16,808,895	\$27,535,934	\$35,194,116	\$381,843,808

(1) Building type as identified in Town’s database.

(2) Flood damage costs were calculated in 2016 dollars as this is what is incorporated into the depth-damage curves.

(3) The climate change scenario (100-year + 33%) produced lower (but similar) flood elevations to the Regional event throughout most of the study area (exception discussed in Section 7.1). Matrix thoroughly reviewed the inundation boundary resulting from the climate change scenario and the increased risk compared to the Regional flood is minimal in extent and magnitude. No additional buildings are at risk as a result of the climate change scenario and therefore the reported Regional values reported can be assumed to apply as a conservative estimate of impacted buildings during the climate change event.

(4) The Honda manufacturing plant in Alliston is reported separately because it has a large combined footprint area and would skew the calculated flood damage costs for industrial buildings.

8.1.4 Average Annual Damages

AAD represent the sum of forecasted damages over a given time period divided by the length of the time period. This metric accounts for the fact that flood damages may be primarily due to large events in a number of years, while little damage may occur during the longer periods of time between events. For example, the AAD over the past 30 years would be represented by the total direct damage costs due to flooding in all flood events that occurred during the past 30 years, divided by 30 years.

Because data on direct damages in historical events is often incomplete, AAD can be estimated using the results of hydraulic models for a series of design storms. For each storm, the total direct damage costs to buildings are plotted against their probability of occurring as illustrated in Figure 28. For example, the 5-year storm has a 20% probability of occurring in any given year and would result in an estimated \$7.5 million dollars. The AAD is then estimated by calculating the area under the damage-probability curve. This approach accounts for the fact that the costliest storms have a smaller chance of occurring. Smaller events may contribute significantly to AAD; although they cause less damage, they occur more frequently. By calculating the area under the curve in Figure 28, AAD costs to the buildings in the flood damage database was calculated as \$5.4 million.

Since the AAD is averaged across a range of events, it provides a single metric to compare the reduction in damages resulting from various mitigation options (GRIC 1982). To quantify the damage reduction, detailed modelling of each mitigation option would be required, after which the AAD estimates are updated using new flood risk results and compared to the existing condition AAD.

Matrix understands that the Town is not responsible for covering these damage costs. Building and/or property repair and maintenance costs are the responsibility of the property and/or homeowners. It should be noted that Figure 28 does not consider damage costs to the Honda Plant in Alliston since the modelling results indicate that this plant is only flooded in the Regional event and AAD cost calculations only considered the return period storms. Damage costs were extrapolated for smaller return periods (1-year and 2-year), since these events were not modelled, and damage costs were thus not available for these events. The AAD costs for each type of vulnerable asset is provided in the sections below.

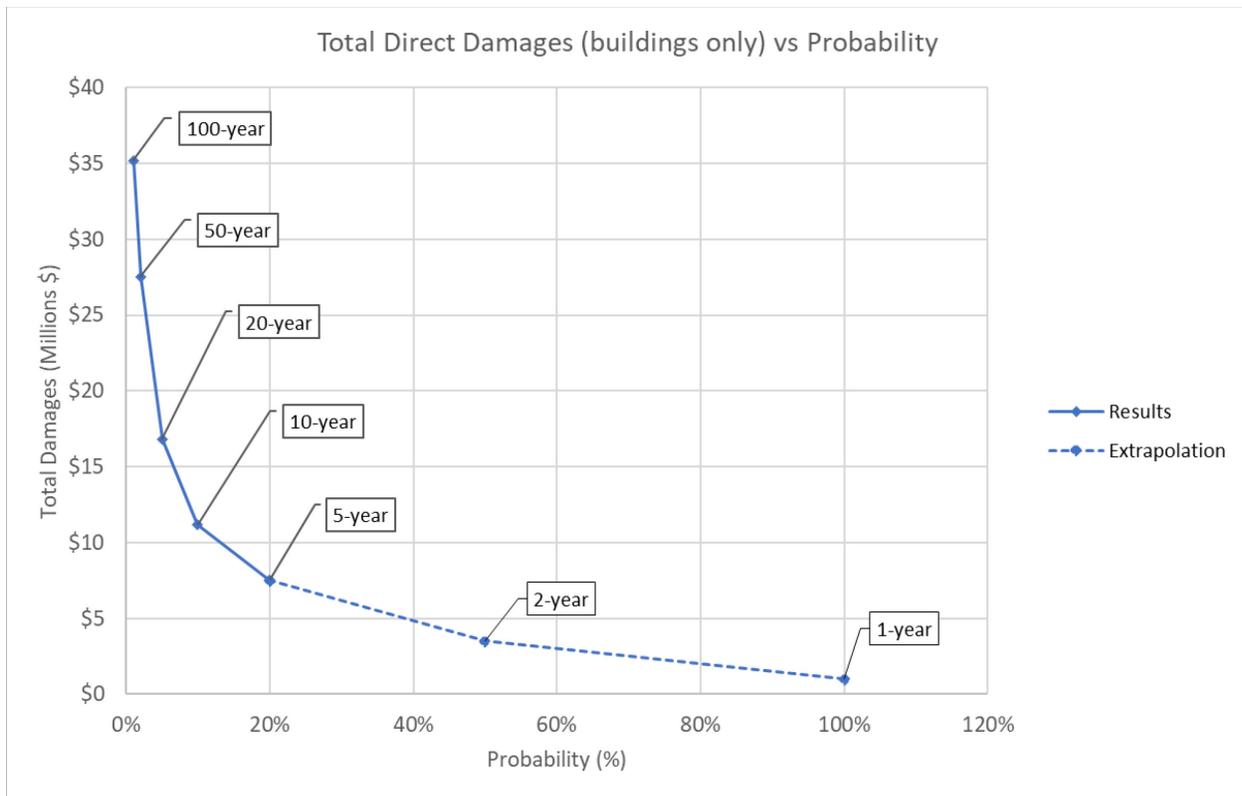


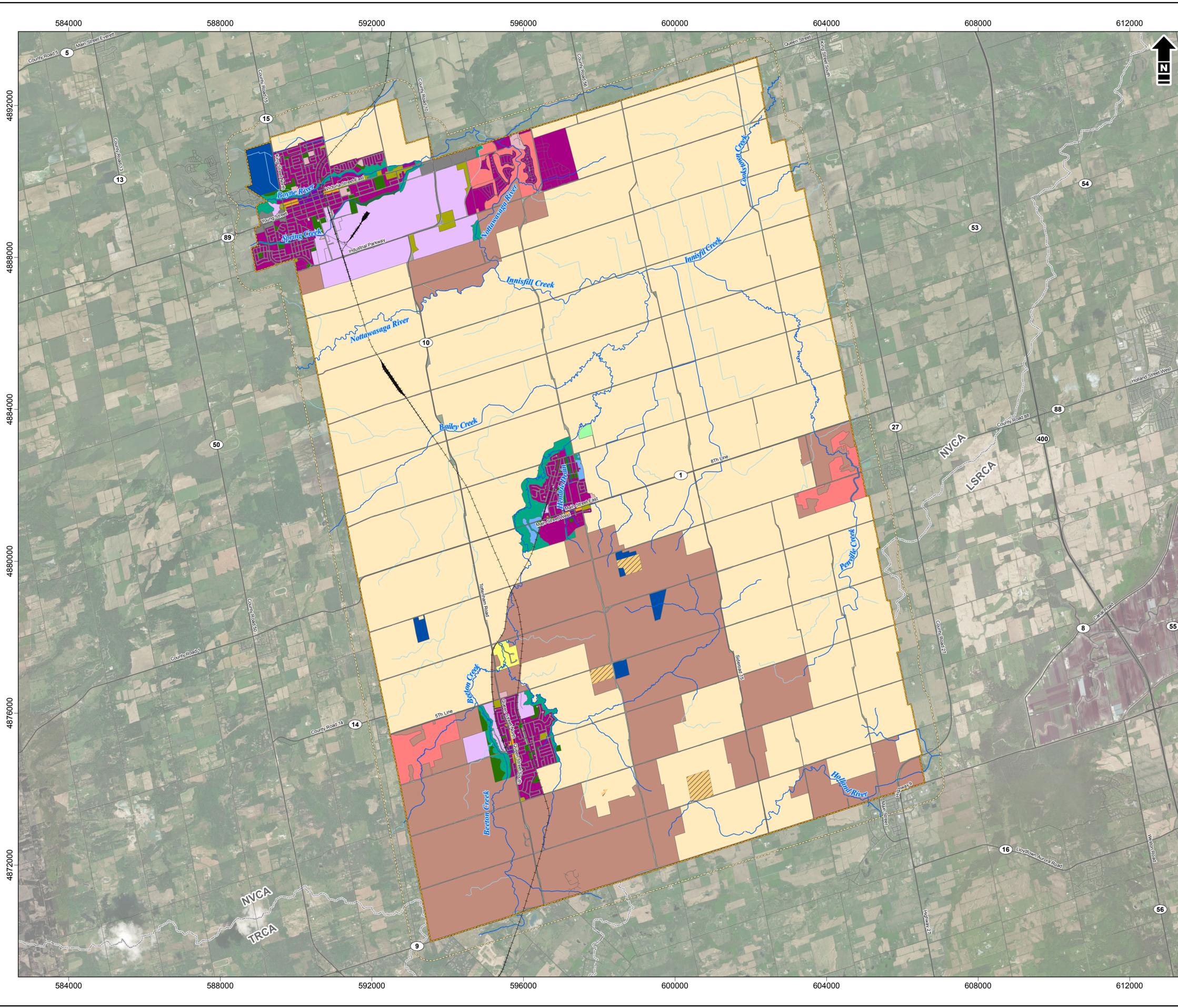
Figure 28 Direct Damage Costs (Buildings) Probability Distribution

8.2 Land Parcels

Damage costs to agricultural land parcels requires detailed local information including growing costs, post-harvest costs, and establishment costs (i.e., costs of replanting after damages). Natural Resources Canada emphasizes the importance of using local data and monthly production costs. Flood damages are also sensitive to flood duration and time of year (NRCAN 2017). Further, flood damage to agricultural lands is complex as it can be influenced by rainfall and climate both directly (physical damage or loss of crops or livestock caused by sustained periods of flood, drought, and freeze/thaw cycles) and indirectly (associated damages caused by pests, weeds, disease, etc. that are influenced by long-term or seasonal variations in weather). Matrix was unable to find appropriate data and/or depth-damage curves specific to agricultural land parcels within the scope of this project; therefore, the vulnerability of land parcels including erosion, loss of crops, etc. was not assessed in terms of financial costs. The vulnerability of land parcels was assessed solely from a technical standpoint including review of frequency of inundation and flood risk for the various modelled events. In future studies if the Town is interested in quantifying damage costs in agricultural areas, collection of the following data is recommended: type of crop grown; type and number of livestock; details of cultivation and harvest practices (i.e., timing).

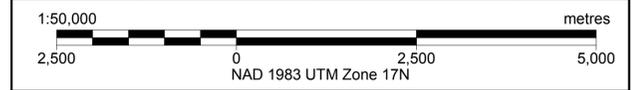
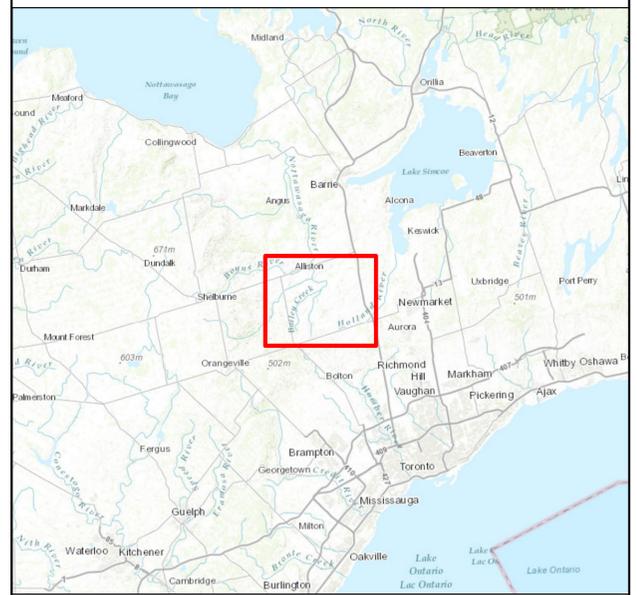
The land use data used in this assessment shown in Figure 29 was based on the Town's Official Plan (Town of New Tecumseth 2019) which indicates the land use strategy for growth and development on the 20-year horizon. As such, some land uses indicated in the Official Plan are for future conditions. For example, some lands indicated as Corridor Commercial have not yet been developed. Table 18 provides a breakdown of the percentage of each land use that is at risk to flooding under each of the modelled events. For example, for the 5-year event, 12% of the 17,811 m² of agricultural land use is susceptible to flooding. Note that this table combines the total area of all flood inundated lands (i.e., sum of low, medium, and high risk); a detailed breakdown of each of the low-, medium-, and high-risk areas for each land use type is provided in Appendix L.

In addition to inundated area, the number of flood vulnerable land parcels were identified for each modelled event. Table 19 indicates the number of land parcels where some portion of the property is inundated; a property is considered vulnerable when any portion is inundated. The actual inundation extents on any given property is shown in the risk mapping presented in Appendix H1. Detailed tables that differentiate between low-, medium-, and high-risk are provided in Appendix L2 and L3.



- Municipal Boundary
- Study Area (500m from Municipal Boundary)
- Conservation Authority Jurisdiction Boundary
- Highway
- Road
- Railway
- Watercourse**
- Main Branch
- Tributary
- Land Use**
- Agricultural
- Major Commercial
- Corridor Commercial
- Downtown Core Transitional
- Employment Area
- Environmental Protection
- Institutional
- Open Space - Rural
- Open Space - Urban
- Recreational
- Residential - Rural
- Residential - Urban
- Rural
- Special Study
- Waste Disposal

Service Layer Credits: Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community
 Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User



The Corporation of the Town of New Tecumseth
 Drainage Master Plan Phase 1

Land Use

Date: March 2020 Project: 28997 Submitter: T. Martin Reviewer: K. Molnar

Table 18 Inundated Area per Total Land Use Type for Modelled Events

Land Use ⁽¹⁾	Total Area (ha)	Flood Vulnerable Area (Percentage of Total Area)					
		5-year	10-year	20-year	50-year	100-year	Regional
Aggregate Extraction	104	0%	0%	0%	0%	0%	0%
Agricultural	17,881	12%	14%	15%	18%	19%	30%
Corridor Commercial	33	2%	3%	4%	5%	5%	6%
Downtown Core Commercial	16	1%	2%	2%	2%	2%	3%
Downtown Core Transitional	13	1%	1%	2%	3%	3%	4%
Employment Area	904	4%	6%	7%	9%	10%	22%
Environmental Protection	197	39%	42%	45%	47%	49%	57%
Institutional	35	2%	2%	2%	3%	3%	6%
Major Commercial	60	0%	0%	0%	0%	0%	0%
Open Space - Rural	10	22%	41%	53%	62%	69%	92%
Open Space - Urban	81	5%	6%	10%	11%	12%	18%
Recreational	520	9%	9%	9%	10%	11%	13%
Residential - Rural	34	20%	21%	22%	23%	23%	33%
Rural	5,518	4%	4%	5%	5%	5%	7%
Special Study	10	0%	1%	1%	1%	1%	7%
Unclassified	19	5%	6%	16%	33%	43%	47%
Urban Residential	936	1%	2%	2%	2%	2%	4%
Waste Disposal	142	16%	18%	21%	23%	25%	40%

(1) Land use as identified in the *New Tecumseth Official Plan* (Town of New Tecumseth 2019).

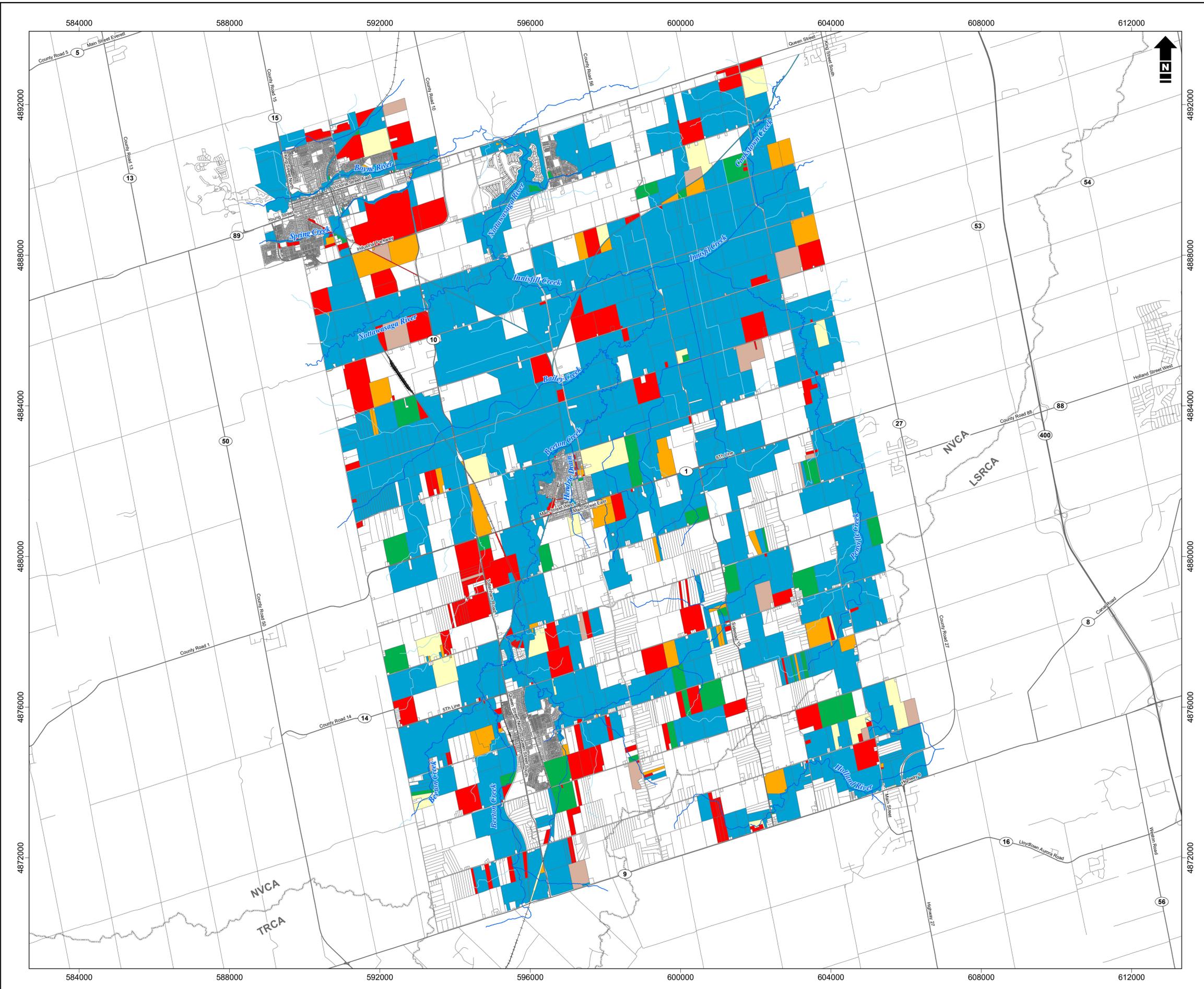
Table 19 Number of Flood Vulnerable Land Parcels by Flood Event

Land Use ⁽¹⁾	Total Number of Parcels	Number of Land Parcels at Risk					
		5-year	10-year	20-year	50-year	100-year	Regional
Aggregate Extraction	5	0	0	0	0	0	0
Agricultural	1,468	600	610	623	636	642	728
Corridor Commercial	93	12	12	13	12	12	13
Downtown Core Commercial	198	3	3	4	6	9	10
Downtown Core Transitional	147	16	17	18	20	22	22
Employment Area	92	24	37	40	44	45	57
Environmental Protection	63	47	47	47	47	48	50
Institutional	17	4	4	5	6	6	7
Major Commercial	24	1	1	1	1	1	2
Open Space - Rural	1	1	1	1	1	1	1
Open Space - Urban	85	13	14	16	19	19	25
Recreational	36	17	18	18	19	19	19
Residential - Rural	28	7	7	8	8	8	8
Rural	1,057	272	273	273	278	278	296
Special Study	6	1	2	3	3	4	4
Unclassified	19	1	1	1	1	1	1
Urban Residential	11,002	219	238	297	360	427	1,028
Waste Disposal	9	3	3	3	3	3	3
Total	14,350	1,241	1,288	1,371	1,464	1,545	2,274

(1) Land use as identified in the *New Tecumseth Official Plan* (Town of New Tecumseth 2019).

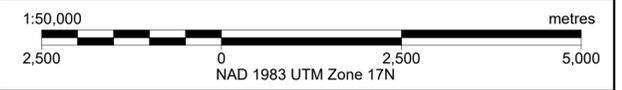
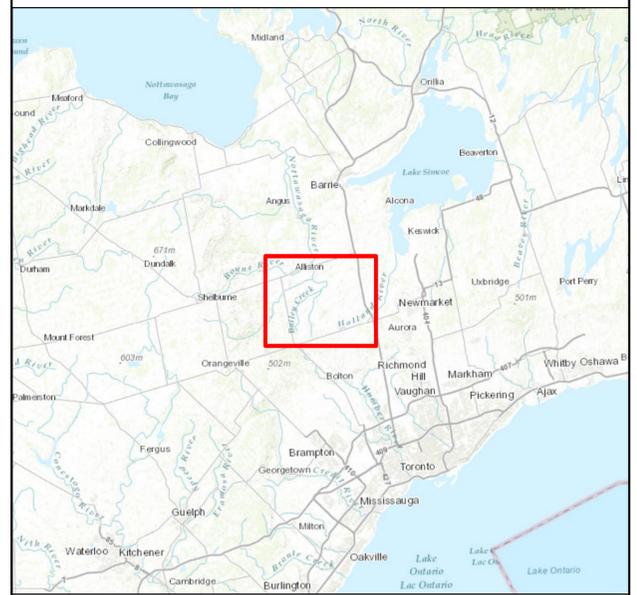
8.2.1 Average Annual Damages - Land Parcels

As discussed in Section 8.1.4, the financial damage cost for land parcels was not calculated due to the number of unknowns associated with agricultural land and production value. Figure 30 provides a spatial representation of the frequency of inundation for each property. In this figure, a property is considered vulnerable when any portion is inundated. An interactive HTML file that shows this same information is included as Appendix K (digital only) for use by the Town.



- Conservation Authority Jurisdiction Boundary
- Highway
- Road
- Railway
- Watercourse**
- Main Branch
- Tributary
- Parcel Inundation Frequency**
- 5-yr
- 10-yr
- 20-yr
- 50-yr
- 100-yr
- Regional
- No Flooding

Service Layer Credits: Sources: Esri, HERE, Garmin, Intermap, increment P Corp., GEBCO, USGS, FAO, NPS, NRCAN, GeoBase, IGN, Kadaster NL, Ordnance Survey, Esri Japan, METI, Esri China (Hong Kong), (c) OpenStreetMap contributors, and the GIS User Community



The Corporation of the Town of New Tecumseth
Drainage Master Plan Phase 1

Parcel Inundation Frequency

Date: March 2020 Project: 28997 Submitter: P. Bishop Reviewer: K. Molnar

Disclaimer: The information contained herein may be compiled from numerous third party materials that are subject to periodic change without prior notification. While every effort has been made by Matrix Solutions Inc. to ensure the accuracy of the information presented at the time of publication, Matrix Solutions Inc. assumes no liability for any errors, omissions, or inaccuracies in the third party material.

8.3 Roads and Rails

Understanding flood risk along transportation routes is important for emergency planning including identification of evacuation routes and first responder emergency routes. The flood risk maps presented in this study can be used by the Town's emergency management team to develop emergency response routes as appropriate for their needs.

In a primarily rural area such as New Tecumseth, the road network is one of the most valuable and costly assets. Major flood damage to the road network could be costly to the Town and we understand the Town wishes to proactively budget for flood mitigation and repairs in case of such an event. The major storm event of June 2017 is believed to have resulted in over \$850,000 in costs to the Town's public works department (Pritchard 2017).

The length of road at risk of flooding was calculated for each modelled event and categorized using the flood risk categories presented in Section 6 overlaid with the existing road shapefile obtained by Matrix from LIO. The results of this assessment are displayed graphically in Appendix L.5 and a summary is provided in Table 20 below, grouped by road type. Definitions of road types are provided in Table 21.

An item of note here is that crossings that were modelled using the "cut" approach (Section 5.4.2) indicated flow across the road; however, in these locations, the flow actually occurs under the road and there is no road overtopping. These locations were reviewed and lengths of at-risk road were calculated to account for this.

Note that culverts modelled using the "block" approach were revised to a "cut" approach for the 5-year, 10-year, and 20-year simulations since it is too conservative to assume full blockage for these smaller events (refer to Section 5.4.3).

Table 20 Flood Risk at Roads

Modelled Event	Road Type (¹)	Length of Road at Risk (m) (²)			
		Low	Medium	High	Total
5-year	Arterial	208	1	10	220
	Collector	543	1	15	560
	Local	-	-	2	2
	Total	2,333	2,189	3,803	8,324
10-year	Arterial	222	-	12	233
	Collector	546	-	16	563
	Local	-	-	2	2
	Total	2,313	2,645	5,056	10,013
20-year	Arterial	250	-	13	263
	Collector	569	-	17	586
	Local	-	-	2	2
	Total	3,094	2,498	6,478	12,070
50-year	Arterial	332	-	13	344
	Collector	640	-	21	661
	Local	-	-	2	2
	Total	4574	2709	9,211	16,494
100-year	Arterial	362	-	14	376
	Collector	673	1	26	699
	Local	-	-	2	2
	Total	5,060	3,901	11,738	20,700
Regional	Arterial	862	42	24	928
	Collector	1,741	200	219	2160
	Local	120	8	3	131
	Total	8,852	3,860	24,612	37,325
Total Length of Roads in Study Area (³)					438,000⁽³⁾

(1) Road type as provided by Land Information Ontario.

(2) Refer to Section 6 for definition of low-, medium-, and high-risk flooding.

(3) Total length of Land Information Ontario Road centerline layer within the Town of New Tecumseth municipal boundary, including boundary roads (e.g., Adjala Tecumseth Townline), not including provincial highways (Highway 9 and Highway 89).

Table 21 Land Information Ontario Road Classes

Road Type	Road Type Description
Arterial	A major thoroughfare with medium to large traffic capacity
Collector	A minor thoroughfare mainly used to access properties and to feed traffic with right-of-way.
Local	Includes both Local/Strata and Local/Street. Local/Strata: A low speed thoroughfare dedicated to providing access to properties with potential public restriction, trailer parks, First Nations, strata or private estates. Local/Street: A low speed thoroughfare dedicated to providing full access to the front of properties.

Source: MNRF 2020

The road vulnerability analysis was completed at all roads including provincial highways that are not owned or maintained by the Town; flood risk at Highway 9 and Highway 89 were identified separately in Table 22 below. The results shown in Table 22 indicate that Highway 9 and Highway 89 meet the MTO overtopping criteria for flow over roadways (WC-13 and SD-7, MTO 2008) as there are no highways at risk of flooding until the Regional event; watercourse crossings on Highway 9 and Highway 89 would be designed to convey the 25-year to 100-year design storm depending on the road classification and structure dimension (MTO 2008).

Table 22 Flood Risk at Provincial Highways

Modelled Event	Length of Highway at Risk (m) ⁽¹⁾			
	Low	Medium	High	Total
5-year	-	-	-	0
10-year	-	-	-	0
20-year	-	-	-	0
50-year	-	-	-	0
100-year	-	-	-	0
Regional	120	8	3	131
Total Length of Provincial Highways in Study Area ⁽²⁾				22,898 ⁽²⁾

(1) Refer to Section 6 for definition of low, medium, and high-risk flooding.

(2) Total length of Land Information Ontario Road layer (Expressway/Highway Road Class) within the Town of New Tecumseth municipal boundary, including boundary roads (Highway 9 and Highway 89).

The railways within the study area were analyzed in the same manner as the roads. Since ownership and maintenance of the railways is not within the Town’s jurisdiction, these results are intended as supplementary information only.

Table 23 Flood Risk at Railways

Modelled Event	Length of Rail at Risk (m) ⁽¹⁾			
	Low	Medium	High	Total
5-year	182	65	21	267
10-year	203	77	25	305
20-year	207	77	26	310
50-year	201	86	30	317
100-year	199	89	30	318
Regional	484	301	153	937
Total Length of Railways in Study Area ⁽²⁾				54,495

(1) Refer to Section 6 for definition of low, medium, and high-risk flooding.

(2) Total length of all Land Information Ontario Railway layer within the Town of New Tecumseth municipal boundary.

The low-, medium-, and high-risk categories identified in the tables above are based on risk to life criteria for healthy adults as defined in Section 6. There is no standardized set of flood risk criteria related to road, bridge, or culvert damage. While road washout may be a function of flood depth and velocity, other important factors affecting road stability during a flood include soil type, road construction methods and materials, and current physical condition of the road, among others. Furthermore, roads can be damaged during a flood due to erosion at the embankment without any flow on the road itself. Therefore, the identified lengths of inundated roads do not necessarily indicate the length of road repairs required after a flood and therefore these values may not accurately represent the complete damage.

8.3.1 Average Annual Damages - Roads and Rails

The length of inundated roads can be used by the Town to quantify potential damage costs in any given storm event. As discussed above, there are many variables that can contribute to flood-related road damage. These not only include variables related to flood characteristics, but also the physical properties of the roadway itself.

The results of this study provide the amount of inundation and relative flood risk, but the total amount and type of resulting damage cost is unknown. Roadway flood damage

repair costs could consist of minor embankment and/or shoulder erosion, surface erosion or lifting, or even full road washout. The costs associated with repairs for these additional damages vary widely. The Town’s 2018 Road Needs Study Update indicates that road repair costs range from \$130/m for resurfacing low traffic roads to \$2,300/m for full reconstruction of urban arterial roads (R. J. Burnside 2019). Further information would be required regarding road cross-section (rural versus urban), number of lanes, surface treatment, and traffic loads in order to assign a maintenance class in accordance with the Road Needs Study. However, these additional data would not change the uncertainty regarding type and extent of repairs required following roadway inundation. The results of this study are intended to be used as a screening for future study and do not provide sufficient detail to be used for assigning dollar values to roadway repair.

Table 24 summarizes the average length of roads that experience high risk flooding and any risk (i.e., total of low-, medium- and high-risk flooding) on an annual basis. These lengths do not refer to specific segments of road, but instead reflect the length of road anywhere in the study area that could be expected to be impacted in any given year if an average were taken over many years.

Table 24 Average Annual Risk to Roads and Rails

Road Type	Average Annual Road Risk (m)	
	High	Any Risk
Arterial	9	191
Collector	15	550
Local	2,677	6,840
Highway	0	0
Rail	19	256

9 Flood Risk Areas

Given the size of the study area and the uncertainty of the hydrologic inputs, the 2D HEC-RAS model developed for this project is best used as a high-level screening tool that can indicate areas requiring further study. Matrix reviewed the 2D HEC-RAS results in detail to identify areas with high-risk flooding based on the risk criteria provided in Section 6. Individual flood risk areas will need to be studied in more detail to determine if mitigation is required. The developed 2D HEC-RAS model is robust and can be used as a base to evaluate flood remediation design options in later studies.

9.1 Culvert Capacity

During the flood characterization task, the flood risk areas identified in Table 25 were found to have culvert capacity limitations. Note that the summaries provided herein are not meant to review culvert capacity relative to the Town's design standards, as these were discussed in Section 6.2.2, but rather these areas were identified because of high-risk flooding upstream of the culverts during large storm events.

Table 25 Flood Risk due to Culvert Capacity Limitations

Flood Risk Area	Modelled Event Causing High-Risk	Triggering Criteria	Source of Flood Risk
New Culverts			
Sir Frederick Banting Road	50-year	Depth	No culvert at topographic low
Industrial Parkway and CPR	5-year	Depth	No culvert at topographic low
7 th Line at CPR and SSR	5-year	Depth	No culvert under SSR
Replacement Culverts			
14 th Line at 20 th Sideroad	Regional	Depth	Undersized culvert(s)
5 th Line at 20 th Sideroad	Regional	Depth	Undersized culvert(s)
10 th Line at 10 th Sideroad	Regional	Depth	Undersized culvert(s)
Tottenham Road at 10 th Line	50-year	Depth	Undersized culvert(s)

CPR - Canadian Pacific Railway

SSR - South Simcoe Railway

9.1.1 Sir Frederick Banting Road

Flows in Wilson Drain downstream (east) of CPR breaches its banks during the 20-year event and larger. Flow that extends beyond the main channel corridor travels southeast to a dry/intermittent local drainage feature approximately 950 m north of Highway 89 near the historic Sir Frederick Banting homestead. There is no culvert at the low point on Sir Frederick Banting Road under existing conditions. As a result, flood water builds up on the upstream (west) side of Sir Frederick Banting Road starting at the 20-year event. Roadway overtopping occurs in the Regional event and flow continues eastward through an agricultural property following the local drainage topography (Figure 31). The ponding west of Sir Frederick Banting Road extends into the residential neighbourhood

via John W. Taylor Avenue starting at the 50-year event. Flooding in this neighbourhood is shown on John W. Taylor Avenue and the connecting residential streets. It is unlikely that the major drainage system design for this urban neighbourhood considers this additional flow from Wilson Drain.

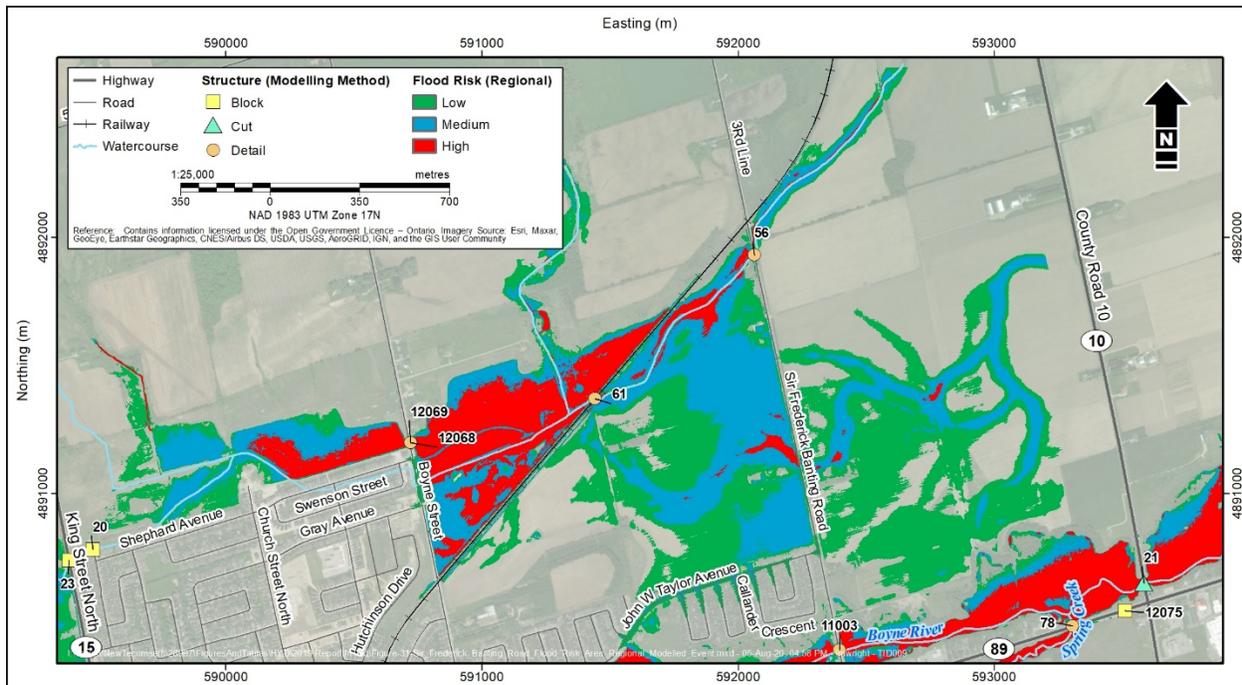


Figure 31 Sir Frederick Banting Road Flood Risk Area (Regional Event)

This location was investigated in the future condition assessment in Section 7.2.3. The results indicated that adding a culvert at the low point on Sir Frederick Banting Road could significantly reduce flood risk in the neighbourhood to the west while not increasing flood risk to properties downstream of the new culvert. The Road Needs Study Update (R. J. Burnside 2019) also indicated that brushing (the clearing of brush and vegetation along the roadside) is required along Sir Frederick Banting Road (from 140 m North of John W Taylor Avenue to the municipal boundary).

Further assessment of culvert requirements including using updated flows should be reviewed in subsequent studies if the Town wishes to pursue installing a culvert at this location. In addition, there is a storm sewer system on Sir Frederick Banting that could potentially be utilized to redirect flows away from this area. Matrix recommends that these municipal sewers be included in the urban drainage system modelling completed in Phase 2 of the DMP.

9.1.2 Industrial Parkway and Canadian Pacific Railway

High risk flooding on the west side of CPR (Figure 32) occurs under all modelled storm events. The flooding in this location originates from Upper Nottawasaga River as it spills toward the Spring Creek in the Boyne River watershed near 13th Line, details of which were discussed in Section 6.3.1. The spilled flow is trapped on the west side of the CPR and causes high risk flooding along the overland drainage path shown approximately 750 m south of Industrial Parkway. Matrix is not aware of any culverts under CPR between Industrial Parkway and 13th Line. As the ponding upstream of CPR increases, the high-risk area immediately south of Industrial Parkway also increases. The model results indicate that the railway may be overtopped 100 m south of Industrial Parkway starting at the 50-year event. This spill then continues downstream toward Spring Creek (refer to Section 9.2.2 for information on the downstream impacts).

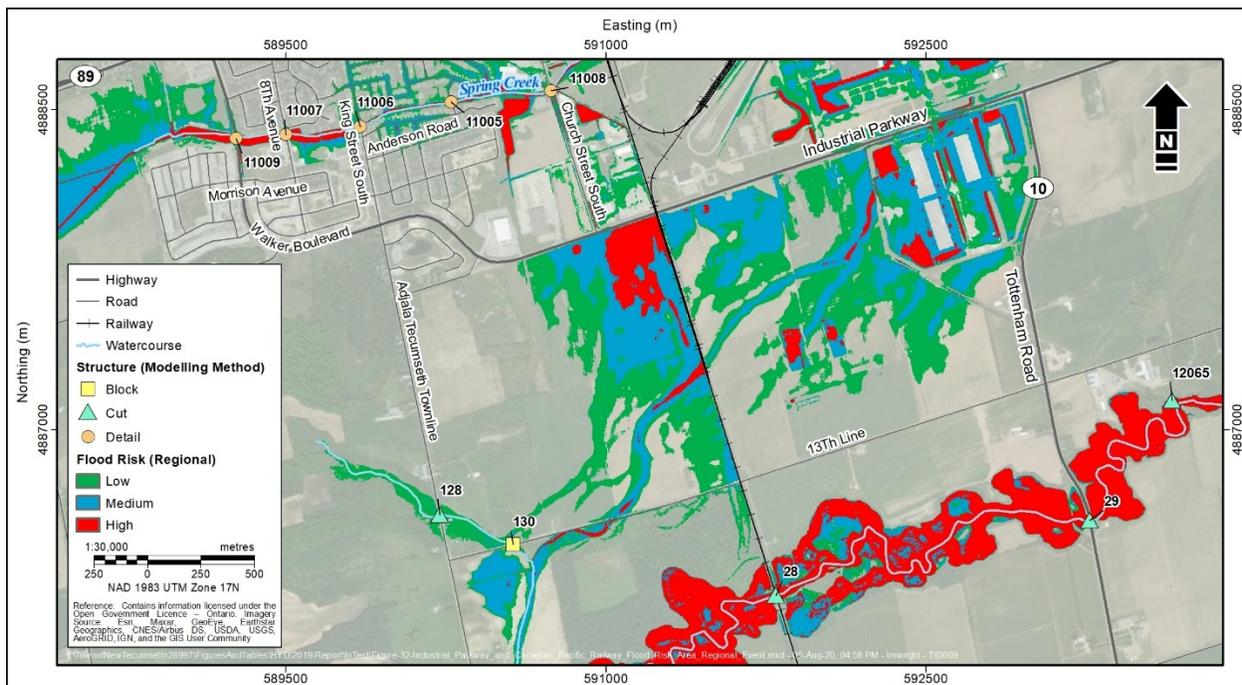


Figure 32 Industrial Parkway and Canadian Pacific Railway Flood Risk Area (Regional Event)

Future studies should be completed to better understand flooding in this area, commencing with a detailed review of subwatershed delineation, hydrology, and spill conditions from the Upper Nottawasaga River. Additional review of existing conditions should then be completed along the CPR corridor between Industrial Road and 13th Line. If there are indeed no culverts here, Matrix recommends that the Town consider constructing culvert(s) at the identified low points 100 m and/or 750 m south of Industrial Parkway. The need for ditching and other grading may also be required in coordination

with these culvert(s) and should be investigated; however, the Road Needs Study Update (R.J. Burnside 2019) makes no recommendations for ditching in this area.

9.1.3 14th Line at 20th Sideroad

A historic railway, which currently serves as the Trans Canada Trail, creates a barrier to flow on two tributaries to Cookstown Creek (Figure 33). There is a high-risk flooding area on the north side of the trail, which prevents safe access and egress for one residence. The two culverts across 14th Line were modelled in detail (MSI_132 and MSI_133). The culvert under the trail was not identified during the background review and was therefore represented using the cut method, removing the trail obstruction from the LiDAR. In the future, Matrix recommends the Town consider installing larger culverts in this area to minimize flooding, or to alter the elevations of the trail such that it does not create a barrier to flow. As this corridor is no longer used as a railway, the profile constraints are greatly reduced and therefore regrading the trail may be the most cost-effective flood mitigation solution. Due to the “cut” modelling approach applied to the trail crossing, subsequent modelling should incorporate the existing structure using the detailed approach before recommending upgrades.

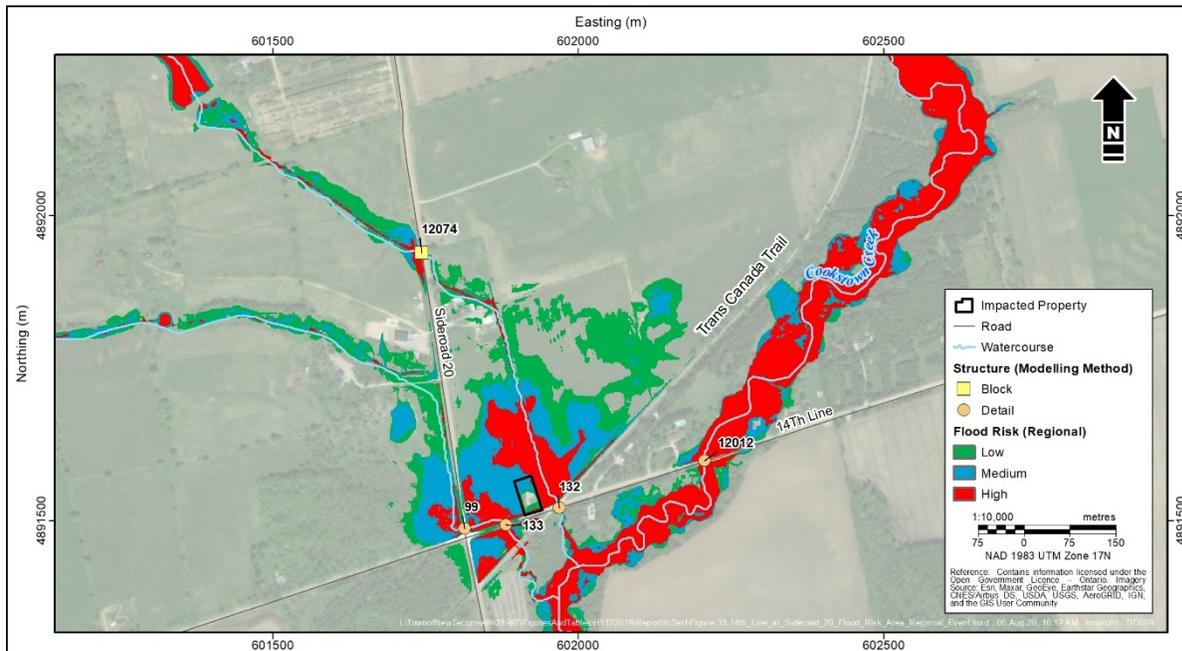


Figure 33 14th Line at 20th Sideroad Flood Risk Area (Regional Event)

The Road Needs Study Update (R.J. Burnside 2019) indicated that ditching is required along 14th Line from 20th Sideroad to the east Town boundary. Along 20th Sideroad (from 13th Line to 15th Line), a combination of ditching and brushing is recommended.

Implementing these recommendations may also improve the local drainage conditions.

9.1.4 5th Line at 20th Sideroad

The residents along 5th Line west of 20th Sideroad have reported flooding in the past along Penville Municipal Drain. Review of the 2D HEC-RAS model results indicate that the culvert at Sideroad 20 (Structure MSI_95) acts as a restriction creating backwater during all modelled events. A recent study of this area indicates that the existing culvert on 20th Sideroad conveys approximately 8 m³/s before roadway overtopping occurs (R.J. Burnside 2018). The return period flows in the R.J. Burnside report differ from the NVCA provided flows used in this study. As a result, the model results of this study do not indicate overtopping on 20th Sideroad for the 5-year through 100-year modelled events. Matrix modelled Structure MSI_95 as a larger culvert in the future condition simulation and the results are discussed in Section 7.2.2.

The model results shown in Figure 34 indicate that two properties on 5th Line are impacted by flooding during the Regional event (flow rate of 11.13 m³/s upstream of 20th Sideroad) as a result of the backwater conditions from the 20th Sideroad culvert. One of these properties experiences high risk flooding during the Regional event. Due to the discrepancy in design storm flows in this channel (NVCA flows versus R.J. Burnside flows), Matrix recommends that the subsequent stage of the DMP include a review and update to hydrology within the Town's jurisdiction to confirm peak flows before making a recommendation related to proposed culvert capacity.

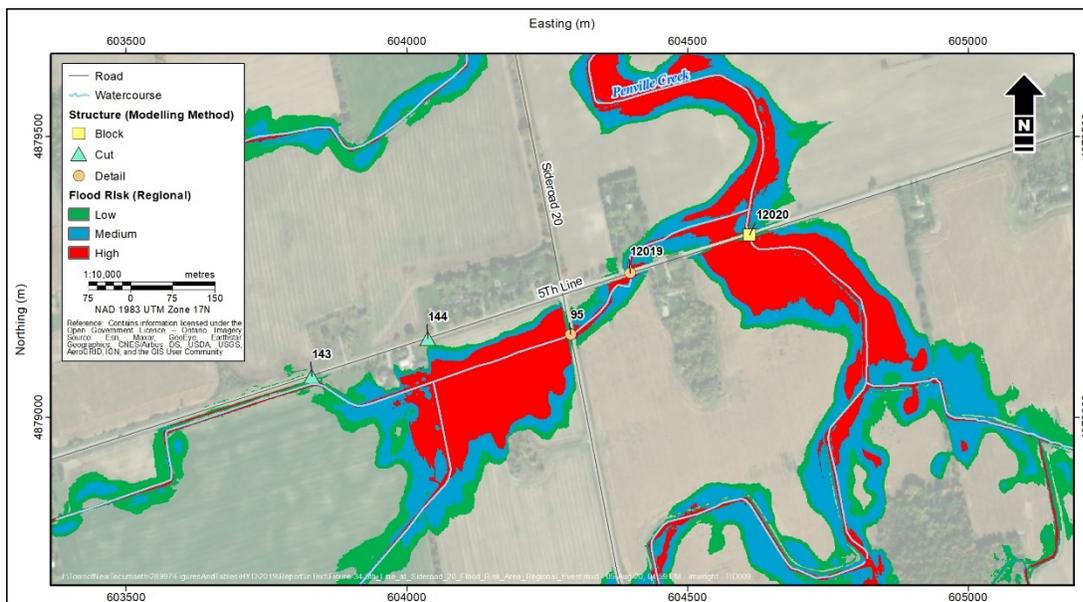


Figure 34 5th Line at 20th Sideroad Flood Risk Area (Regional Event)

The Road Needs Study Update (R.J. Burnside 2019) indicated that minor ditching is required along 5th Line Road from 20th Sideroad to the east Town boundary. Along 20th Sideroad, the study recommended minor ditching and brushing from 4th Line Road to 5th Line Road. These recommendations should be reviewed in combination with culvert upgrades.

9.1.5 10th Line and 10th Sideroad

The spill from Beeton Creek to Bailey Creek between the Trans Canada Trail and 10th Sideroad creates a high-risk flood area north of 10th Line (refer to Section 6.3.2). There are multiple agricultural properties and buildings impacted by the high-risk flooding in this area. The model results indicate that the existing bridge (Structure 12028 on 10th Sideroad) adequately conveys the modelled 20-year event on Bailey Creek and thus meet the Town’s design guidelines. However, considering the modelled spill condition, it may be beneficial to implement crossing improvements on Beeton Creek (Structure 12032, located further south near 9th Line) to reduce water elevations to below the spill elevation.

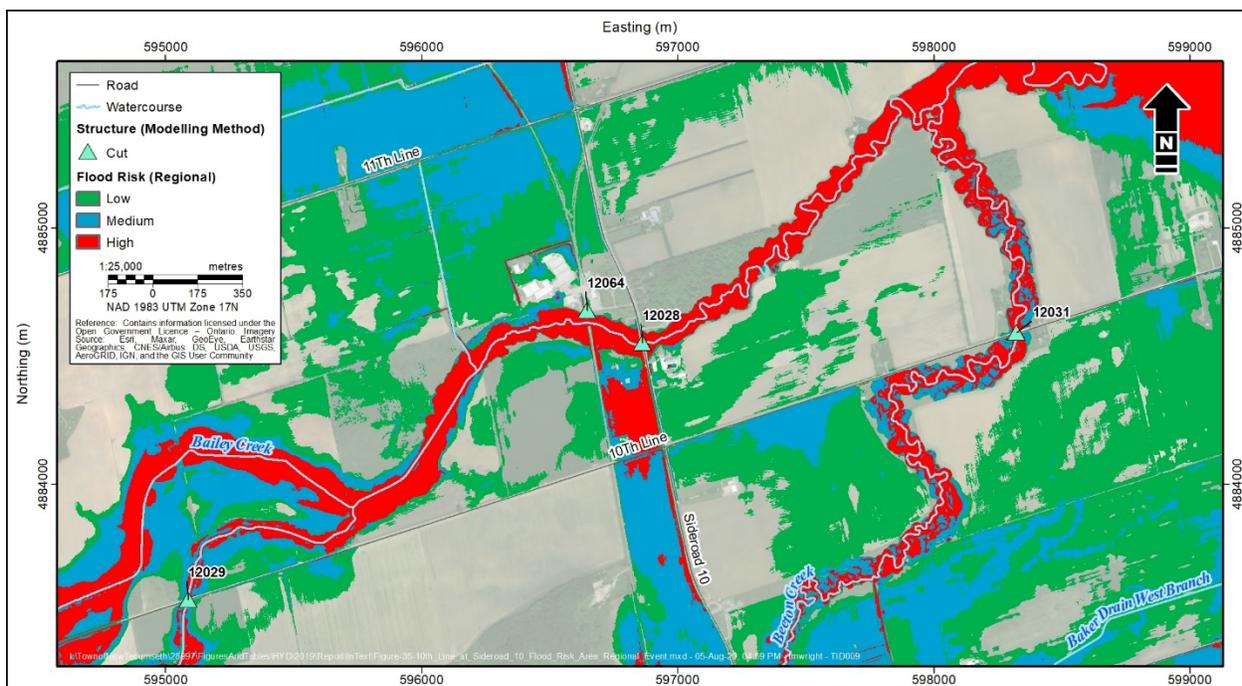


Figure 35 10th Line at 10th Sideroad Flood Risk Area (Regional Event)

The Road Needs Study Update (R.J. Burnside 2019) indicated that minor ditching and brushing is required along 10th Line from Tottenham Road to Sideroad 15; however, no recommendations were made along 10th Sideroad in this area.

A report by WSP indicates some concerns regarding the NVCA HEC-RAS representation of the 10th Sideroad bridge (Structure 12032, referred to as the Strangways bridge; WSP 2020). Further assessments in this area should confirm the bridge is adequately represented in the model. Matrix recommends that detailed review of the structures under 10th Sideroad be completed in subsequent studies including consideration for improved ditching along 10th Sideroad and potential regrading of the Trans Canada Trail.

9.1.6 Tottenham Road at 10th Line

There is a large high-risk area west of Tottenham Road near 10th Line. There are two properties on 10th Line west of the CPR and adjacent to Bailey Creek which have compromised access and egress during the 50-year event and larger. The bulk of the high-risk flooding is contained within agricultural fields. The farm buildings at the corner of Tottenham Road and 9th Line are impacted by low-risk flooding during the Regional event, but the high-risk zone does not reach the buildings (Figure 36).

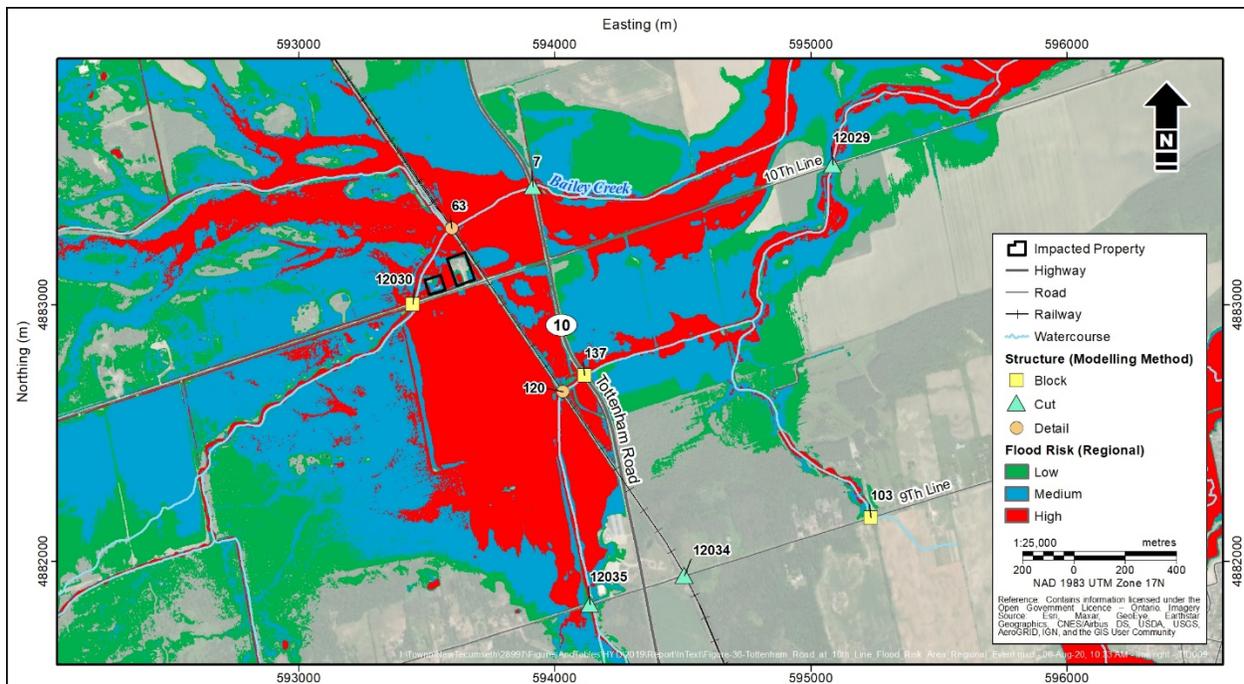


Figure 36 Tottenham Road at 10th Line Flood Risk Area (Regional Event)

The Road Needs Study Update (R.J. Burnside 2019) indicated that minor ditching and brushing is required along 10th Line from Tottenham Road to 15th Sideroad but no recommendations were made along Tottenham Road in this area.

A detailed review of the 2D HEC-RAS model results suggest that the capacity of the CPR crossing (MSI_120) and the Tottenham Road crossing (MSI_137) contribute to flooding in this area. Neither MSI_120 nor MSI_137 were included in NVCA's HEC-RAS model or available background data. MSI_120 was modelled using the "detail" approach as two 1.5 m CSP culverts estimated from imagery and LiDAR. MSI_137 was modelled using the "block" approach. Bailey Creek Swamp is a known area of flooding characterized by its natural, low-lying floodplain between two tributaries of Bailey Creek (refer to Section 9.3.4). The modelled 20-year event results show low to medium-risk flooding in this area, and thus it is believed that MSI_120 meets the Town's current design standards. However, further study of the flows and culverts in this area are recommended to confirm the cause of flooding and review potential recommendations for structure upgrades. Recommended next steps include:

- complete detailed field investigation to confirm dimensions of MSI_120 and MSI_137
- complete cross-section survey along Bailey Creek to confirm bathymetry
- determine existing condition drainage area and peak flows contributing to Bailey Creek and tributaries
- confirm existing hydraulic performance of the channel and crossings (Structures MSI_120 and MSI_137) through development of a site-specific hydraulic model or trimming the 2D HEC-RAS model to an appropriate extent and updating both structures using field data
- review potential flood reduction opportunities such as increasing channel capacity and/or culvert dimensions

9.1.7 7th Line at Canadian Pacific Railway and South Simcoe Railway

There is an area of high-risk flooding during the 5-year event adjacent to Beeton Creek west of the SSR tracks near 7th Line and CPR (Figure 37). The tributary to Beeton Creek from the west (referred to as BE-100 by NVCA) does not have an adequate outlet to Beeton Creek. Matrix was unable to locate a culvert under the SSR in this area as none were found during the background review or field inspection. While there are no buildings in this immediate area, the flooding will impact traffic on 7th Line.

The Road Needs Study Update (R.J. Burnside 2019) makes no recommendations for ditching in this area. Future studies should be conducted to confirm if a culvert is

present in this location, and to determine the feasibility of culvert upgrade or installation to provide adequate conveyance for this tributary.

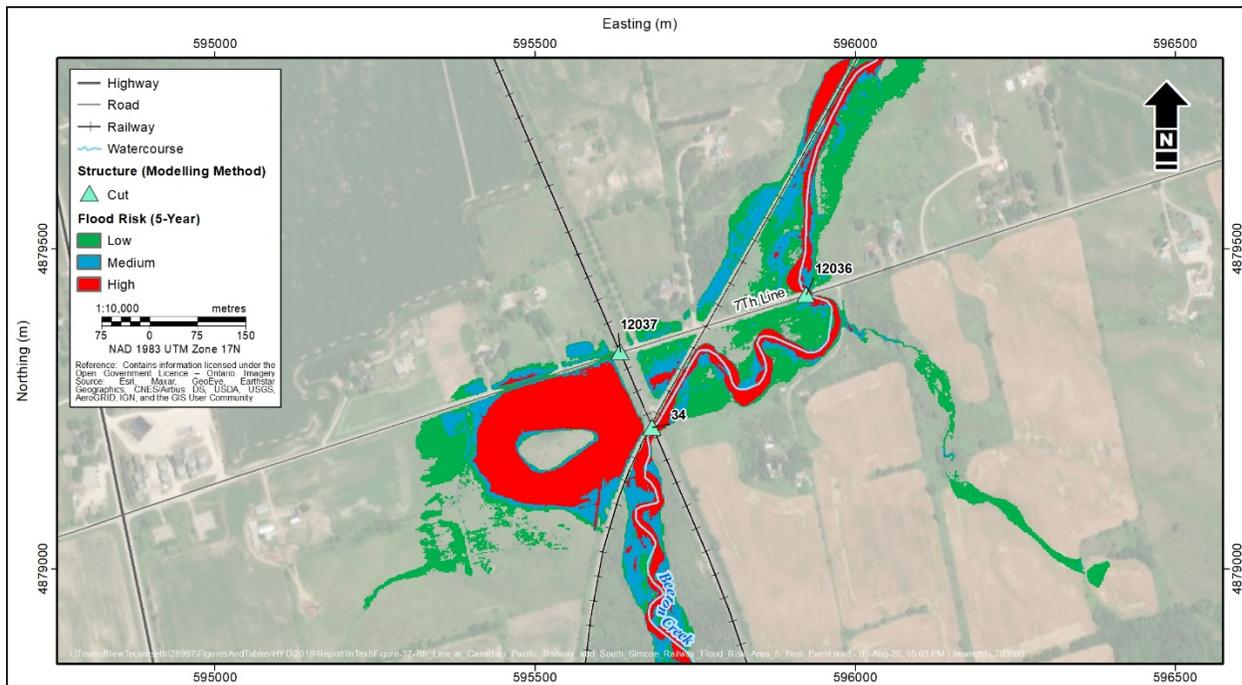


Figure 37 7th Line at Canadian Pacific Railway and South Simcoe Railway Flood Risk Area (5-year Event)

9.1.8 Blocked Culverts

The following locations with high risk flooding were identified upstream of culverts that were modelled using the blocked approach in the 2D HEC-RAS model:

- 4th Line (Mill Street W) west of Tottenham Conservation Area (Structure MSI_102)
- 4th Line 600 m east of Adjala Tecumseth Townline (Structure MSI_53)
- 4th Line 1,100 m east of Adjala Tecumseth Townline (not previously identified; no structure ID)
- Adjala Tecumseth Townline 400 m north of 3rd Line (Structure MSI_5)
- Adjala 5th Sideroad west of Adjala Tecumseth Townline (not previously identified; no structure ID)
- Highway 89 east of C.W. Leach Road (Structure MSI_122)
- CPR near Tottenham Road and Highway 9 (Structure MSI_19)
- 6th Line 1,600 m west of Tottenham Road (2 culverts in series; Structures MSI_83 and MSI_84)

The model results in these locations are impacted by the adopted modelling technique and therefore the level of conservatism must be considered when interpreting the results in these areas. In future studies, if the model is trimmed to focus on smaller areas, it is recommended that these and other blocked culverts be revised and modelled in detail.

9.2 Channel Capacity

Channel capacity limitations create another source of flooding when culvert limitations are not a factor. This includes areas where channels are not well defined or do not have capacity to convey the modelled events, leading to flow extending beyond the main channel and floodplain. Table 26 summarizes the flood risk areas identified as high risk caused by limited channel capacity.

Table 26 Flood Risk due to Channel Capacity Limitations

Flood Risk Area	Modelled Event Causing High-Risk	Triggering Criteria	Source of Flood Risk
Alliston Wastewater Treatment Plant	50-year	Depth; Depth × Velocity	Infrastructure in the floodplain
Honda Manufacturing Plant	Regional	Depth	Undefined channel
9 th Line at 15 th Sideroad	5-year	Depth	Channel capacity causes spill to here. No culvert was identified at topographic low
10 th Line East of Tottenham Road	100	Depth; Depth × Velocity	Road in floodplain
6 th Line at Tottenham Road	Regional	Depth × Velocity	Road in floodplain
Schomberg north of Highway 9	10-year	Depth	Buildings at top of valley slope

9.2.1 Alliston Wastewater Treatment Plant

Pockets of high risk flooding are shown at the Alliston Wastewater Treatment Plant (WWTP) as a result of high water elevations in the Boyne River and the location of the WWTP within the floodplain (Figure 38).

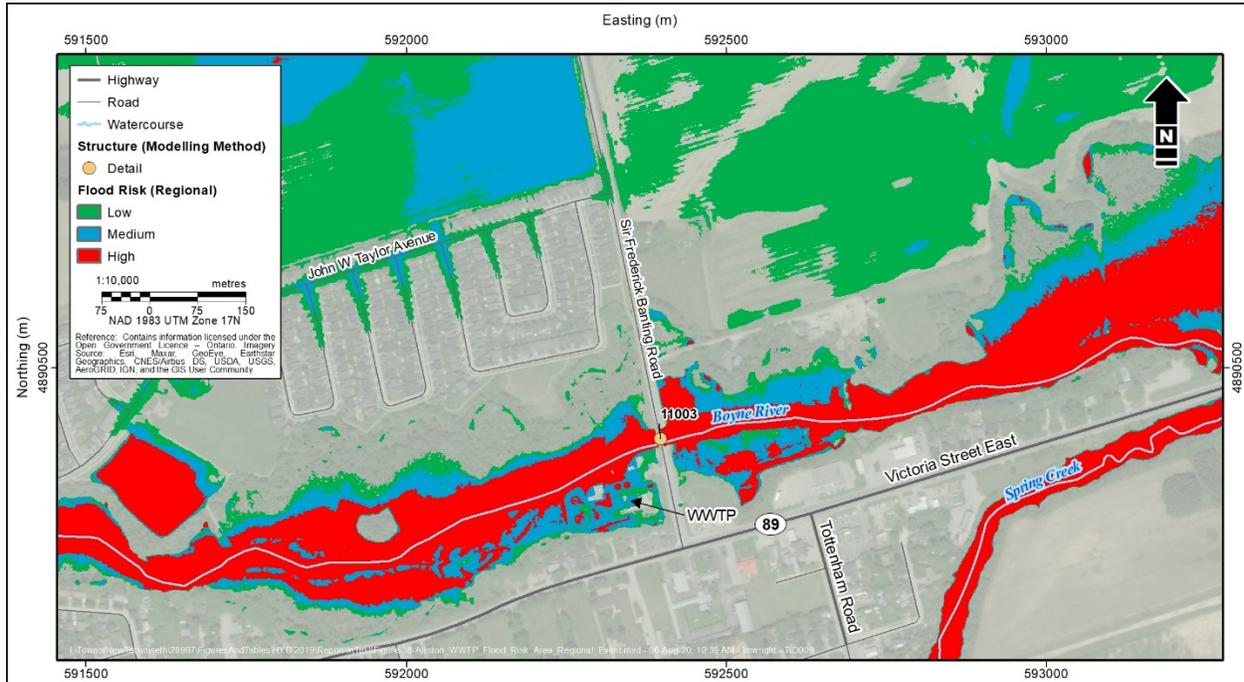


Figure 38 Alliston Wastewater Treatment Plant Flood Risk Area (Regional Event)

Elevated flood risk at WWTPs occurs in many municipalities due to the required proximity of WWTP infrastructure to watercourses. The bridge at Sir Frederick Banting Road is not the cause of flooding at this location and does not exacerbate the water elevations; the modelled water elevations in the Boyne River are high downstream of Sir Frederick Banting Road as well.

The Alliston WWTP, like many others, was constructed within the floodplain. While this is not a significant flood risk that needs to be mitigated, it was identified for the Town to ensure appropriate preventative measures are in place to protect against potential flooding. These preventative measures may include an emergency response plan in the event of a flood; floodproofing electrical, mechanical, and other essential equipment; and/or having emergency backup power in place.

9.2.2 Honda Manufacturing Plant

Downstream of the Upper Nottawasaga River spill location (refer to Sections 6.3.1 and 9.1.2), the spilled flow from the Upper Nottawasaga River continues through the industrial area crossing Industrial Parkway near the main entrance to the Honda manufacturing plant (Figure 39). No culverts were identified in this area. As the flow approaches the industrial area, it is somewhat concentrated along the observed overland drainage path; however, there is no defined channel through the industrial area leading to widespread flooding.

It is possible that during the design and construction of this area, a portion of the channel was buried (i.e., installed as a pipe belowground) to maximize the use of the available lands. Modelling piped private infrastructure was not within the scope of this project, but could be investigated in future studies, if desired. Incorporating any existing underground drainage infrastructure into a model in this area could show a reduction in flood risk, however, as the spill from the Upper Nottawasaga River was previously unidentified, it is likely that existing infrastructure is undersized.

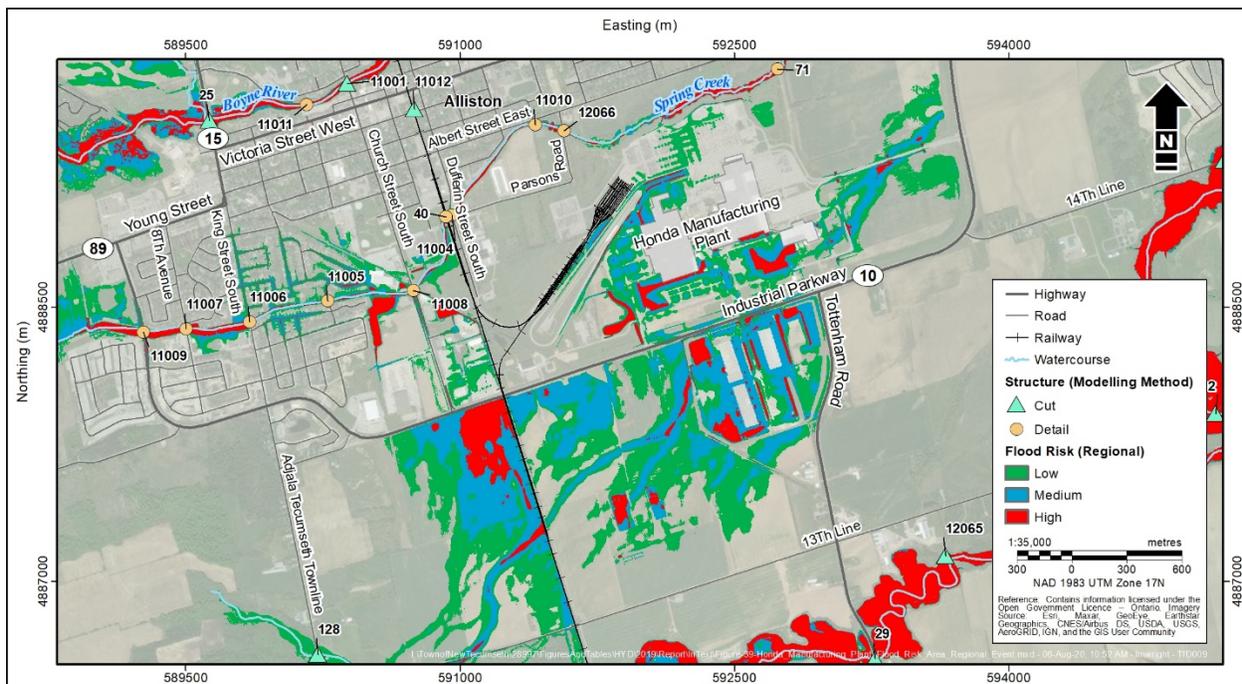


Figure 39 Honda Manufacturing Plant Flood Risk Area (Regional Event)

A future 62 ha industrial subdivision, known as the Walton Subdivision, is proposed south of the Industrial Road and east of CPR (C.F. Crozier 2016). The plans for this area include 22 ha of Town owned SWM property for peak flow control as well as water quality control. We recommend that the urban drainage system modelling completed in

Phase 2 (e.g., PCSWMM) incorporate the recommendations from the Walton Subdivision along with the storm infrastructure within the Honda manufacturing plant property (if drawings are available) to confirm flood risk.

9.2.3 9th Line at 15th Sideroad

This flood risk area north of 9th Line is the same location at which spill from Baker Drain to Beeton Creek occurs (refer to Section 6.3.3); however, this section discusses the flooding which remains in the Baker Drain subwatershed. There is a small area of high risk flooding in the agricultural area south of 9th Line and west of 15th Sideroad, which lies within a widespread area of low and medium risk flooding caused by insufficient channel capacity in the Baker Drain East Branch (Figure 40).

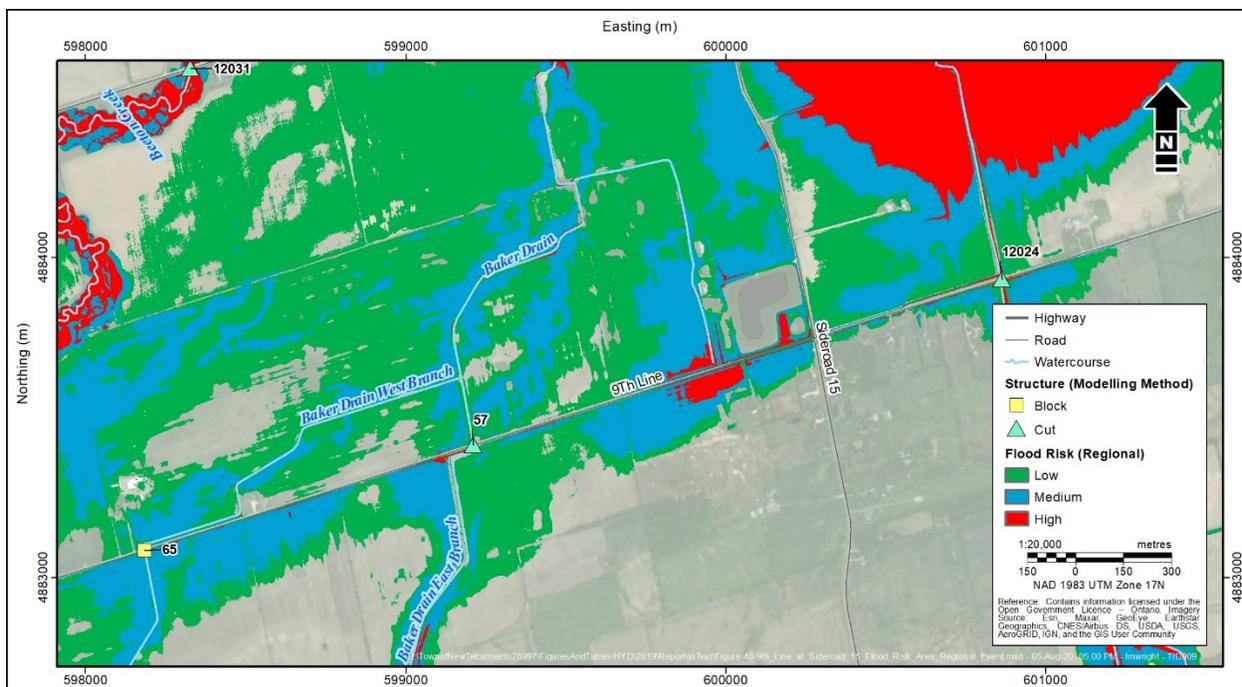


Figure 40 9th Line at 15th Sideroad Flood Risk Area (Regional Event)

The Baker Drain East Branch has berms along each bank as discussed in Section 6.2.1.4 which limit channel capacity and the ability for overland flow to enter (or re-enter) the channel. The high-risk flood area is located at a topographic low at which no culvert was identified by Matrix. Confirmation of culvert presence or consideration of a new culvert at this location is recommended for future study. North of 9th Line, the widespread low-risk flooding continues as a result of insufficient channel capacity. Ditching was not recommended in this area (R.J. Burnside 2019); however, the need for increased ditch capacity should also be reviewed here in the future. Additionally, future studies should review the contributing flow to this area, assess options for flood

mitigation, and provide recommendations for mitigation such as channel and culvert dimensions.

9.2.4 10th Line East of Tottenham Road

Section 9.1.6 summarized the flooding west of Tottenham Road near 10th Line caused by culvert capacity limitations. Downstream of that location, the high-risk flooding area continues for the Regional event as shown in Figure 41. East of Tottenham Road, the flooding is caused by insufficient channel capacity and/or the fact that 10th Line is in the floodplain. High risk flows overtopping 10th Line will prohibit safe passage during the Regional event. No overtopping of 10th Line is shown during the 100-year and smaller events. Flows from Bailey Creek encroach on the 10th Line right-of-way during all modelled events. Considerations for future studies should review peak flows as well as the scour/erosion potential in more detail due to high depth and velocity shown in the modelling results for this area.

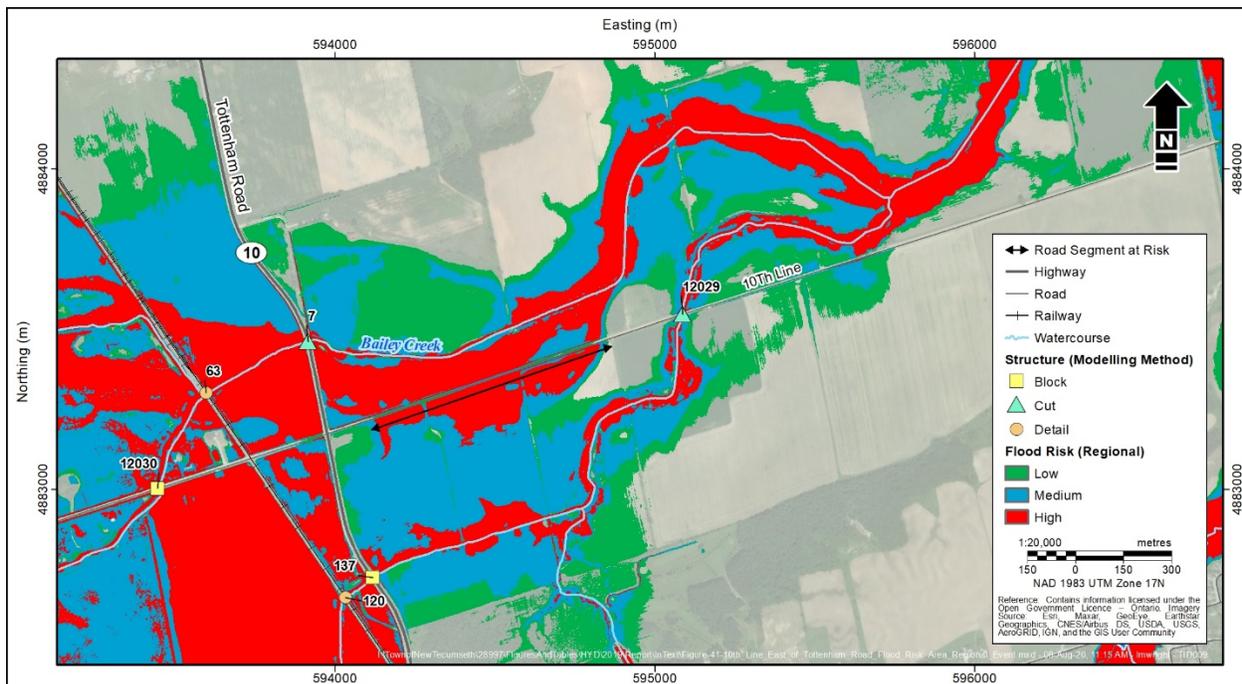


Figure 41 10th Line East of Tottenham Road Flood Risk Area (Regional Event)

9.2.5 6th Line East of Tottenham Road

Similar to the flood risk area above, 6th Line east of Tottenham Road is situated in the floodplain of Beeton Creek. The flows on the north bank of Beeton Creek may create erosion/scour issues for the road base and/or asphalt of 6th Line. Modelling results in this area indicate high risk depth x velocity starting at the 20-year event. Furthermore, the flow overtops 6th Line during the Regional event impeding safe travel as shown in Figure 42. In fact, Town staff have reported that the bridge downstream at 6th Line (Structure 12039) was almost washed out during the 2017 flood. Matrix recommends that, following hydrology updates, the Town consider studying the scour/erosion potential in more detail due to the high depth and velocity shown in modelling results for this area.

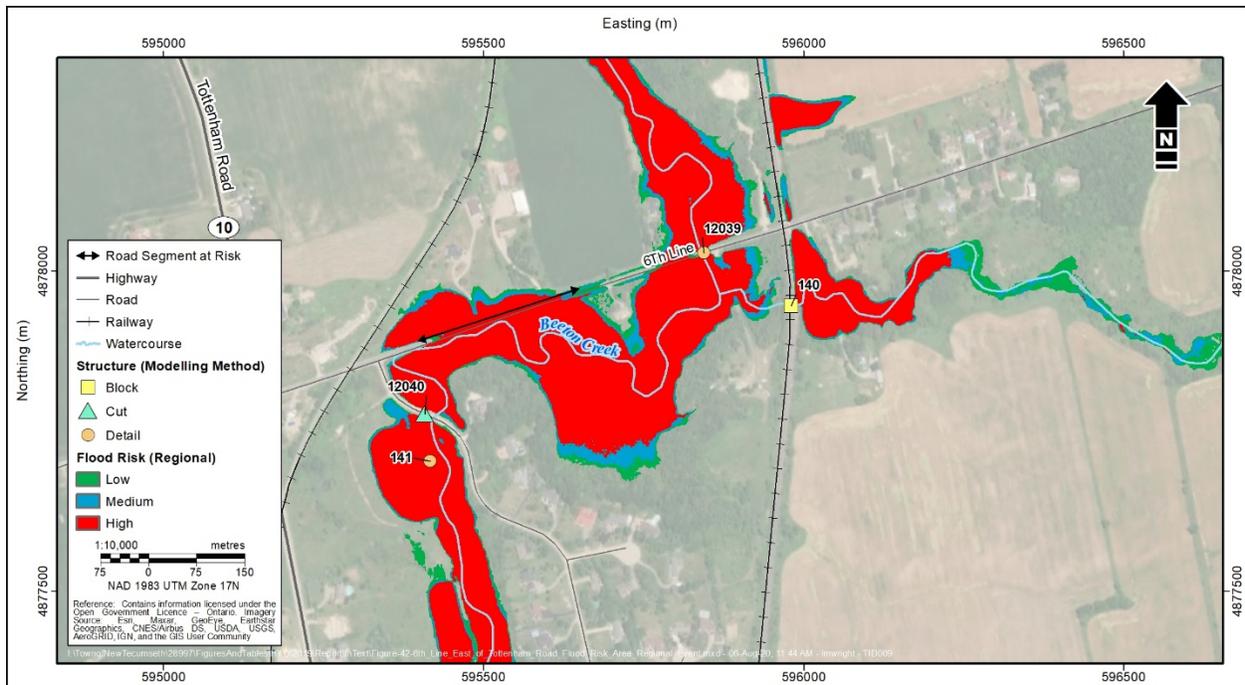


Figure 42 6th Line East of Tottenham Road Flood Risk Area (Regional Event)

9.2.6 Schomberg River North of Highway 9

Historic meanders of the Schomberg River have left a cut-off meandering floodplain adjacent to the current channel which is now fairly straight. A primary example of this situation is shown in Figure 43. During the Regional event near County Road 27 north of Highway 9 flow in the floodplain creates areas categorized as high risk where the floodplain meanders. This flooding is currently not impacting any buildings as they are situated outside the historic floodplain; however, there is a residential area with houses quite close to the top of bank. Review of existing floodlines suggests that LSRCA is likely already aware of the risks in this area.

The Schomberg River bridge at County Road 27 was modelled using the cut approach in the 2D HEC-RAS model. There appears to be an additional relief culvert about 140 m north of this bridge that was cut from the LiDAR DEM allowing flow to freely pass through. If any future studies and/or modelling are completed in this area, additional information on this relief culvert should be included in the assessment to ensure existing conditions are well-represented.

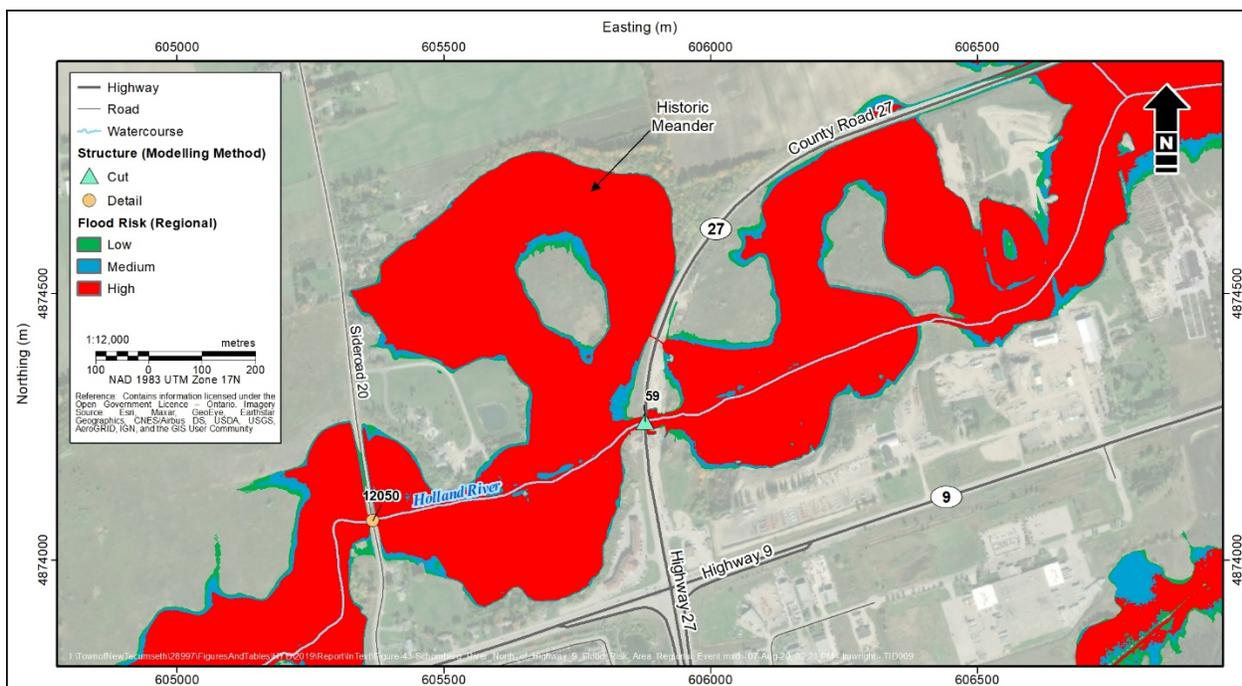


Figure 43 Schomberg River North of Highway 9 Flood Risk Area (Regional Event)

9.3 Previously Identified At-risk Areas

The locations summarized in Table 27 are known areas of flood concern within the study area and are discussed in detail in this section.

Table 27 Flood Risk at Previously Identified Locations

Flood Risk Area	Modelled Event Causing High-Risk	Triggering Criteria	Source of Flood Risk
Alliston at Spring Creek	Regional	Depth	Undersized channel
Beeton at Hendrie Drain	100-year	Depth	Undersized channel
Beeton Flats	5-year	Depth	Glacial lake
Bailey Creek Swamp	Regional	Depth	Wetland area

9.3.1 Alliston - Spring Creek

A residential area west of Church Street adjacent to Spring Creek was previously identified as a flood vulnerable area under the Regional event (NVCA 2018). While the results of the 2D HEC-RAS model indicate flooding on streets and some properties within this neighbourhood, the extents and severity of modelled flood risk are not as high as previously indicated by NVCA. The results of the 2D HEC-RAS model does not indicate flooding from the 100-year event in this location; however, safe access and egress will be limited during the Regional event along a number of residential streets (Figure 44). The flooding in this area is not contained to the public rights-of-way and encroaches onto private property.

At this time, it is unknown why the results of this modelling differ from that previously reported by NVCA. Further review of this area is recommended in Phase 2 of the DMP incorporating the urban drainage system and hydrology updates. However, as previously noted that NVCA's floodline will continue to be the regulated flood extents in this area until NVCA completes a comprehensive floodplain mapping update for its jurisdiction.

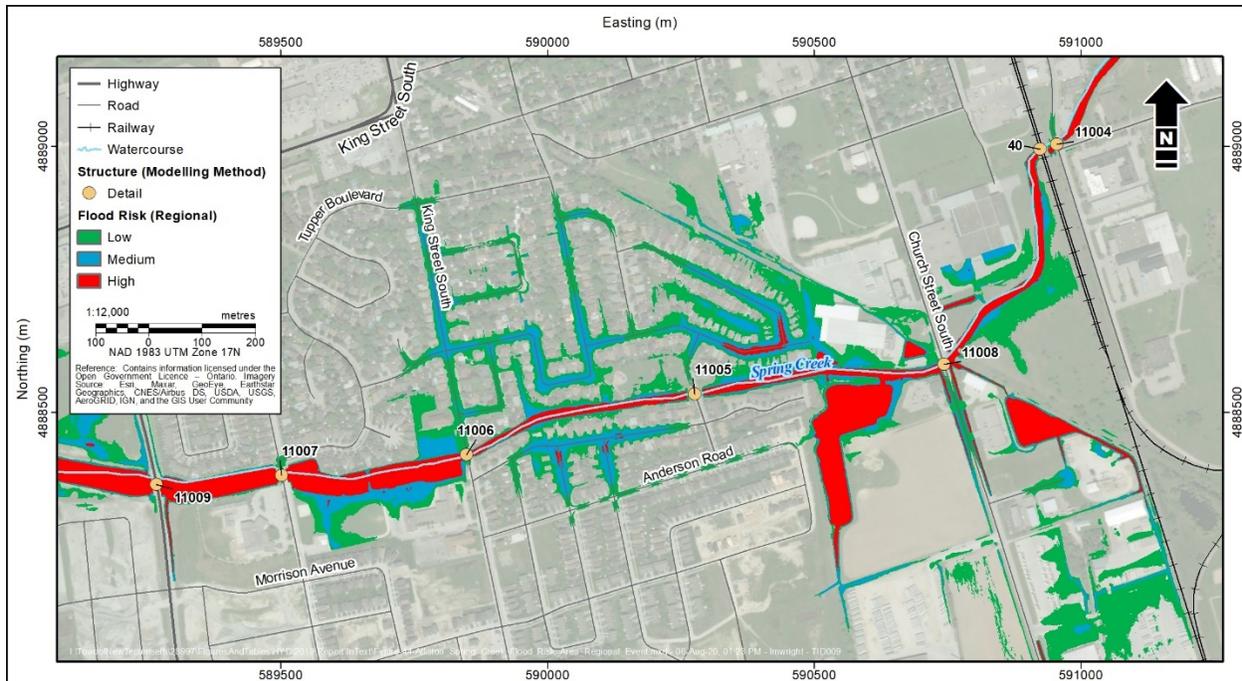


Figure 44 Alliston - Spring Creek Flood Risk Area (Regional Event)

9.3.2 Beeton - Hendrie Drain

The Town has experienced flooding issues related to the Hendrie Drain through Beeton. The results of the 2D HEC-RAS model confirm the elevated flood risk in this area. The model results shown in Figure 45 indicate high risk flooding occurs during the Regional event in the yards between Haines Street and Centre Street, as well as in the open area south of Stewart Street. While there is flooding on many streets in this area, it is categorized as low risk and thus provides safe vehicular and pedestrian travel for access and egress purposes.

In addition, the primary flow path for a tributary to Baker Drain West Branch appears to be cut off due to recent development north of Main Street and east of Patterson Street. Based on the current topographic data, this flow has no defined outlet and thus spills along Main Street and Patterson Street to Hendrie Drain. It is likely that underground infrastructure was installed in coordination with this development; however, the investigation of such infrastructure needs to be investigated as part of future phase of the study.

Additional consideration within Beeton includes the existing sewer system and overland flow drainage system (ditches) which are not included in this model. Due to the sensitivity of this area, additional review of the inflows from the south and use of smaller

catchments within the community are warranted. Beeton will be of key importance for more detailed modelling using a dual drainage model in Phase 2 of the DMP.

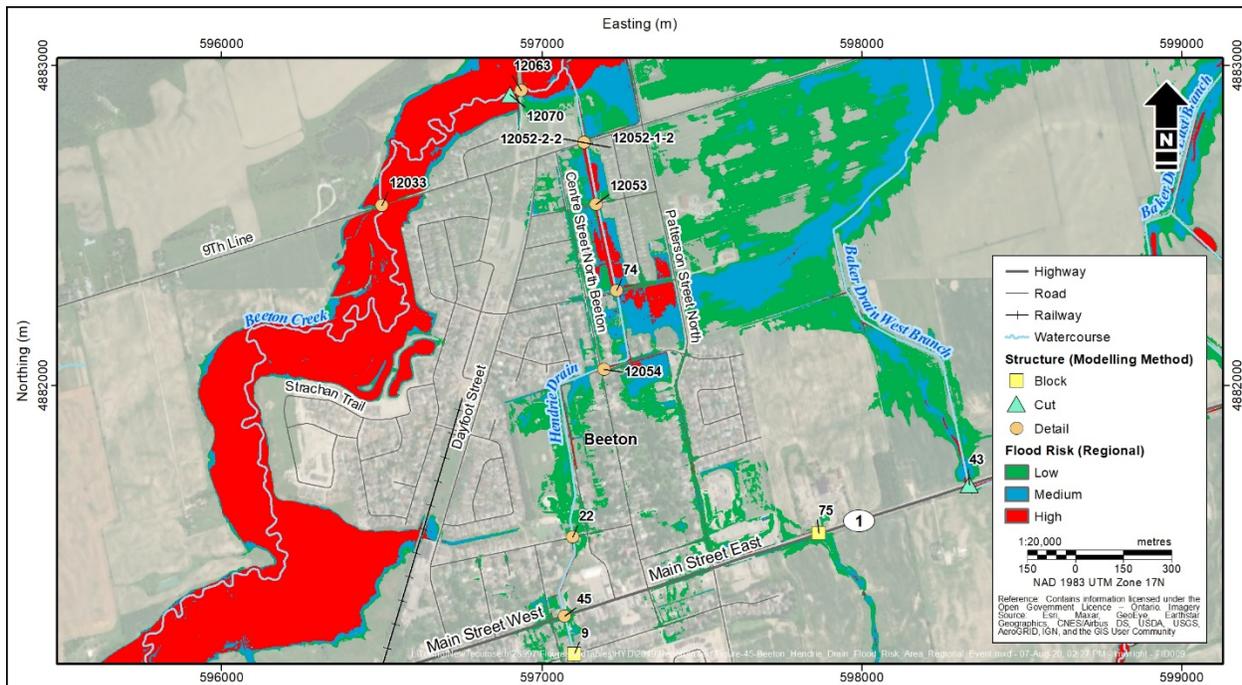


Figure 45 Beeton - Hendrie Drain Flood Risk Area (Regional Event)

9.3.3 Beeton Flats

The area known as Beeton Flats is a large natural online storage feature that covers an area generally bounded by 13th Line, Sideroad 10, 9th Line, and Highway 27 and is a remnant of a glacial lake (Cannon n.d.). As expected, flood risk in this area is high during all modelled storm events; the Regional flood risk results are shown in Figure 46. Because this area behaves like a lake during large flood events; the water depths are high, but velocities are low. There is no feasible remedy for this flood risk area due to the size and depth of flooding. Mitigation for flood risk in this area includes education for both emergency responders and the public related to potential for inaccessible roads during high flow events. Matrix also recommends limiting development in this area to prevent population increase within a high-risk flood zone.

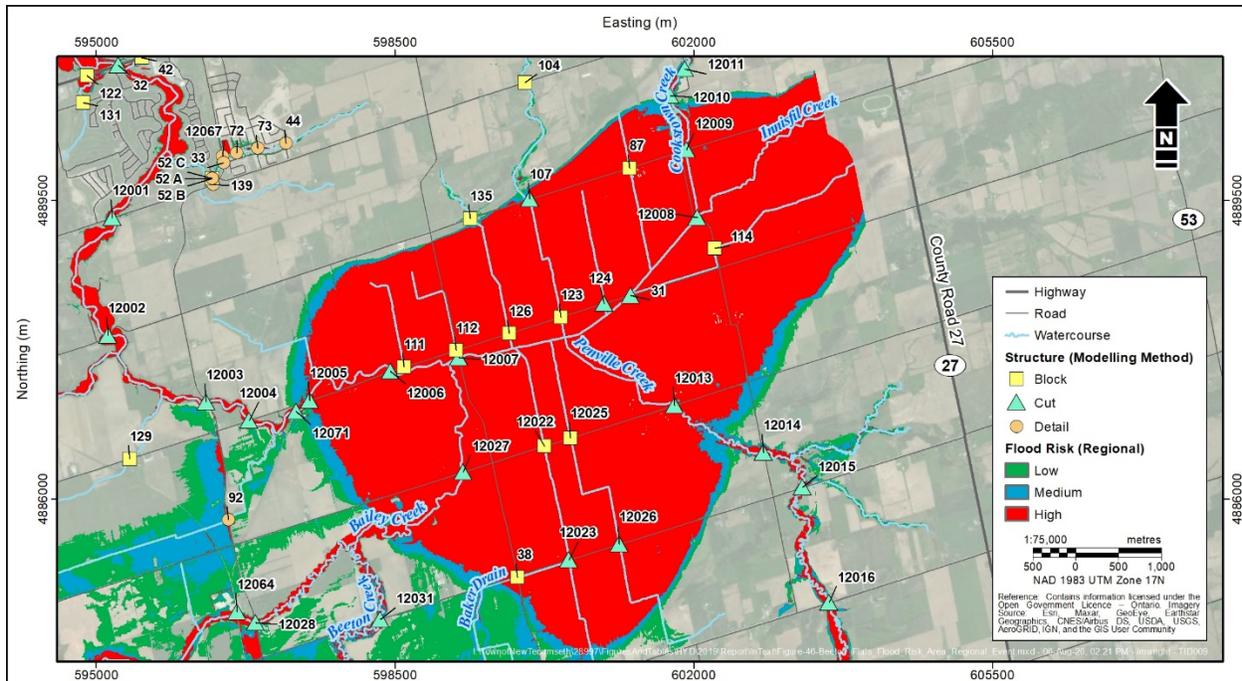


Figure 46 Beeton Flats Flood Risk Area (Regional Event)

9.3.4 Bailey Creek Swamp

The Bailey Creek Swamp is an area of known flooding between two tributaries of Bailey Creek on the west boundary of the Town (Cannon n.d.). Much of the high-risk flood areas present here are localized and have the potential to be remediated and were described in other flood risk areas above. However, there is also low and medium risk flooding throughout the Bailey Creek Swamp as frequently as the 10-year event.

Regional modelled results are shown in

Figure 47. There is no feasible remedy for this widespread low and medium flood risk due to the size and natural characteristics of this area. Flooding here is not caused by road grading or other anthropogenic means. Mitigation for flood risk could include education for both emergency responders and the public related to the potential for inaccessible roads during high flow events. Matrix also recommends limiting development in this area to prevent population increase within this flood risk zone.

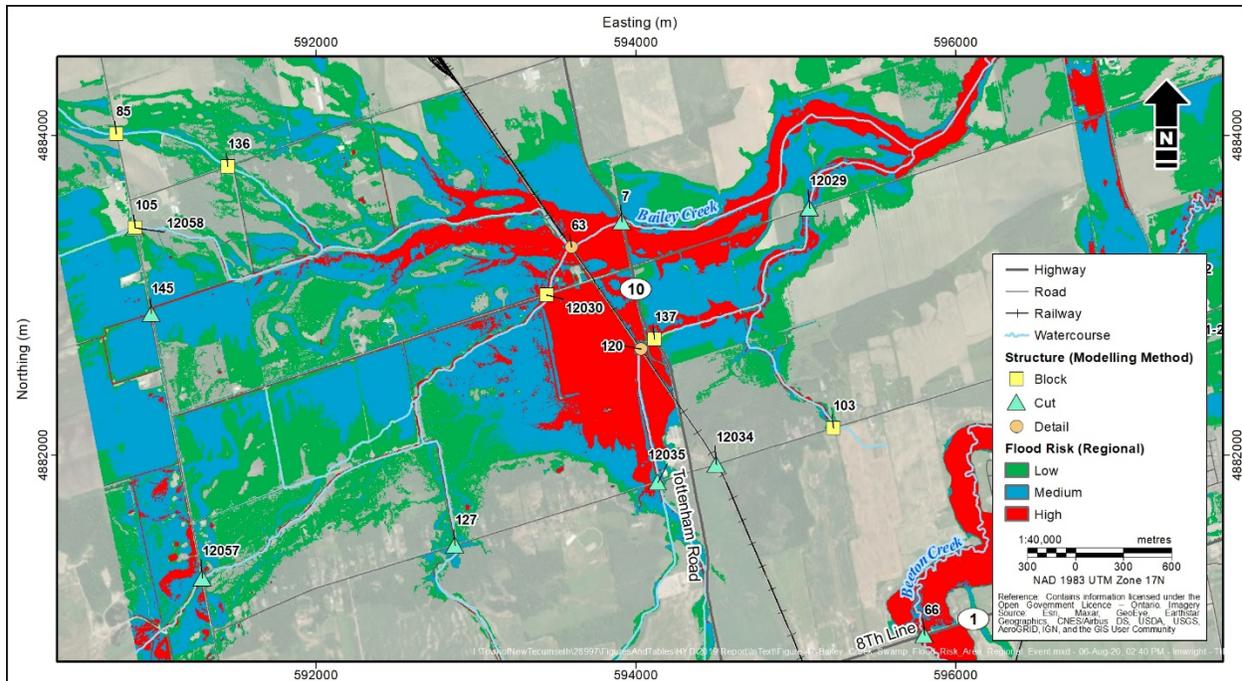


Figure 47 Bailey Creek Swamp Flood Risk Area (Regional Event)

10 Mitigation Planning

The Town of New Tecumseth is a thriving municipality in close proximity to the Greater Toronto Area. With rural and agricultural land uses currently covering much of the Town’s jurisdiction, there is an opportunity for further economic growth and development. Following the flooding that was observed after the June 2017 major storm event, the Town recognized that review of flood mitigation opportunities in its jurisdiction are required to ensure that its current and future residents are not at undue risk to flooding. The findings of the Phase 1 flood risk assessment presented in Section 6 provides the Town with a high-level understanding of existing flooding conditions within its jurisdiction. This section presents potential flood mitigation measures for areas with identified flood risks. Matrix recommends that these flood risk areas be assessed in further detail in subsequent phase(s) of the DMP.

Generic remediation measures for these sources of flooding are listed below, many of which would require additional detailed engineering studies and coordination with conservation authorities and other stakeholders. Details of specific flood remediation measures for each identified flood risk area are provided in Table 28.

- remediation for flooding related to culvert capacity:
 - ✦ construction of new culvert(s) at topographic low point(s)
 - ✦ replacement of undersized culvert(s) with larger structures
 - ✦ addition of relief culvert(s) alongside existing culvert(s)
 - ✦ regrading of roads or trails to reduce obstructions within the floodplain
 - ✦ general condition assessment of structures for coordination with capacity assessments
 - ✦ development of a general maintenance program including cleaning of debris / accumulated sediment within and around culverts

- remediation for flooding related to channel capacity:
 - ✦ reconnection of incised channels to floodplain to increase conveyance capacity and promote energy dissipation (this could be helpful where berms are currently confining channels)
 - ✦ creation of increased channel cross-section for channels which flood during frequent storms (this is not typical in natural channels, but could be an issue in constructed municipal drains)
 - ✦ general maintenance of channels within public jurisdiction (ditch cleaning, etc.)

- remediation for flooding cause by spills:
 - ✦ at the sources of the spill; refer to channel or culvert capacity items
 - ✦ at the receptor of the spill; be aware of the incoming flows and plan/design accordingly

- general maintenance of drainage system within public jurisdiction
 - ✦ ditch cleaning, brushing, and/or regrading

A preliminary mitigation plan has been provided in Table 28 including next steps for assessment as well potential environmental assessment (EA) requirements. Completing detailed studies of individual mitigation measures on their own may be costly to the Town and therefore Matrix recommends that these assessments be combined in a comprehensive study. Assessment of the mitigation measures as identified can be captured following Schedule B of the Municipal Class EA process. Therefore, it is recommended that these be included as part of the subsequent phase(s) of the DMP. In addition, to complete a detailed assessment of these areas a comprehensive hydrology update within the Town's jurisdiction would be beneficial. Matrix recommends that this hydrology update be included as part of the subsequent phase of the DMP.

The potential flood mitigation measures are based on the primary sources of flooding identified by Matrix from the existing condition modelling results. The mitigation plan prepared for each of the flood risk areas includes identification of next steps and requirements for additional studies. A priority ranking of the flood risk areas is provided based on severity of flood risk and frequency of flooding as well as estimated flood damage costs. Note the flood risk areas were organized in the table by the identified priority ranking (high to low) and therefore the order does not match the layout of headings within Section 9.

Table 28 Flood Risk Areas Mitigation Plan

Flood Risk Area and Description	Priority Ranking	Comments	Potential Implementation Timeline	Potential EA Requirements ⁽¹⁾	Next Steps and Potential Remedial Measures
Culvert Capacity - New Culverts					
<p>Sir Frederick Banting Road (Section 9.1.1)</p>	<p>High</p>	<p>This would be a small project with limited scope but with potential to provide significant remediation to modelled flood risk on agricultural lands and residential neighbourhoods.</p> <p>There are stormwater management (SWM) features upstream of this location as well as storm sewers on Sir Frederick Banting Road and John W. Taylor Drive that may already provide some flood relief in this area. An urban drainage system model (PCSWMM) would provide a more detailed understanding of flooding at Sir Frederick Banting Road.</p> <p>Future works in this area could include a new culvert at the topographic low point. It is unknown at this time if additional conveyance (i.e., a new ditch or storm sewer) would mitigate potential increases in downstream flood risk.</p>	<p>Short term</p>	<p>Urban drainage assessment: Drainage Master Plan (DMP) Phase 2</p> <p>New Culvert: Class Environmental Assessment (EA) Schedule A+</p> <p>Conveyance if expanded right-of-way required: Class EA Schedule B</p>	<ol style="list-style-type: none"> 1. Urban modelling for Alliston in Phase 2 of the DMP including all urban and overland drainage system features to confirm this recommendation (confirm required culvert dimensions in accordance with appropriate design standards, review the need for additional storm sewers, ditches, etc. to mitigate increased flows downstream). 2. Initiate consultation with landowner(s) downstream of proposed culvert to discuss recommendations. 3. Coordinate future needs with Sir Frederick Banting Road traffic study.
<p>Industrial Parkway and Candian Pacific Railway (CPR) (Section 9.1.2)</p>	<p>Low</p>	<p>Originates as spill from Upper Nottawasaga River toward Spring Creek along existing overland flow path.</p> <p>High flood risk limited to agricultural field; no impacts to buildings.</p> <p>Future works in this area could include a new culvert under the CPR.</p>	<p>Long term</p>	<p>Requires coordination with CPR.</p> <p>New Culvert: Class EA Schedule A+</p>	<ol style="list-style-type: none"> 1. Initiate coordination with Nottawasaga Valley Conservation Authority (NVCA) in subsequent stages of the DMP to review spill toward Industrial Parkway from Upper Nottawasaga River. 2. Initiate discussion with CPR to confirm if there are existing culverts under CPR between Industrial Parkway and 13th Line. 3. Review need for ditching and/or grading. 4. Initiate EA study to review culvert upgrade opportunities.

Flood Risk Area and Description	Priority Ranking	Comments	Potential Implementation Timeline	Potential EA Requirements ⁽¹⁾	Next Steps and Potential Remedial Measures
7 th Line at CPR and South Simcoe Railway (SSR) (Section 9.1.7)	Low	The tributary to Beeton Creek west of SSR is lacking an adequate outlet to the main branch as there was no culvert crossing found under SSR.	Long term	Requires coordination with SSR and CPR New Culvert(s): Class EA Schedule A+	1. Initiate discussion with SSR to confirm the presence of an existing culvert. 2. Initiate discussion with SSR and CPR to discuss the need for drainage improvements in this area.
Culvert Capacity - Replacement Culverts					
5 th Line at 20 th Sideroad (Section 9.1.4)	High	Area of known flooding concern with repeated flood reports from residents. Modelled results confirm findings of previous studies that suggest existing culvert at 20 th Sideroad meets its design capacity. Future works in this area could include a new or upsized culvert.	Short term	Flow review: DMP Phase 2 New Culvert: Class EA Schedule A+	1. Confirm peak flows in Penville Municipal Drain using updated intensity-duration-frequency (IDF) parameters and topography, confirm existing culvert capacity and assess the need for additional culvert conveyance and ditching/grading. 2. Initiate consultation with local landowner(s) to discuss recommendations.
10 th Line at 10 th Sideroad (Section 9.1.5)	High	The existing culvert under 10 th Sideroad north of 10 th Line is adequately sized for the flows in Bailey Creek. However, spill from Beeton Creek occurs during the Regional event which conveys additional flow to 10 th Sideroad culvert, which was likely not considered during the design of the structure. Future works in this area could include a new or upsized culvert(s) or bridge(s).	Short term: Review of flows and bridge sizes Long term: increase capacity	Assessment: DMP Phase 2 Crossing Upsizing: Class EA Schedule B	Review/confirm existing culvert capacity (Structure ID 12032) with consideration for spill from Beeton Creek and assess the need for additional culvert conveyance and ditching/grading.
14 th Line at 20 th Sideroad (Section 9.1.3)	Medium	High flood risk only impacts one residential building; however, safe access and egress is limited during Regional event. Future works in this area could include a new or upsized culvert(s).	Short term	New Culvert(s): Class EA Schedule A+	Initiate EA study to review/confirm existing culvert capacity (MSI_132 and MSI_133), assess the need for additional culvert conveyance and ditching/grading, and review benefits of regrading Trans Canada Trail to lower blockage to flow.

Flood Risk Area and Description	Priority Ranking	Comments	Potential Implementation Timeline	Potential EA Requirements ⁽¹⁾	Next Steps and Potential Remedial Measures
Tottenham Road at 10 th Line (Section 9.1.6)	Low	The model results are inconclusive as to the origin of extensive flooding in this location. The CPR culvert may contribute to increased flood elevations upstream of the railway; however, the culvert is adequately sized to convey the 20-year design event. This location is within the downstream portion of Bailey Creek Swamp and therefore low-lying elevations may contribute to elevated flood risk. Larger crossing capacity may reduce flooding east of CPR.	Long term	Requires coordination with CPR. New Culvert(s): Class EA Schedule A+	There may be little that can be done to reduce flooding in this area due to the Baily Creek Swamp. The culverts at Tottenham Road (MSI_137) and CPR (MSI_120) appear to compound this flooding somewhat.
Channel Capacity					
Honda Manufacturing Plant (Section 9.2.2)	High	A new industrial subdivision is proposed south of Industrial Parkway which will divert flows via a storm sewer to Nottawasaga River. This diversion may provide flood relief to the Honda plant that is not reflected in the two-dimensional (2D) HEC-RAS model. An urban drainage system model including both municipal and private sewer systems would provide a more detailed understanding of flooding in this area.	To be confirmed	Assessment: DMP Phase 2	Urban modelling for Alliston in Phase 2 of the DMP including all urban and overland drainage features in this area including municipal and private sewer systems to confirm extents of flooding and review potential mitigation measures.
10 th Line East of Tottenham Road (Section 9.2.4)	High	The roadbed in this area is at an elevated risk of erosion from riverine sources. Future works in this area could include erosion protection works for the road.	Short term	Roadway armouring: Class EA Schedule A	Erosion protection works for the roadway should be considered for this location.
6 th Line at Tottenham Road (Section 9.2.5)	High	The roadbed in this area is at an elevated risk of erosion from riverine sources. Future works in this area could include erosion protection works for the road.	Short term	Roadway armouring: Class EA Schedule A	Erosion protection works for the roadway should be considered for this location.

Flood Risk Area and Description	Priority Ranking	Comments	Potential Implementation Timeline	Potential EA Requirements ⁽¹⁾	Next Steps and Potential Remedial Measures
Alliston Wastewater Treatment Plant (Section 9.2.1)	Low	Flood mitigation at the Alliston Wastewater Treatment Plant (WWTP) is likely not required due to its location in the floodplain. The Sir Frederick Banting Road bridge is not the primary factor contributing to flooding, as flood elevations are high downstream of the bridge as well. Recommendations here include planning and policy measures.	Short term	n/a	Ensure preventative measures are in place to protect against potential flooding (emergency response/evacuation plans, floodproofing essential equipment rooms, emergency backup power, etc.)
9 th Line at 15 th Sideroad (Section 9.2.3)	Low	Overland flooding in this area is primarily a result of channel capacity along Baker Drain, both north and south of 9 th Line. Future works in this area could include channel modifications and potentially culvert improvements, or the addition of a new culvert at the low point.	Short term	New Culvert(s): Class EA Schedule A+ Channel works requiring land acquisition: Class EA Schedule B	Future studies could review the flow through this area and provide recommendations on appropriate channel and culvert dimensions.
Schomberg River north of Highway 9 (Section 9.2.6)	Low	Existing residences at the top of valley slope are immediately adjacent to the floodplain.	n/a	n/a	No immediate works or studies are recommended here. This location is included for awareness purposes.
Previously Identified Flood Risk Areas					
Alliston at Spring Creek (Section 9.3.1)	High	NVCA has identified this residential area of Alliston as a flood vulnerable area. However, the 2D HEC-RAS model results of this study do not match those of NVCA's. Storm sewer system components present in this area may provide flood relief that is not reflected in the HEC-RAS model. An urban drainage system model would provide a more detailed understanding of flooding in this area.	To be confirmed	Assessment: DMP Phase 2	1. Review Spring Creek peak flows during subsequent stages of the DMP to confirm differences between NVCA and Matrix modelling results. Ensure urban drainage system modelling that will be completed for Alliston in Phase 2 of the DMP includes all drainage features in this area to confirm extents of flooding and review potential mitigation measures.

Flood Risk Area and Description	Priority Ranking	Comments	Potential Implementation Timeline	Potential EA Requirements ⁽¹⁾	Next Steps and Potential Remedial Measures
Beeton at Hendrie Drain (Section 9.3.2)	High	Hendrie Drain is a known source of flooding through the community of Beeton. New subdivisions with storm sewer systems were developed in Beeton in recent years which are not included in the 2D HEC-RAS model. These features may provide flood relief in this area. An urban drainage system model (PCSWMM) would provide a more detailed understanding of flooding in Beeton.	To be confirmed	Assessment: DMP Phase 2	Ensure urban drainage system modelling that will be completed for Beeton in Phase 2 of the DMP includes all drainage features in this area to confirm extents of flooding and review potential mitigation measures.
Beeton Flats (Section 9.3.3)	Low	Beeton Flats is a large, natural feature remnant of a glacial lake. Due to its low-lying nature, elevated flood risk is observed during all modelled events due to depths of ponding. There is no feasible remedy for flood risk in Beeton Flats due to the size and depth of flooding.	n/a	n/a	<ol style="list-style-type: none"> Educate local residents and emergency responders about the potential for inaccessible roads during high flow events. Limit development in Beeton Flats to prevent population increase in identified high risk flood zone.
Bailey Creek Swamp (Section 9.3.4)	Low	Bailey Creek Swamp is an area of known flooding between two tributaries and is not directly caused by road grading or other anthropogenic means. NVCA has indicated that spill from Nottawasaga River into Bailey Creek upstream of the study area boundary may contribute to widespread flooding in this location. There is no feasible remedy for flood risk in Bailey Creek Swamp due to the size and natural characteristics of this area.	n/a	n/a	<ol style="list-style-type: none"> Initiate coordination with NVCA to review occurrence of spill into Bailey Creek watershed from upstream. Educate local residents and emergency responders about the potential for inaccessible roads during high flow events. Limit development in Bailey Creek Swamp to prevent population increase in identified high risk flood zone.

(1) The DMP can include assessments up to a Schedule B EA and therefore is it assumed that these will be completed during Phase 2 of the DMP.

In addition to the mitigation planning presented in Table 29, the following locations were identified as needing review of ditch capacity, maintenance, etc. Note that these do not require full studies but should be completed in conjunction with further detailed hydraulic and flood mitigation analyses at the identified flood risk locations.

Table 29 Recommendations for Ditching

Location	Recommendation
CPR corridor and Industrial Parkway	Review ditch maintenance needs on west side of CPR between 13 th Line and Industrial Parkway as well as the south side of Industrial Parkway west of CPR in conjunction with potential new culvert on CPR.
10 th Sideroad at 10 th Line	Review maintenance needs for west ditch on 10 th Sideroad north and south of 10 th Line in conjunction with Structure 12032 improvements.
Tottenham Road between CPR and 10 th Line	Review maintenance needs for west ditch on Tottenham Road in conjunction with MSI_120.
9 th Line west of 15 th Sideroad	Review maintenance needs for north and south ditches in conjunction with Baker Drain East Branch detailed assessment.

CPR - Canadian Pacific Railway

11 Next Steps

The purpose of Phase 1 of the DMP was to gain a comprehensive overview of flood risk in the Town’s jurisdiction and identify next steps for flood mitigation study requirements. Assessment of the mitigation measures identified as an EA Schedule A or B in Table 28 is recommended to be included in future phases of the DMP. This section details the following assumed components for the remainder of the DMP:

- hydrology update (to be coordinated with NVCA)
- two-dimensional HEC-RAS model refinement
- urban drainage system modelling of three settlement areas
- detailed flood mitigation assessment (i.e., Schedule A and B EA studies of locations listed in Table 28)
- climate change consideration

Based on the details and assumptions below, the technical portion of the next phase(s) of the DMP are estimated to cost \$690,000 (not including project management, meetings, final report and other miscellaneous tasks).

11.1 Hydrology Update

The peak flows provided by NVCA and used in the hydraulic model are over 30 years old. There has not been significant development in the headwaters of the Upper Nottawasaga River and Innisfil Creek subwatersheds and therefore this data was considered suitable for use in this study. However, developing a more current understanding of peak flows within the Town's jurisdiction would be beneficial to ensure the Town has the most up-to-date and detailed understanding of flood risk in its jurisdiction. This likely requires a full hydrologic study and should be coordinated with NVCA as part of future phases of the DMP. To update peak flows, the following tasks would be required:

- review/update rainfall input (IDF parameters)
- delineate riverine catchment areas using LiDAR topography
- parameterize catchments using current land use, soils, and topographic data
- calibrate and validate hydrologic model
- calculate peak flows for design storms and Regional storm
- review climate change considerations
- reporting

It is estimated that the cost to complete the above work is \$90,000.

11.2 Two-dimensional HEC-RAS Model Updates

Given the size of the study area, the 2D HEC-RAS model prepared for Phase 1 of the DMP was developed as a screening tool to identify existing areas at risk to riverine flooding and those that require further flood mitigation studies. Some simplifications were required in the mesh development to ensure the model is robust but not too detailed for such a large study area (i.e., using the channel centreline for breaklines on small watercourses, simplifying and merging building footprints, etc.)

Matrix understands that the Town and/or NVCA may use the 2D HEC-RAS model in future riverine studies, such as to review spills and/or flood risk areas identified in Section 9 in further detail. This is different than the recommendations for urban drainage system modelling of the three settlement areas as urban infrastructure modelling requires different software (i.e., PCSWMM). If using the 2D HEC-RAS model in the future, Matrix recommends that the model be updated to use the new hydrologic data

(discussed in Section 11.1), the existing condition 2D HEC-RAS model should be rerun to confirm modelled flood extents, risk, and flood damage assessment calculations. With updated flows, the results are likely to be different than those available and presented in this report. The updated existing condition results should then be used to confirm flood risk areas prior to initiating EA studies.

The estimated cost to complete this task (assuming revised peak flows are available from the previous task) is \$40,000.

11.3 Detailed Flood Mitigation Assessments

When studying the identified flood risk areas in further detail (see Table 28), the 2D HEC-RAS model should be trimmed to the desired area while considering appropriate boundary conditions for the trimmed model. Model trimming will enable more detailed review of local conditions while maintaining acceptable simulation times. Small channels previously defined by breaklines along centrelines should be incorporated using their banks. These 3D breaklines of the banks were prepared by PHB during the LiDAR classification process (PHB 2020). Use of breaklines along banks of all watercourses will require a finer mesh resolution in the channels and was therefore not feasible for the larger study area. Refine the mesh throughout the trimmed model domain to an appropriate resolution. Any culverts currently modelled using the cut or block approaches, as described in this report, should be updated with detailed information for site-specific studies. Finally, the model should be updated to use the new hydrologic data (discussed in Section 11.1) and the model rerun to confirm existing flood risk prior to assessing of potential mitigation measures.

Future condition assessments should then be completed by simulating the future condition in the trimmed models to analyze various flood mitigation alternatives (i.e., new culvert, culvert upgrades, channel conveyance improvements, etc.). It is understood that these assessments will follow a typical EA approach as part of the subsequent phase(s) of the DMP. As such, a Public Information Centre is also recommended to present the study findings to the public and stakeholders and seek feedback.

The estimated cost to assess the flood risk areas identified in Table 28 is \$390,000. If studying these areas individually, the total cost to complete the assessments could be significantly larger than this.

11.3.1 Tottenham Dam Failure - Hydraulic Review

In addition to the flood risk areas identified in Table 28, an assessment of the potential flood risk resulting from the Tottenham Dam failure should be completed. The Tottenham Dam flood control structure was assumed to remain in place for this study. To understand the potential flood risk to the community of Tottenham resulting from failure of the dam, the terrain in the 2D HEC-RAS model could be trimmed and adjusted to reflect removal of the dam. Simulations with and without the dam should then be completed for a range of flow events to determine the resulting flood risk. The estimated cost to complete this task is \$20,000.

11.4 Urban Drainage System Modelling

In addition to the above refinements, Matrix understands that the Town is interested in identifying existing urban drainage deficiencies and flood risk, particularly within their three settlement areas. The 2D HEC-RAS model is not appropriate for this purpose and as such, the modelled results presented in this report may not reflect the complete picture of flooding conditions in urban areas. Therefore, urban drainage system modelling completed in Phase 2 of the DMP should include use of different modelling software such as PCSWMM that is capable of incorporating storm sewers, SWM facilities, local overland drainage paths, and other urban drainage components. Separate models should be prepared for Alliston, Beeton, and Tottenham. Matrix recommends that the Town prepare a comprehensive database of existing storm sewers and SWM features in the three settlement areas to be used as a starting point for model development.

Matrix previously reviewed data requirements to support development of a PCSWMM model. The Town provided Matrix with their existing storm sewer datasets and our review is summarized as follows:

- Sewer network shapefile provided for all municipal sewers including pipe locations, dimensions, materials, and inverts:
 - ✦ Upstream and downstream inverts are missing for approximately 70% of sewers in the current shapefile. Matrix recommends that the Town review as-built drawings and/or conduct topographic survey as required to populate sewer database with invert data prior to initiating Phase 2.
 - ✦ Dimensions are unavailable for 5% of sewers in the shapefile. Matrix recommends that the Town review as-built drawings and/or conduct survey as required to populate sewer database with dimensions.

- ✦ Pipe material are unavailable for 27% of sewers in the shapefile. Matrix recommends that the Town review as-built drawings to populate sewer database with pipe materials.
- Catch basin locations are available from the Town for the municipal drainage system to which urban drainage catchments can be delineated for optimal flow input to the PCSWMM model.
- Existing SWM pond locations are available and supporting SWM reports were provided. Matrix recommends that the Town compile the SWM pond stage-storage-outflow curves for use in the PCSWMM model.
- Private storm sewer and/or SWM systems that are not owned and operated by the Town were not provided. To enhance understanding of existing flooding conditions, particularly in large industrial areas such as the Honda manufacturing plant, Matrix recommends that the Town obtain design and/or as-built drawings of large industrial properties to review the presence of urban drainage system components.

It is estimated that the cost to prepare an urban drainage system model for each of the three settlement areas (i.e., three models; one for each settlement area) and conduct a comprehensive hydraulic flood risk assessment and characterization is \$150,000.

11.5 Climate Change Consideration

In theory, the climate change assessment results presented in Section 7.1 indicate that the 100-year inundation boundary could become the Regulatory limit in portions of the study area if the current climate change predictions occur as previously outlined. Prior to creating a new policy around this prediction, Matrix recommends that NVCA complete a comprehensive review and update of the hydrology for the Nottawasaga River watershed.

As part of confirming existing flows within the watershed and the Town, Matrix recommends that the existing IDF be updated based on the latest data, and not rely on outdated curves. Research results indicate that the use of outdated IDF curves could result in critical under-design or expensive over-design of key water infrastructure (Burn and Taleghani 2013; Douglas and Fairbank 2011, Madsen et al. 2009). Efforts to deal with the stationary climate assumption were undertaken by MTO and the Ontario Ministry of Environment, Climate Change and Parks through a multi-phase project to create interpolated IDF curves for Ontario using Regional approaches and modelling (). The use of this tool, publicly available through the MTO's website, provides a ready-to-use, and regularly updated and reliable source for IDF data for the Town (MTO

2016). Larger municipalities in Canada are working to update IDF data for their design guidelines. Examples include City of Hamilton (Cole Engineering 2014), City of London (Prodanovic and Simonovic 2017), and City of Calgary (WaterSMART 2017). However, there is recognition from these municipalities that climate change is a moving target and IDF data will continue to change. As a result, Matrix is currently undertaking another review of IDF data for City of Hamilton. In recognition of the required ongoing updates, we recommend that the Town use the MTO's IDF tool as a reference instead of duplicating this work by developing IDF curves for Town use.

A cost estimate for this task has not been provided as it is assumed it would be incorporated into the flow update exercise as described in Section 11.1.

11.6 Coordination with Other Studies

The Town's Road Needs Study identified a number of potential drainage issues throughout the Town's jurisdiction including the need for ditching, shoulder cuts, and/or brushing to improve drainage (R.J. Burnside 2019). Matrix recommends that prior to initiating any of these projects, the Town should reference the flood risk areas identified in this study to determine if improvements to the drainage system could be coordinated with the roads project. One of the primary uses of the model developed for this study is as a screening tool to provide an overview of flood risk the Town's jurisdiction. While it may need site-specific reviews and revisions, this screening tool can be used to identify areas of elevated risk that should be considered in coordination with other projects as appropriate.

12 Conclusion

Matrix completed Phase 1 of a multi-phase DMP for the Town in response to increased concern over the potential flooding damage to residential areas, municipal infrastructure, and agricultural resources. The intention of the DMP is to plan and implement future drainage improvements in a comprehensive fashion with Phase 1 providing the Town's flood characterization and the detailed hydraulic assessment tailored to the Town and designed to encompass a large area. This report acts as an indicator for areas of flood concern in terms of severity, frequency, and associated damage, as well as a mitigation plan and prioritization for subsequent projects within the Town's jurisdiction.

To accomplish the flood characterization, Matrix developed a hydraulic model using a full 2D approach in HEC-RAS. The model input parameters were derived from a background review of all applicable SWM reports and available data as well as a Matrix

conducted field study and survey of the Town's bridges and culverts. A refined mesh specific to the nuances of the Town's configuration (i.e., buildings, land use, conveyance structures, etc.) was created for the model under quasi-steady state runs of the design storm to consider the previously defined hydrology of the site. The model is limited to inherent assumptions associated with a large-scale hydrologic and hydraulic model as well as its representation of solely riverine sources and flow. It is anticipated that Phase 2 of the DMP will incorporate urban infrastructure model. In addition, Matrix developed the cost relationship of flood damage in the form of a flood damage database for future project and policy guidance. Finally, a sensitivity analysis was performed to account for the uncertainty of climate change as well as some anticipated future construction.

The results of the model demonstrated widespread flooding throughout the Town. Matrix identified specific flood risk areas, their individual causes (pre-existing, or culvert or channel capacity issues), and proposed next steps for mitigation and/or further investigation. The Town should focus future flood mitigation on these key areas and undertake detailed studies complete with defined boundaries and a coarser mesh for the priority sites in coordination with the appropriate agencies. The urban areas were evaluated on a high-level basis in this report and should be evaluated in more detail with the DMP Phase 2 to include the functionality of the sewer network.

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