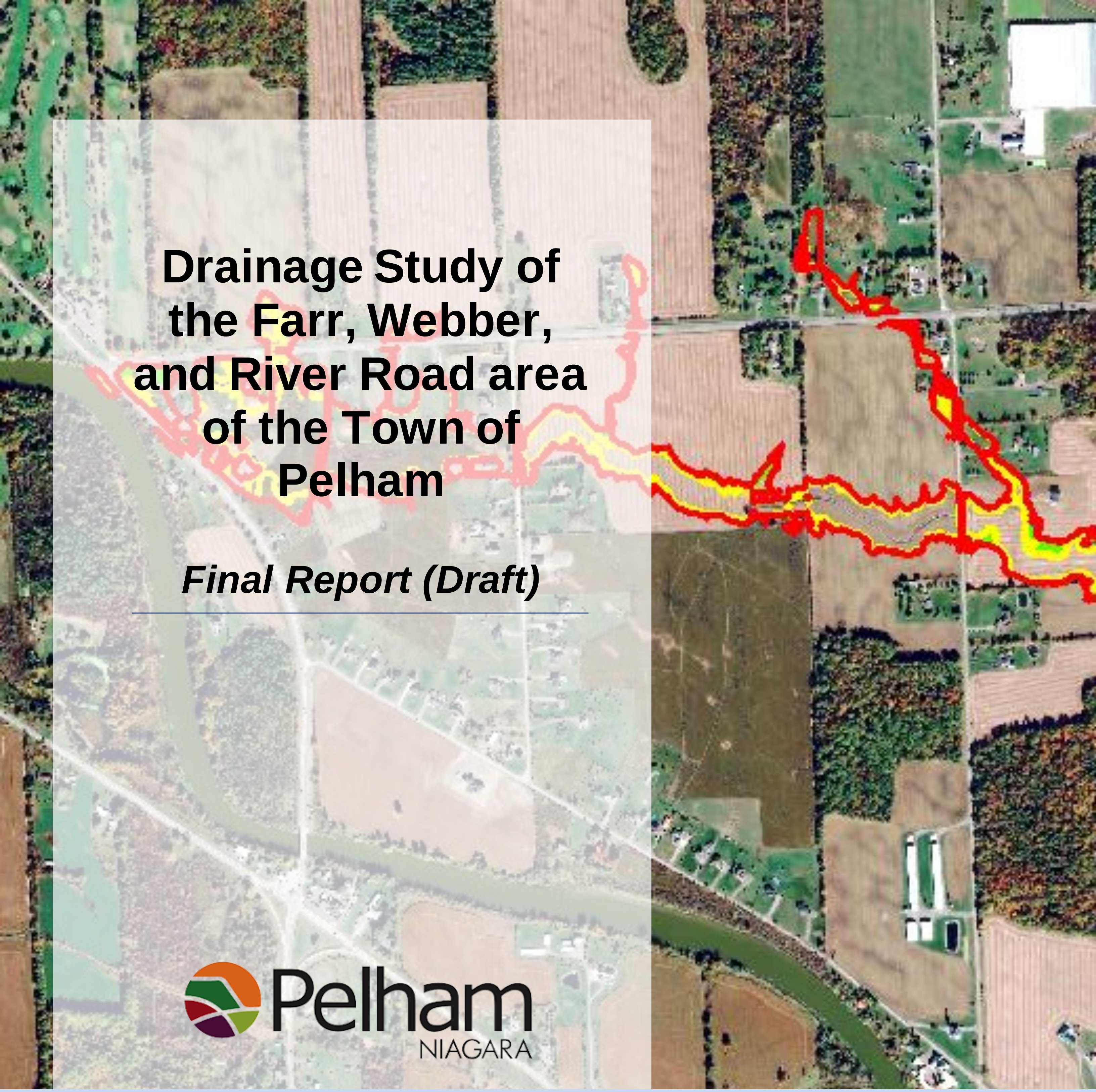

Drainage Study of the Farr, Webber, and River Road area of the Town of Pelham

Final Report (Draft)

February, 2023

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AHYDTECH GEOMORPHIC
ADVANCED HYDROLOGY HYDRAULIC GEOMORPHOLOGY

TABLE OF CONTENTS

APPENDICES

LIST OF FIGURES

LIST OF TABLES

ABBREVIATIONS AND ACRONYMS

CN	Curve Number
CSP	Corrugated Steel Pipe
DEM	Digital Elevation Model
DTM	Digital Terrain Model
GSC	Geodetic Survey of Canada
HEC	Hydrologic Engineering Center
LiDAR	Light Detection and Ranging
LULC	Land Use Land Cover
MNR	Ministry of Natural Resources
NAD	North American Datum
NPCA	Niagara Peninsula Conservation Authority
NRCS	Natural Resources Conservation Service
NSDB	National Soil Database
OHN	Ontario Hydro Network
RS	River Station
RTK	Real Time Kinematic
SSC	Soil Survey Complex
UTM	Universal Transverse Mercator

1 INTRODUCTION

AHYDTECH Geomorphic Ltd. has been retained by the Town of Pelham to provide consultancy services for a drainage study of the Farr, Webber, and River Road area. The existing condition of the drainage system of the area is in an unclear state which needs to be identified and resolved. The Farr, Webber and River Road area of the Town of Pelham is within the jurisdiction of the Niagara Peninsula Conservation Authority (NPCA).

Figure 1-1: Study Area near the Farr, Webber and River Road

The scope of the study includes developing a hydrologic model for the watershed and development of a 1D hydraulic model to determine the extent of flooding. AHYDTECH is undertaking a series of tasks to identify the drainage patterns, assess extent of flooding, and potential solutions to eradicate the existing drainage issues of the study area.

2 BACKGROUND

The Town of Pelham is within the Regional Municipality of Niagara comprising of communities namely: Fonthill, Fenwick, Ridgeville, Effingham, and North Pelham. The Farr, Webber and River Road areas of the Town of Pelham is located in Fenwick. Flooding issues has apparently been a concern here for a long time. A study conducted in 2017 by GM Blueplan Engineering, illustrated flooding issues and concerns in the urban areas of Fenwick along with other locations. Unplanned rural residential development, lack of a stormwater management plan connected with rural residential usage and the lack of a municipal drain are the causes of damage and degradation of the land is facing. Overland flows are directed to a variety of natural and manmade drainage channels that discharge into the Welland River, where only a few of the natural channels are regulated. The natural flow of the creek has also been altered due to unplanned development of the residential houses. The Town has undertaken the drainage study in order to resolve the drainage concerns and pertinent issues. The Farr, Webber and River Road area of the Town of Pelham encompasses an area of approximately 1.27 sq.km **(Figure 2-1)**. The drainage study area features from the west of Victoria Avenue and east of Church Street, corresponding to an area bounded by the south of Webber Road, the east and west side of Farr Street, north of the River Road and east of Victoria Road.

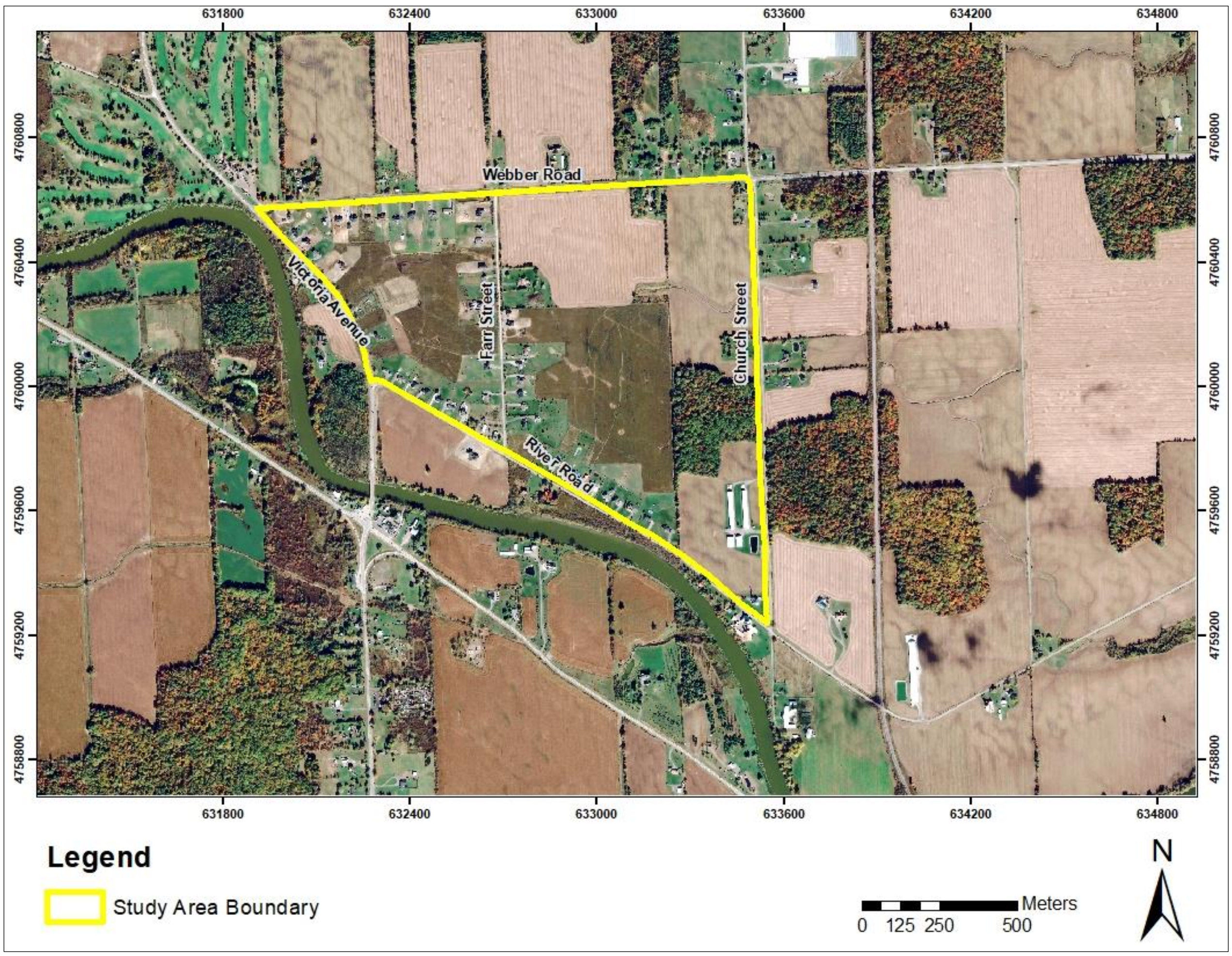


Figure 2-1: Extent of the Study Area

In order to identify the drainage patterns, assess extent of flooding, and potential solutions to eradicate the existing drainage issues of the study area, AHYDTECH has undertaken the following steps:

1. Collected and acquired necessary data from open sources and the Town for the development of hydrologic and hydraulic models;
2. Reviewed reports and documents regarding studies carried out previously close to study area;

3. Conducted field visits to collect topographic data and assess the existing condition of the study area ;
4. Developed a hydrologic model through SWMHYMO to obtain the variation of peak flows corresponding to different return period conditions
5. Developed a hydraulic model using HEC-RAS 1D to simulate flooding extent in response to flows of different return period events obtained from the hydrologic model;
6. Identified properties under flood risk and recommended mitigation options;
7. Prepared reports detailing methodology and results obtained from the study;
8. Carried out consultations with the Town, NPCA and pertinent stakeholders regarding the progress of the study.

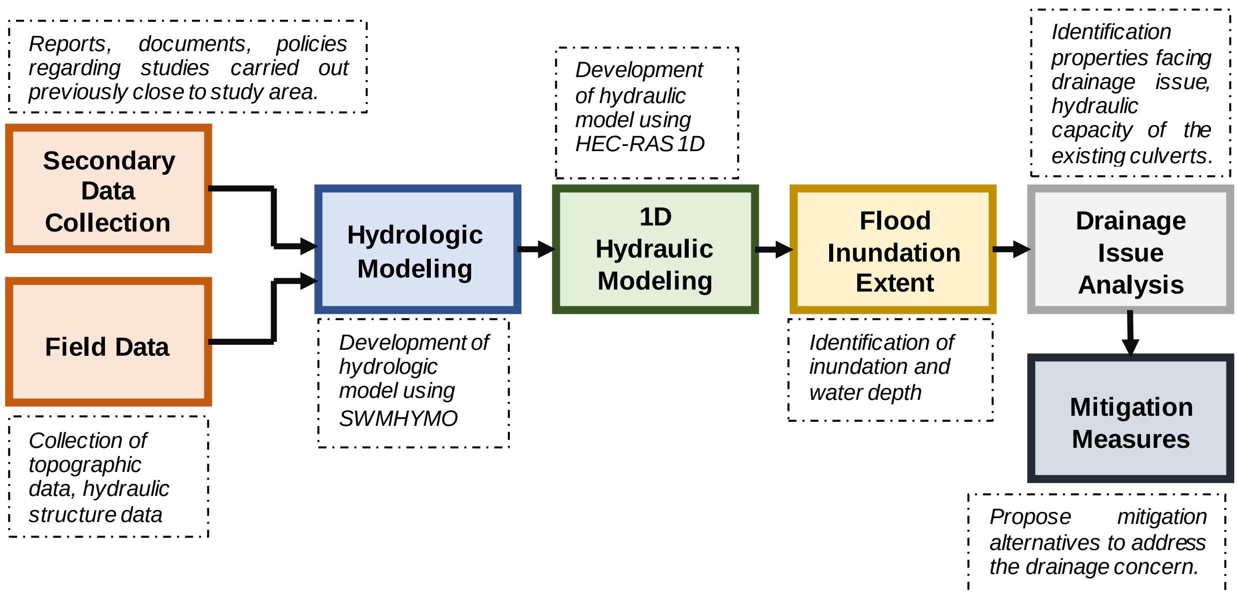


Figure 2-2: Flowchart showing the steps followed to complete the objectives of the study

2.1 COLLECTION & REVIEW OF EXISTING DATA

AHYDTECH project team had a project start-up meeting with the Town of Pelham staff virtually on 9th May 2022. AHYDTECH presented the workplan and the data required from the Town for the drainage study analysis. AHYDTECH reviewed and processed the following data and information:

- Rainfall data for storm events for specific return period events (2-, 5-, 25- and 100-year) obtained from Ministry of Transportation IDF Curve Lookup Table
- Land use, Land cover, Soil, and Digital Elevation Modelling (DEM) data
- Drainage, subwatershed and catchment boundaries
- Stream network data
- Policies documents for hydrologic and hydraulic model development
- Existing reports and information related to this study.

2.2 CHARACTERIZATION OF THE STUDY AREA

The study area encompassing the Farr, Webber and River Road area is largely rural, where trees and croplands are predominant. There are quite a few residential houses along the Farr, Webber, and River Roads, some of which are being subjected to the drainage issues currently. The land elevation of the watershed slopes downward from the west towards east, however there is an alteration in the direction of the stream at a certain location (details provided in **Section 3**). Since the study area is largely rural including some croplands, the soil characteristics of the study area is assumed to have moderate infiltration capacity. Welland River is flowing through the west and south of the study area. The topography of the area is nearly flat, and the elevation of the ground surface is comparatively low; hence, the area serves as an outlet area of the catchment and drainage issues are a common occurrence here.

3 DESKTOP ANALYSIS

Desktop analyses were performed to delineate the existing and historical drainage pattern within the study area. Whether or not the channels meet the definition of a watercourse, detailed study has been performed based on the present surveyed and historical available data. To estimate the flow pattern and possible changes along those channels, historical aerial images of certain years along a span of 20 years were analyzed.

3.1 NPCA DEFINITION OF A WATERCOURSE

A natural watercourse and surface water has distinct features where a natural watercourse is defined as a stream of water having a defined channel, with bed and banks where a surface water has no defined course. Under the *Conservation Authorities Act*, a watercourse is defined as “an identifiable depression in the ground in which a flow of water regularly or continuously occurs”. Watercourses transport water, sediment, and energy, provides habitat for aquatic ecosystems and drinking water for the communities. Thus, a watercourse is a complex, multi-functional, living system (*NPCA Policy Document, 2022).* According to the *Conservation Authorities Act*, a municipal drain is also a watercourse and are therefore regulated by Conservation Authorities.

To assess whether the streams within the study area falls under the definition of a watercourse, the criteria based on which natural and surface water is defined have been seen and it was observed that the streams within the study area does not have a defined channel or bank. Besides, the study area is not serviced by any municipal drain. However, analyzing map from NPCA Watershed Explorer and Contemporary Mapping of watercourses flowline direction 2012, it was observed that the area along the flow channel generated in HEC-GeoHMS **(Section 5.4)** for the study area is under NPCA regulated lands and hence, the channels within the study area were classified as different types of watercourses such as agricultural ditch, constructed agricultural ditch, natural ditch, roadside ditch etc. which have been further illustrated in **Figure 3-1**.

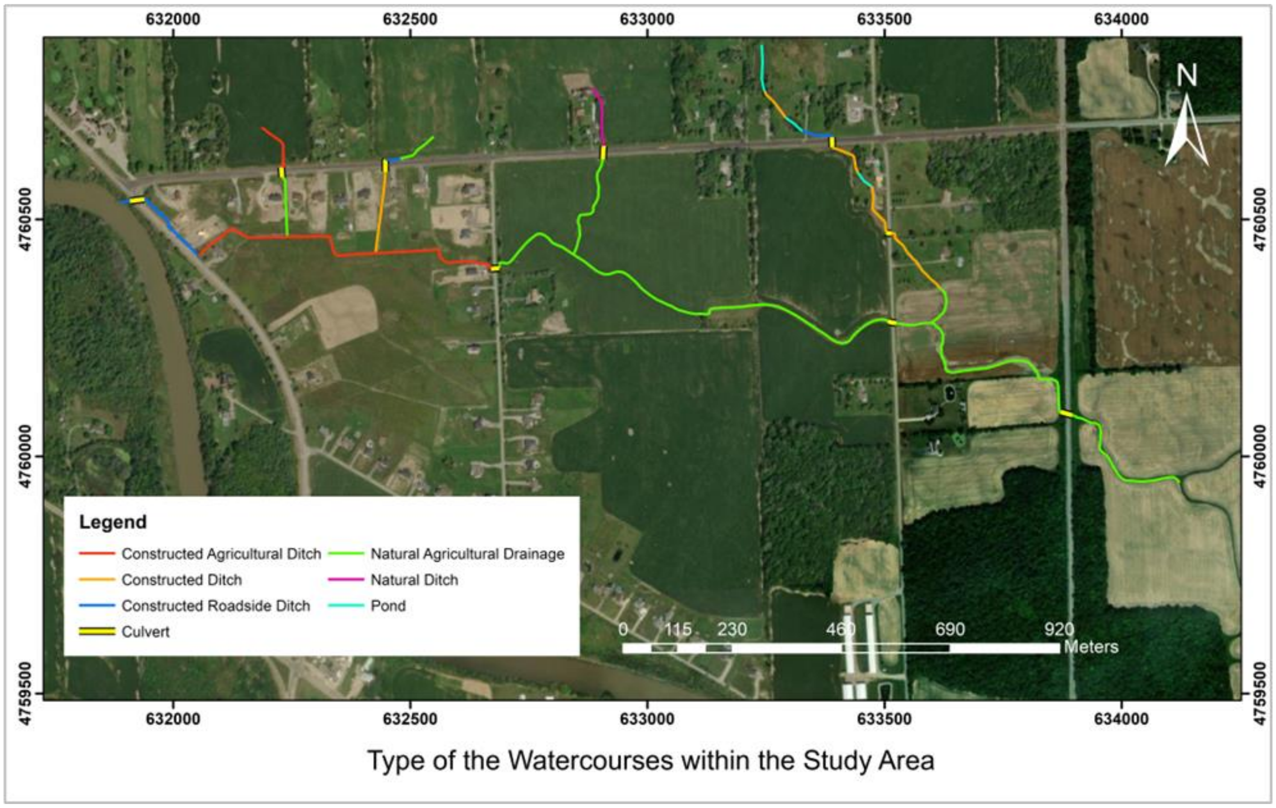


Figure 3-1: Types of Watercourses along the Study Area

3.2 HISTORICAL AERIAL IMAGERY ANALYSIS

Aerial photographs have been analysed from as early as the 2002 which provided a unique resource to describe changes within the study area for a period of 20 years. The following data sets were used for the purpose:

Table 3-1: Resolution and Source of Historical Air Photos

Date	Resolution (m)	Source
2002	0.12	AirPhotos_Niagara
2006	0.12	AirPhotos_Niagara
2010	0.12	AirPhotos_Niagara
2018	0.12	AirPhotos_Niagara
2020	0.12	AirPhotos_Niagara

Aerial Images Analysis of the Year 2002

The elevation from the upstream (west of Farr Street) gradually increases downstream (towards Farr Street), until at Farr Street, the elevation decreases again, and the flow diversion of the stream is seen near the Farr Street. Beyond the Farr Street, towards the east, the elevations gradually decrease. Thus, the elevations to the west are higher than the east side. The image **(Figure 3-2)** also shows the watercourse (marked in blue line) for the year 2002 and the changes that has occurred to the flow path during existing condition (marked in yellow line).

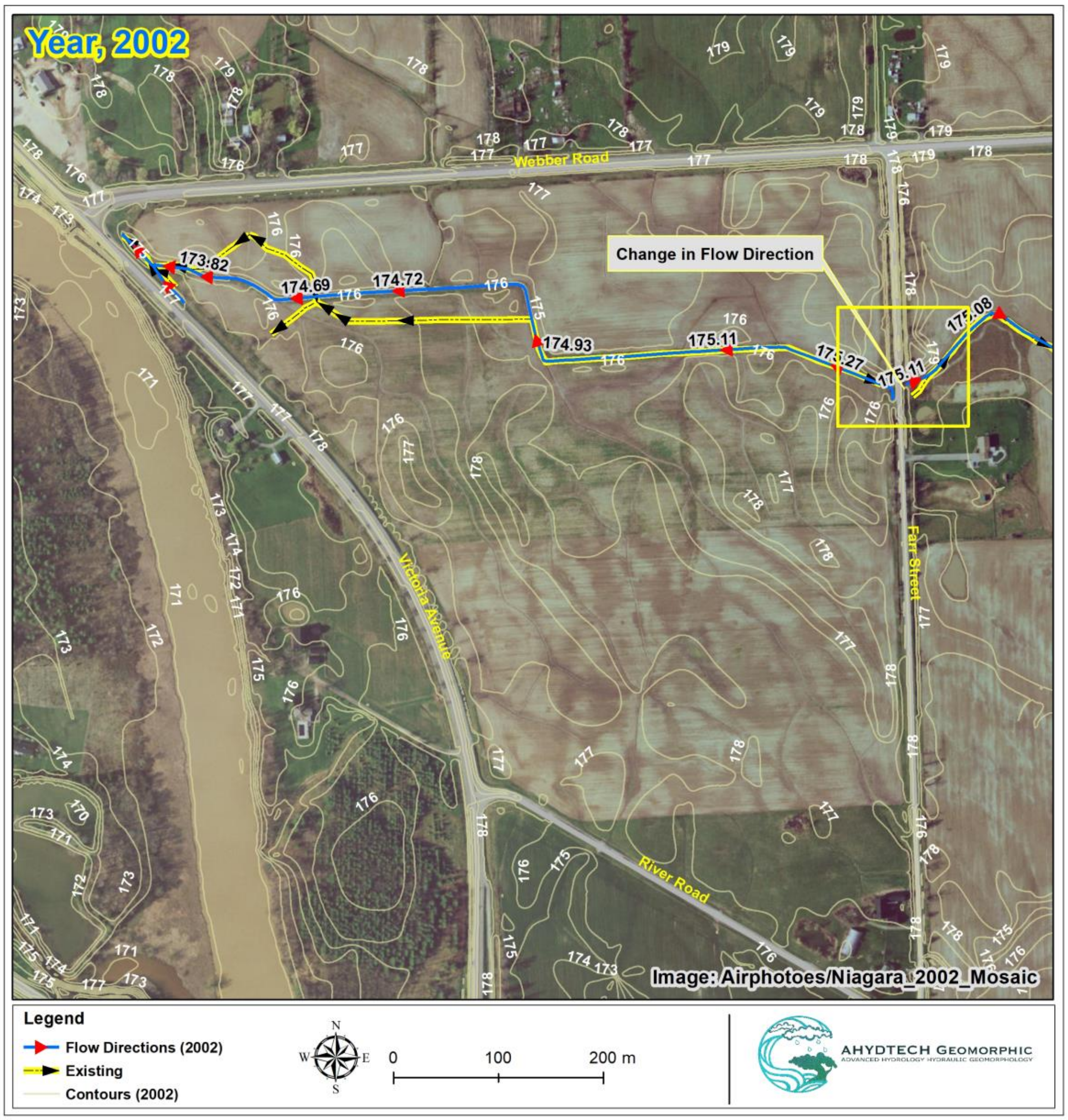


Figure 3-2: Aerial Image of 2002 of the Study Area

Aerial Images Analysis of the Year 2010

2010 shows nearly similar condition to 2002, except for some changes in elevation (compared to 2002) near the Victoria Avenue. The change in flow direction is still observed near the Farr Street. **(Figure 3-3)**.

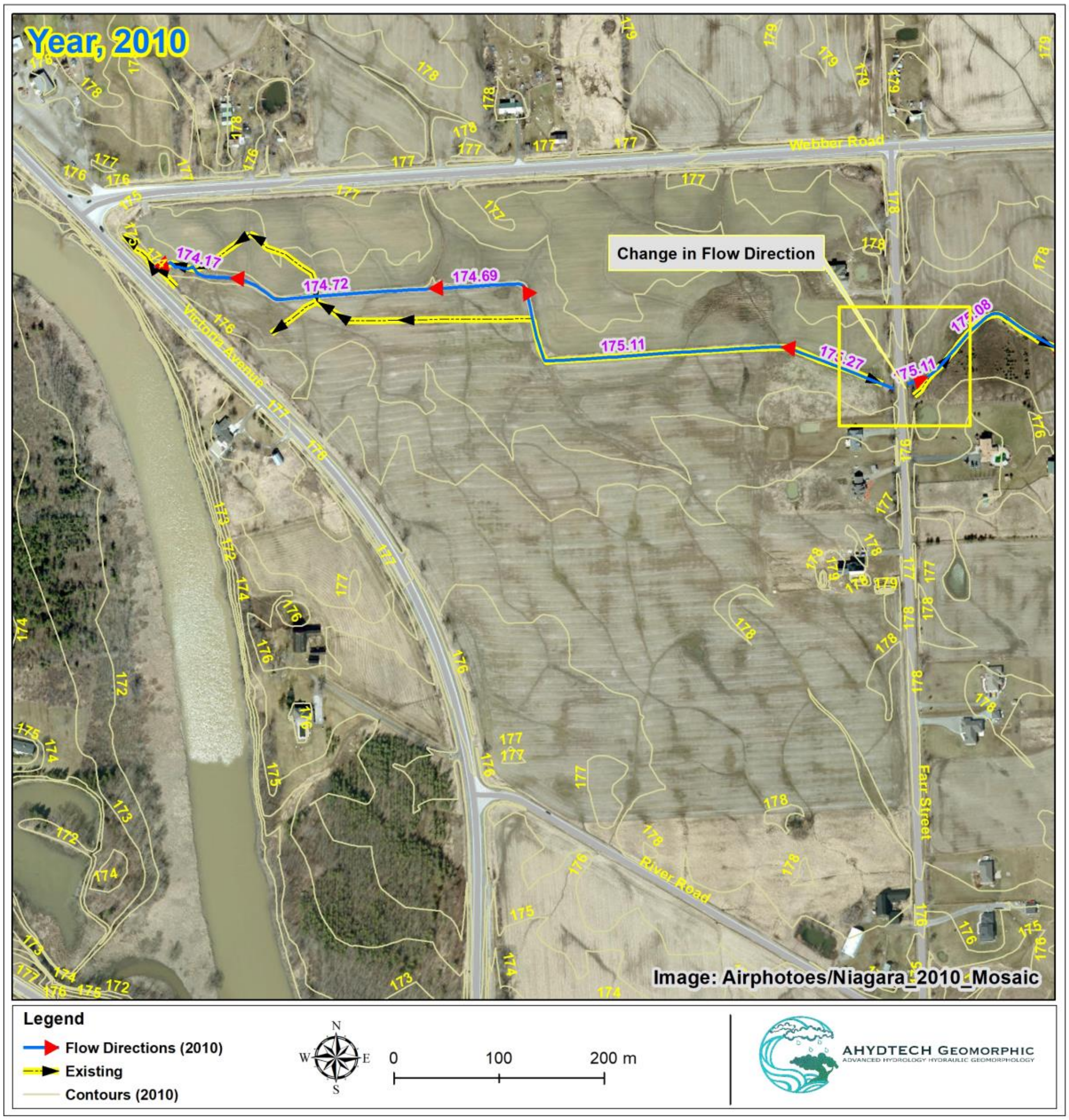


Figure 3-3: Aerial Image of 2010 of the Study Area

Aerial Images Analysis of the Year 2018

Based on the water course peak ground elevation, there is a change of flow direction in the study area. The change in flow direction is observed for 2018 aerial image where the flow change/diversion location shifted towards the west of Farr Road. There was a development of residential houses, which took place within the area compared to the 2010 aerial image. An additional channel culvert is seen between two properties at the Victoria Avenue properties (265 Victoria Avenue and 275 Victoria Avenue) which turns at an angle of 90 degree (approx.). Changes in elevations can also be noticed compared to 2010 **(Figure 3-4)**.

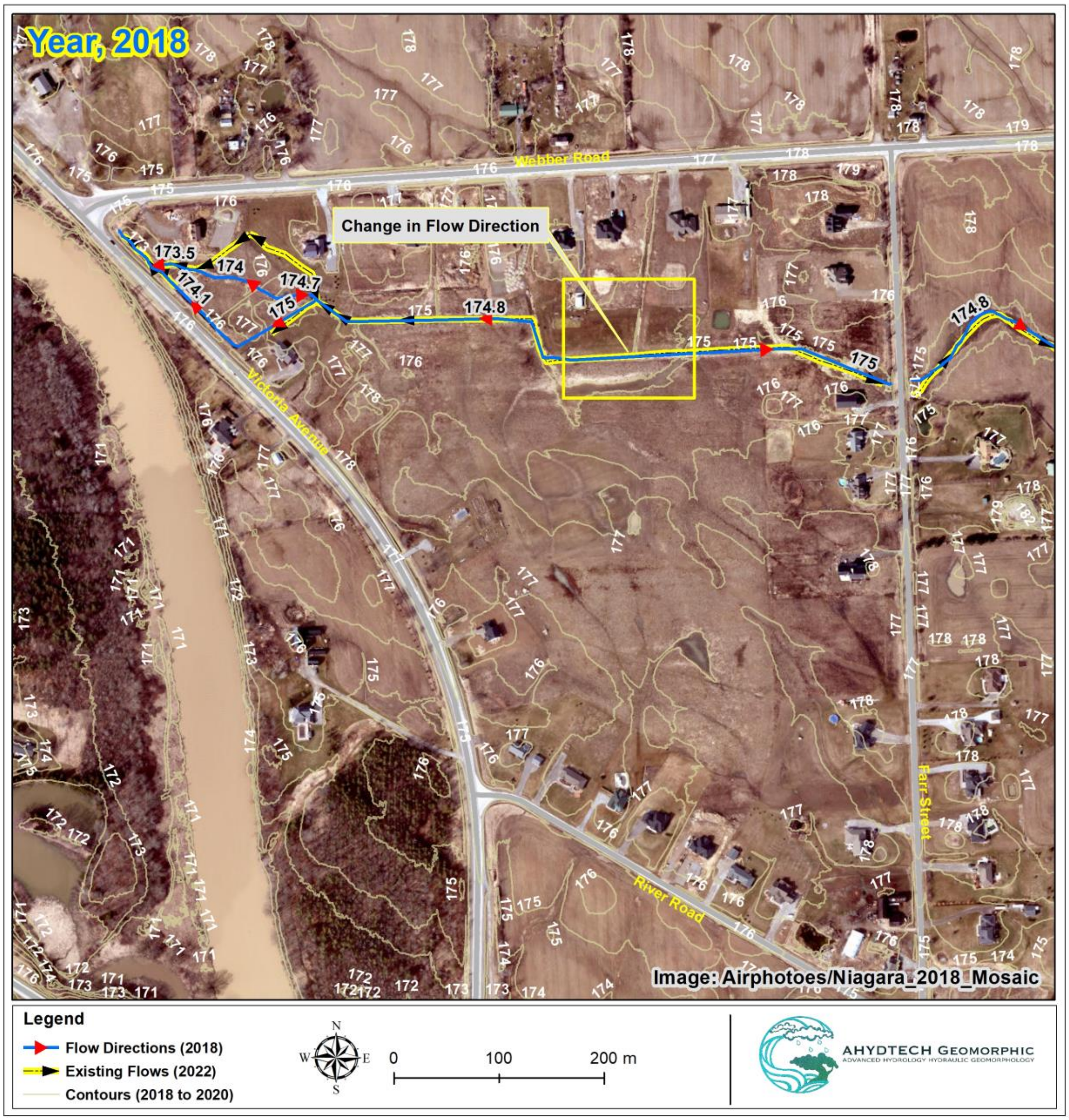


Figure 3-4: Aerial Image of 2018 of the Study Area

Aerial Images Analysis of the Year 2020

The study has done drainage analysis of 2020 aerial image. The flow change/diversion location observed in 2018 is also seen from the 2020 aerial image. A 0.6m diameter and 125m long culvert was installed in the west of Farr Street near the Victoria Avenue. The culvert runs 35m in between 265 Victoria Avenue and 275 Victoria Avenue properties, and then turns 90 degrees under the ditch along Victoria Avenue to join the west channel downstream ***(Figure 3-5)***

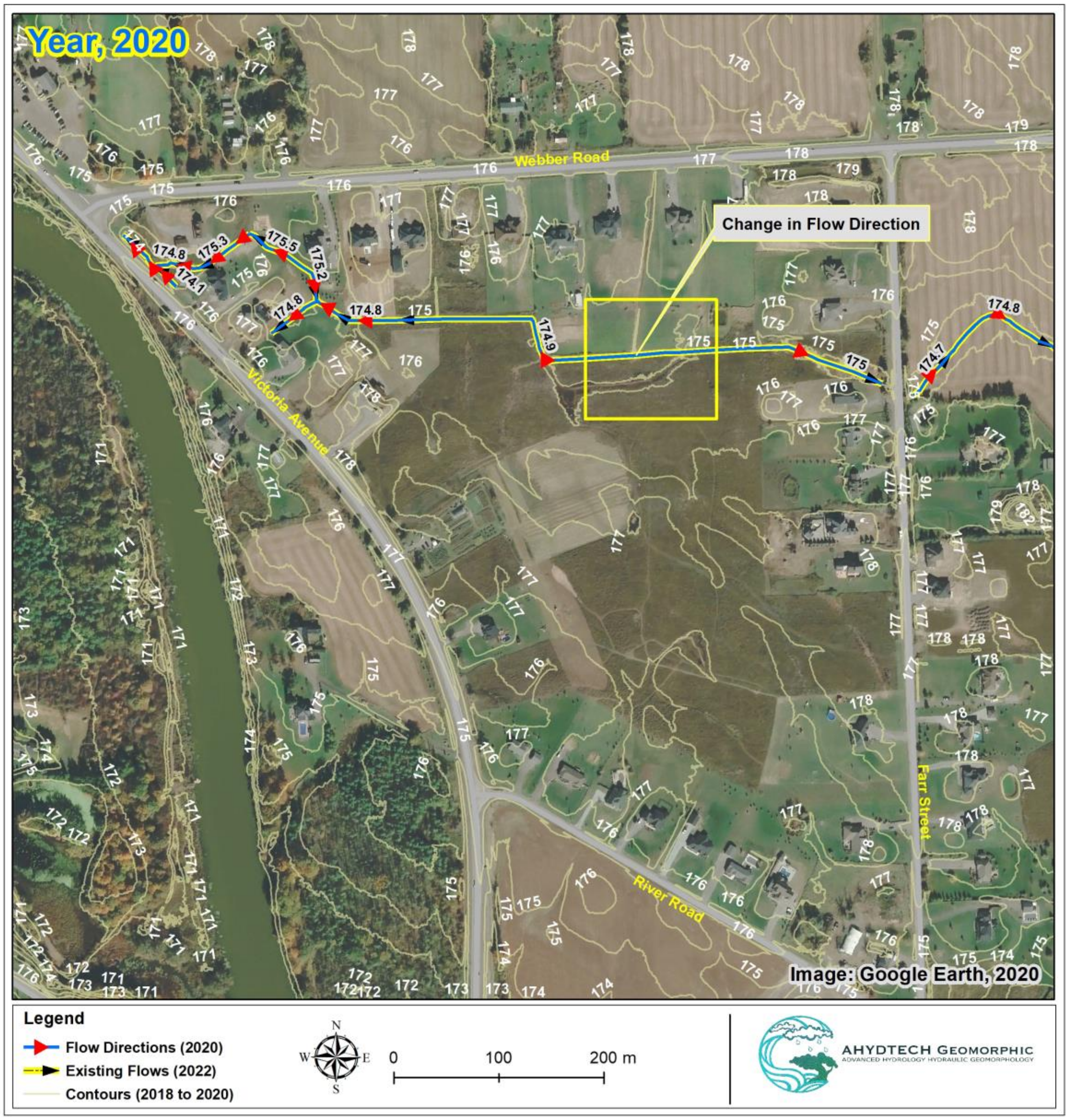


Figure 3-5: Aerial Image of 2020 of the Study Area

Existing Condition (2022 Topographic Survey, AHYDTECH)

As a part of the study, road crossing, water course and topographic field data were collected. It was observed from the field data and 2022 aerial image, the flow change/diversion location in the study area was further shifted towards the west as compared to the flow change/diversion location in the 2020 aerial image. **Figure 3-6** illustrates the existing condition of the study area and what changes have taken place in comparison to the 2020 condition.

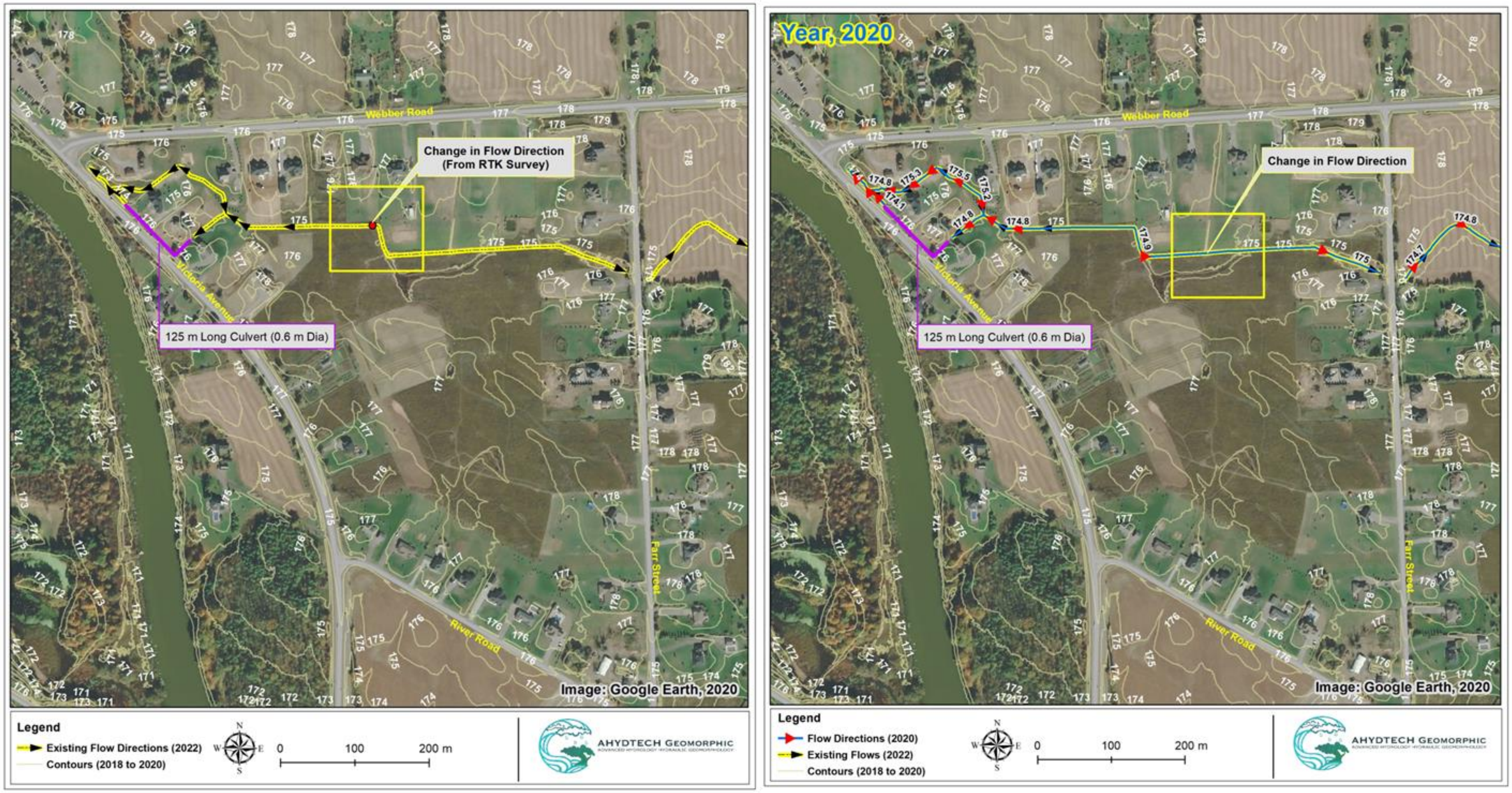


Figure 3-6: Existing Condition at the Study Area

4 DATA COLLECTION AND FIELD ASSESSMENT

4.1 TOPOGRAPHIC SURVEY AND CULVERT DATA COLLECTION

After completing the initial desktop analysis and review of the existing data, AHYDTECH implemented a field program. The field program included topographic surveying of the reaches in the watershed using a standard GNSS RTK/GPS survey techniques. The data was recorded in the geodetic control datum CGVD28, and all data files has been provided in the provincial UTM projections (NAD83). The RTK/GPS was used to acquire dimension of hydraulic structures such as that of bridges and culverts along the reaches were surveyed. The topographic survey includes about 2.9 kilometers starting from the Victoria Road up to the Church Street at the East. Cross-sections were taken using RTK along the main channel and at upstream and downstream of the Culverts. Total 15 culverts were surveyed, and relevant data was collected to perform hydraulic analysis. The culvert data were collected which includes a) Dimensions b) Invert Elevation c) Overt Elevation d) Type of Material e) Road Elevation etc. The topographic data locations and details of the hydraulic structures are provided in **Appendix B.**

4.1.1 CHANGES IN BED ELEVATION

The topography of the channel bed near the Farr Road had been observed to be nearly flat. Analyzing the RTK data collected during field survey, channel elevation profile was created. Furthermore, elevations of the crossings and road decks have been incorporated to visualize the exact location of flow diversion and crossings that directly affecting the channel causing flow obstruction. At approximately 350m east of the Victoria Road, the elevation of the channel bed seemed to be higher, considering which the flow change/diversion location was determined and later incorporated into the hydraulic model. **Figure 4-1** illustrate the longitudinal profile of the channel generated using the RTK data, the elevations of roads and crossings as well as the exact location of flow change/diversion.

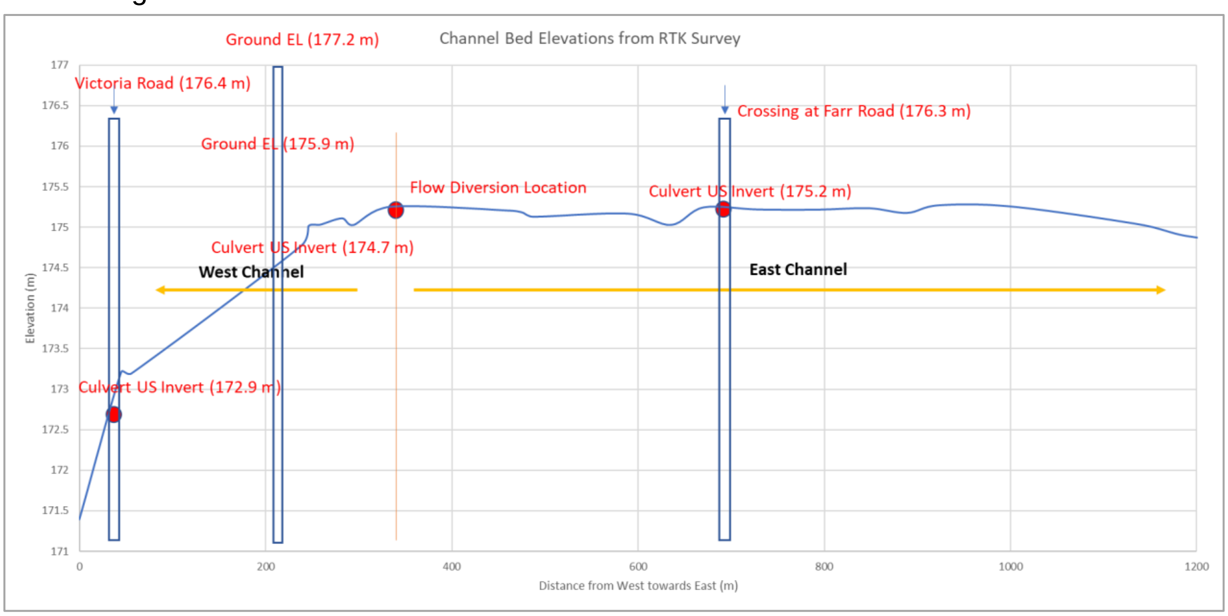


Figure 4-1: Changes in Channel Bed Elevation from West (Welland River) towards East

4.1.2 EXISTING CONDITION OF THE CULVERTS

It was observed from the field program as well as analysis of the historical aerial images, some alignment of the watercourse had shifted from their initial position between the Victoria and Farr Road. There may be a variety of causes for shifting, but land filling and development activities are likely to be two of the main causes. Previously, a channel was running from the Farr Road in the west direction, while another was flowing in the opposite direction, towards the East. After initial evaluation of the field data, it was discovered that the flow diversion point had moved towards the west. Additionally, a few of the culverts are not operating properly. They no longer have as much capacity because of obstruction and structural deficiencies. **Figure 4-2** represents the existing condition of the culverts in the study area.

Near the Victoria Avenue | *Webber Road* | *Farr Road*

Victoria Avenue | *Farr Road* | *Webber Road*

Figure 4-2: Existing Condition of the Culverts

4.1.3 VERIFICATION OF LIDAR DATA

The accuracy of the LiDAR data was checked using the RTK system in order to obtain the coordinates and elevation of several control points within the study area which were later compared with the LiDAR data. Table 3.1 shows the vertical accuracies of LiDAR data with respect to the control points where AHYDTECH has surveyed using GNSS RTK/GPS. The differences between the elevations of LiDAR data and survey data were compared. It is observed that the differences were minimal, and the LiDAR data was determined suitable for use in the study. The Easting and Northing are as per UTM projections (NAD83) while the vertical datum is CGVD13.

Table 4-1: Comparison of vertical accuracies of LiDAR Data and field data

Sl. No	Easting	Northing	LIDAR DTM Elevation(m)	Surveyed Elevation (m)	Difference (m)
1	632448.7	4760429.11	175.465	175.1668	0.2982
2	632455.8	4760428.6	175.566	175.4313	0.1347
3	632655.3	4760404.97	175.395	175.3346	0.0604
4	632708.3	4760405.18	175.3	175.2336	0.0664
5	632992.5	4760352.42	175.09	175.036	0.054
6	633438.3	4760263.47	174.4781	174.57	-0.0919
7	633535	4760278.64	174.32	174.2661	0.0539
8	632109.1	4760468.59	175.12	175.0307	0.0893
9	632073.8	4760438.24	177.14	177.163	-0.023
10	631951.2	4760535.85	173.43	173.2299	0.2001
11	631942	4760547.22	173.97	173.9099	0.0601
12	631928.2	4760534.48	176.4	176.4073	-0.0073

4.2 RAPID FLUVIAL GEOMORPHIC ASSESSMENT

AHYDTECH performed rapid fluvial geomorphic assessment while conducting field survey at the study area. The survey team had undertaken field assessment to characterize existing stream conditions within the study area. Channel stability was assessed and investigating the condition of the channel no fluvial processes were observed along the channel. However, alteration of channel at some locations were observed. Development and advancement of the area required additional landfilling activity which caused channel alteration. Additionally, it affected the conveying capacity of the channels, culverts and other hydraulic structures present within the study area. **Figure 4-3** shows the existing condition of different places.

Figure 4-3: Existing Conditions at Different Locations

5 HYDROLOGIC ANALYSIS

5.1 INTRODUCTION

The assessment of the drainage condition of the study area requires the development of a hydraulic model of the associated watercourse. A hydraulic model has been developed for the watercourse within the study area which needs flow at the upstream boundaries as one of the key input parameters. However, no observed discharge data is available for any section of the watershed thus prompting the development of a hydrologic model. An event-based hydrologic model has been developed in order to compute flows for the corresponding node points of the hydraulic model. The hydrologic model uses rainfall data to simulate runoff for specific storm events (2-,5-,25- and 100-Year). The flows obtained for different return period conditions has been used for the development of the hydraulic model and to assess the inundation extent of the study area.

The hydraulic model being developed has a domain that extends from the Victoria Road and ends near the Balfour Street. From early impressions, the drainage area of the creek appears to be much lower than 5 sq. kilometers. After delineation, the drainage area turned out to be 2.74 sq. kilometers. **Figure 5-1** shows the watercourses, the drainage area and watershed boundaries for the hydrologic and hydraulic analyses.

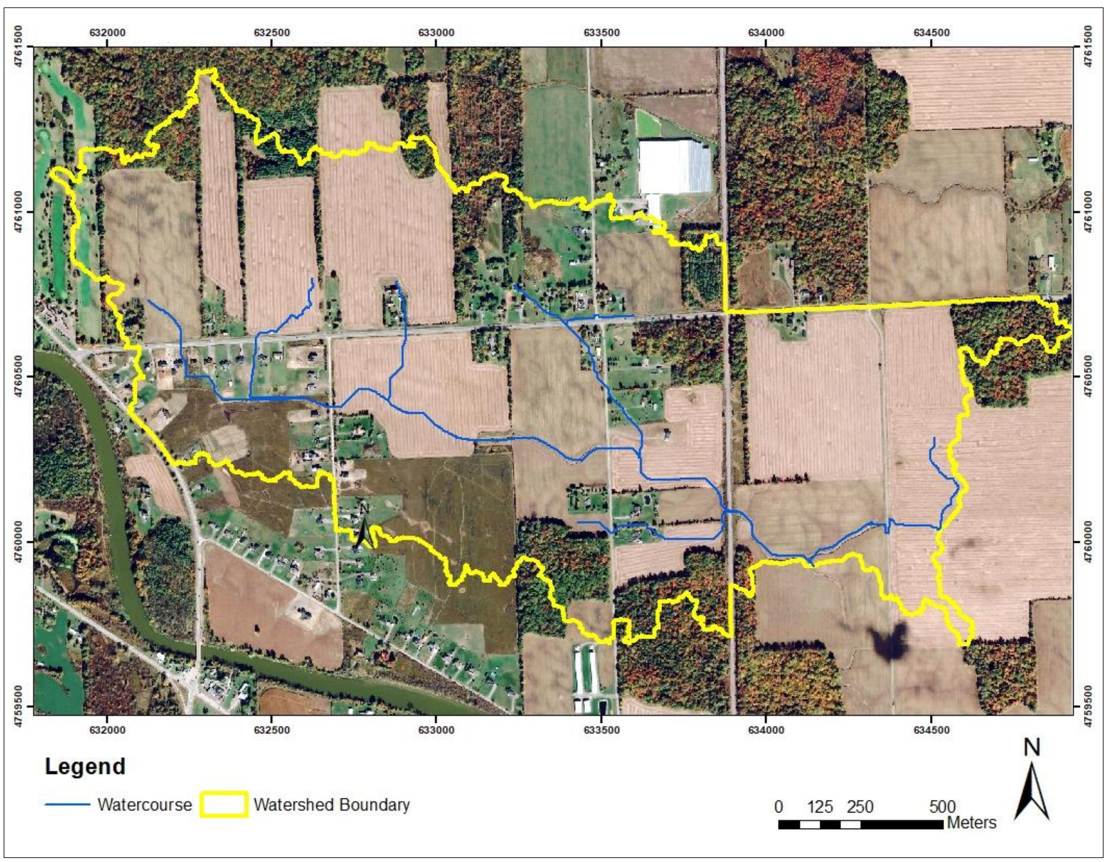


Figure 5-1: Watershed Boundary at Farr, Webber, and River Road Area

5.2 MODEL SELECTION

The watershed has been modelled using SWMHYMO software. SWMHYMO, developed by the J.F. Sabourin and Associates Inc. (JFSA), is a complex hydrologic model that can compute hydrologic analysis at low cost and low computational time. The model is widely used for small to large rural and urban areas. The SWMHYMO model computed flows at the corresponding node points of the hydraulic model. The computed flows have been then routed to simulate the flows for runoff. Before setting up the SWMHYMO model, the spatial delineation of the catchments into sub-basins were carried out using HEC-GeoHMS in the ArcGIS Environment.

The hydrologic model has been developed for a few specific storm events. These events represent storm events corresponding to different return period conditions. Peak flows corresponding to selected return periods was obtained from the hydrologic model where the return periods specified for the study are 2-, 5-, 25- and 100- year. The hydrologic model developed is an uncalibrated one, due to unavailability of discharge data nearby the watershed.

5.3 DATA COLLECTION AND PRE-PROCESSING

For the development of the event-based hydrologic model, the following types of data have been used:

1) Hydrometeorological data of the study area (i.e., Rainfall Data)
2) Hydrogeology of the study area (i.e., Soil Data)
3) Land Use of the study area (i.e., Land-use and Land-cover Data)
4) Topographic data (i.e., Digital Elevation Model)

A brief description of each of the data types has been given in the upcoming sub-sections.

5.3.1 SOIL DATA COLLECTION AND PROCESSING

The soil data for the study area has been collected from the "Soil Survey Complex", prepared by the Ministry of Natural Resources (Ontario, 2019). The Ontario soil survey follows "Canadian System of Soil Classification," and the data layer has varied spatial resolution. The Soil Survey Complex (SSC) data model supports up to three soil components across one map unit (or polygon). The SSC data model is compliant with the National Soil Data Base (NSDB) data model for Detailed Soil Surveys.

Soil has been classified into some groups which are termed as Hydrologic Soil Groups or HSG based on measured rainfall, runoff, and infiltrometer data. The HSGs are of four types: Group A, Group B, Group C and Group D (USDA, NRCS, 2007a). Analysis of soil data for the study area shows that Group C is mostly dominant within the drainage areas. Group C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (USDA, NRCS, 1986). These soils have 20%-40% clay and less than 50% sand and have loam, silt loam, sandy clay loam, clay loam, and silty clay loam textures (USDA, NRCS, 2007b). **Figure 5-2** shows the soil type distribution along the delineated watershed.

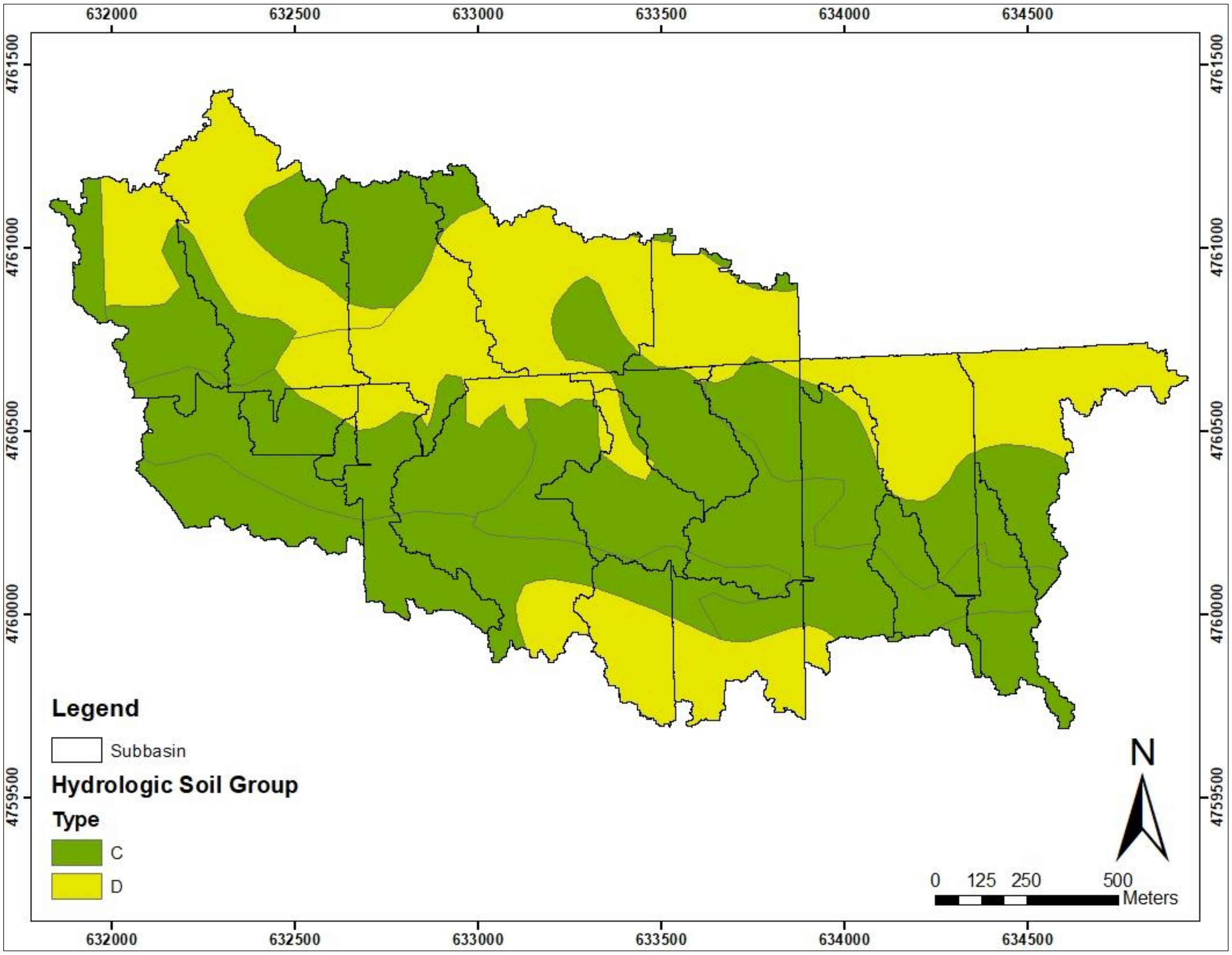


Figure 5-2: Soil Type Distribution along the Delineated Watershed

The percentage for each of the soil group within the watershed of the study area is presented in **Table 5-1**

Table 5-1: Percentage of Soil Type over the Delineated Watershed

Soil Type	Area Covered (sq.km)	Percentage of Area
C	1.6513	60.26
D	1.0889	39.74
Total Area	**2.74**	**100.0**

5.3.2 LAND-USE AND LAND COVER (LULC) DATA COLLECTION AND PROCESSING

The ESRI global land use land cover (LULC) map, derived from ESA Sentinel-2 imagery at 10m resolution, was used as land-use and Land cover data. The dominant LULC type across the delineated watershed is illustrated in **Figure 5-3**.

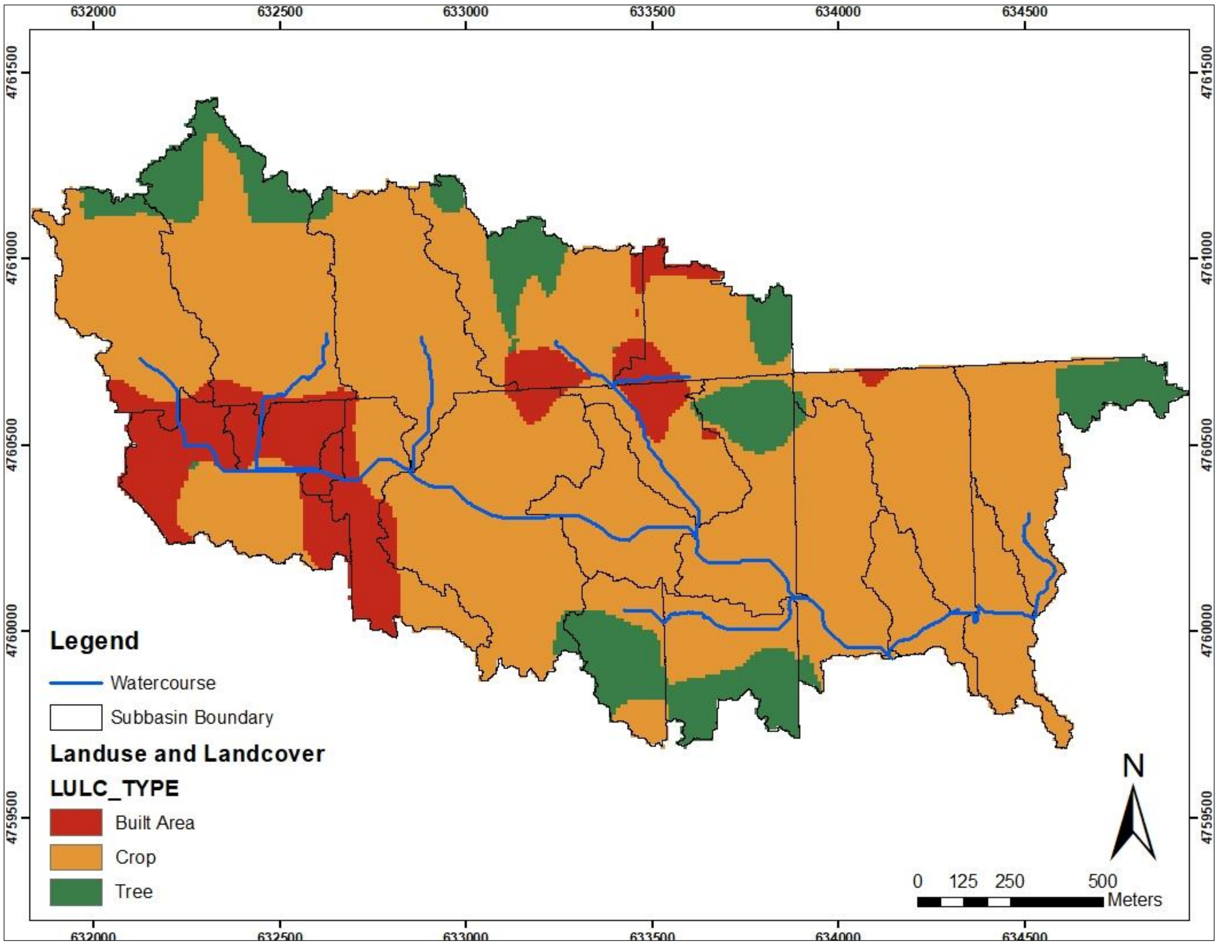


Figure 5-3: Land use and Land type for the Delineated Watershed

5.4 WATERSHED DELINEATION AND CHARACTERIZATION

The watershed was delineated using HEC-GeoHMS, a geospatial hydrology tool which is an extension of ArcGIS software. Required data for the pre-processing include Digital Elevation Models (DEM), digital stream alignments, and stream gage locations. DEM was obtained from [Ontario Geo-Hub] (Resolution of 0.5m) and the watercourse for the study area was obtained from [Ontario Hydro Network (OHN) - Watercourse.]

An outlet was specified based on which the drainage area upstream of the outlet was specified. The delineated watershed has a total of 21 subbasins with an area of approximately 2.74 sq.km **(Figure 5-4).** The subbasins generated was named with the initials beginning with “SB” followed by a twodigit number suffix which provided each subbasin with a specific name for identification purpose. The following steps were followed sequentially in order to obtain the drainage area as a part of the pre-processing part for the development of hydrologic model.

Pre-processing: Catchment Delineation and Characterization of Streams and Subbasins

Based on DEM obtained by AHYDTECH for the study area, the watershed was delineated into subbasins following the workflow of the process as a part of pre-processing. The result of the pre-processing followed the determination of an outlet point in order to obtain subbasins for the catchments. Followed by basin processing, the subbasins and the streams were characterized based on the topographic information.

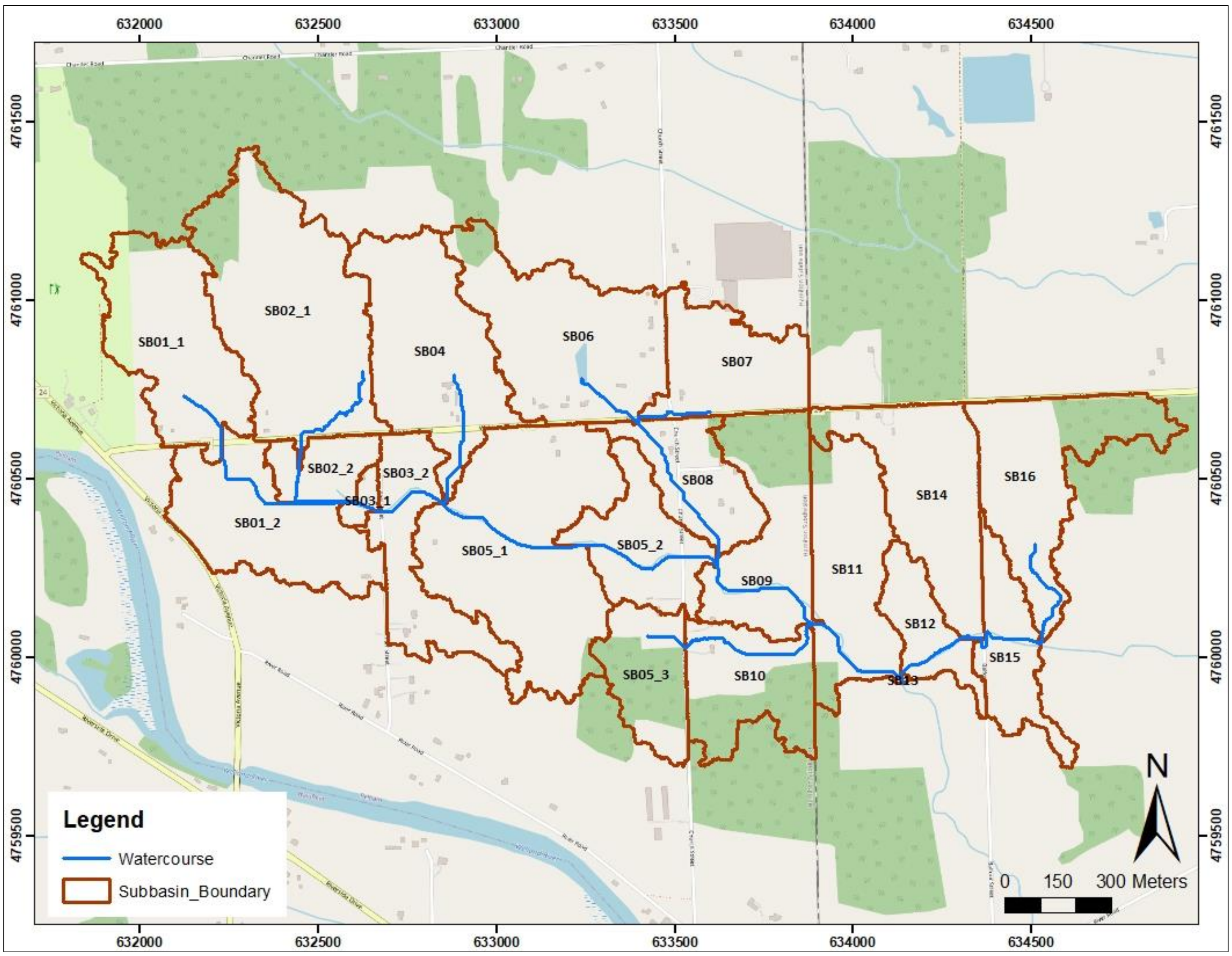


Figure 5-4: Subbasins Delineated for the Study Area

The primary subbasin characteristics are the subbasin area, longest flow-path length, and river length. The distance between the subbasin outlet to the most hydraulically remote point upstream within a subbasin is defined as the longest flow-path length. Longest flow-path is used to determine the time of concentration for a watershed. The river length and the subbasin area has been computed where the former has been used for routing purpose and the latter was computed for surface run-off flows through a series of streams to a particular point in a watercourse. Besides, the Time of Concentration (T_c) has been estimated using TR-55 methodology (details in **Section 5.6**) which breaks watershed flows into sheet, shallow concentrated, and channel flows. TR-55 flow path segments are created for each kind of break.

5.5 METEOROLOGICAL ANALYSIS

Since there is an absence of discharge data nearby the study area, rainfall data has been used to simulate runoff data. For the development of the SWMHYMO model, 24-hour rainfall distributions have been prepared, where AHYDTECH made use of the Ministry of Transportation IDF Curve Lookup Table to determine the rainfall for storm events of the specific return periods (2-,5-,25- and 100-Year). The details of the meteorological data obtained from MTO IDF Curve Lookup Table in provided in **Appendix A.**

5.6 HYDROLOGIC MODEL PARAMETER DETERMINATION

A SWMHYMO data file was developed to analyze the peak flow for the specific return period condition within the delineated boundary. Two commands of the SWMHYMO model, CALIB NASHYD and CALIB STANDHYD were used to determine the flows from each of the subbasins. As identified that the study area was mostly rural, "CALIB NASHYD" command was used to simulate the runoff for the rural area. Again, CALIB STANDHYD command was used for areas where imperviousness was more than 20%. Hydrologic Model parameters and SWMHYMO node point name has been provided in **Appendix C**.

CALIB NASHYD

This hydrograph command, based on Nash's synthetic instantaneous unit hydrograph, is used to simulate runoff from a rural area or very large urban watershed and effects of inflow and infiltration in sanitary sewers. The CALIB NASHYD command requires the following inputs for computation of flows based on the precipitation data **(Table 5-2)**:

Table 5-2: Parameters for CALIB NASHYD

Parameter Name	Description
Area	Area of the catchment (hectares)
DWF	Dry weather flow components (m^3/s)
CN	Curve Number
DT	Computational time step (minutes)
N	Number of lineal reservoirs
T_p	Time to peak (hour)

CALIB STANDHYD

This hydrograph command is used to simulate runoff from urban watersheds with impervious areas greater than 20%. It used two parallel instantaneous unit hydrographs to simulate runoff from pervious and impervious surfaces. The CALIB STANDHYD command requires the following inputs for computation of flows based on the precipitation data **(Table 5-3)**

Table 5-3: Parameters for CALIB STANDHYD

Parameter Name	Description
Area	Area of the catchment (hectares)
DWF	Dry weather flow component (m^3/s)
CN	Curve Number
TIMP	Total imperviousness ratio (between 0.0 and 1.0)
XIMP	Directly connected imperviousness ratio (between 0.0 and 1.0)
LOSS	Type of loss over impervious surface
DT	Computational time step (minutes)
IAper	Initial abstraction on pervious surface (mm)
SLPP	Average pervious surface slope (%)
LGP	Average lot depth (m)
MNP	Roughness coefficient for pervious surface
SCP	Linear reservoir storage coefficient for pervious surface (minutes)
IAimp	Initial abstraction on impervious surface (mm)
SLPI	Average impervious surface slope (%)
LGI	Average overflow travel length (m)
MNI	Roughness coefficient for impervious surface
SCI	Linear reservoir storage coefficient for impervious surface (minutes)

The dry weather flow or base flow has been assumed as zero (DWF = 0.0). The number of linear reservoirs was used as three (N = 3). These values are normally assumed due to limitation of site-specific detailed information.

Time of Concentration and Time to Peak

Time of Concentration (T_c) is attributed to the time taken for a runoff to travel from the most distant point in the watershed to the point of interest. Many methods are available, most of which are empirical and developed for specific condition. Here the 'velocity method' (also known as TR-55 method), originally introduced by Soil Conservation Service (USDA-SCS, 2010) and later elaborated by Natural Resources Conservation Service (USDA-NRCS, 2010). The details for the calculation of T_c are provided in the document *"Urban Hydrology for Small Watersheds – TR-55"*

The velocity method calculates T_c, segmenting the flow into sheet, shallow concentrated and channel types and T_C is obtained by summing up travel time off all the components of the drainage conveyance system as listed below:

Sheet Flow: Sheet flow is flow over the plane surfaces and usually occurs in the headwater of the streams.

Shallow Concentrated Flow: After about 100 ft, sheet flow usually becomes shallow concentrated flow, in which the average velocity is a function of watercourse slope and type of channel.

Open Channels: Open channels are assumed to begin where surveyed cross-sectional information has been obtained, where channels are visible on aerial photographs.

T_c was obtained through the TR-55 method in HEC-GeoHMS which defined different flow regimes along the longest flow path, placing two points along the longest flow path for each subbasin. The first point, AA, marks the break between sheet flow and shallow concentrated flow, which is located about 100 feet from the watershed divide along the longest flow-path. The second point, BB, marks the break between shallow concentrated flow and channel flow and is by default located where the longest flow path first encounters the channel. T_c obtained was then used to calculate the Time to Peak (T_p) value and was estimated to be two-third of the value of T_c.

T_p, which is also sometimes denoted as Lag time, is defined as the time between a rainfall event and the corresponding peak flow. Thus, time to peak can be represented as:

$$T_p = \left(\frac{2}{3}\right) * T_c$$

Curve Number

The Curve Number (CN) Method, developed by the Soil Conservation Service (Soil Conservation Service 1964; 1972) for estimating runoff is widely used in North America and elsewhere. Using Hydrologic Soil Group (A/B/C/D) and LULC Type (ESRI), the Curve Number value was determined for 10m resolution grids. The Curve Number values corresponding to a particular LULC type and Hydrologic Soil Group to the *"Urban Hydrology for Small Watersheds – TR-55"* of USDA and NRCS was adapted for estimating the curve number.

Channel Routing

To simulate the flow of runoffs through channels and reservoirs, "ROUTE CHANNEL" command of the SWMHYMO model was used. The input parameters include: channel length, slope, roughness and cross-section of the channel. Manning's roughness coefficient for both the channel and floodplain was assigned based on the land use and land cover features. The Manning's roughness coefficient used for the SWMHYMO model was in consistent with the HEC-RAS model. The input and the output files of the SWMHYMO model is provided in **Appendix D** and **Appendix E** respectively.

5.7 SENSITIVITY ANALYSIS

Sensitivity analysis was done for the hydrologic model to estimate the influence of model parameters on the flow at the outlet. A table is given below showing the summary of sensitivity analysis of the hydrologic model. The first row is marked with yellow color indicating that simulation was done with all the initially assumed values. From the second row onwards, the blue cells indicate how we have changed the parameter values, to observe the change in peak discharge for the 100-year storm, at the outlet of the SWMHYMO model (outlet at SB13). For example, (x1.15) means we have multiplied the initially assumed values for each subbasin by the factor 1.15.

Table 5-4: Sensitivity Analysis of the Hydrological Parameters

Simulation No.	Time to Peak	Curve Number	Initial Abstraction	Slope	Manning's n		Flow at Outlet (cms)
					Channel	Bank	
A0	×1	×1	×1	×1	0.055	0.035	11.784
A1	×1.15	×1	×1	×1	×1	×1	11.056
A2	×0.85	×1	×1	×1	×1	×1	12.681
B1	×1	×1.15	×1	×1	×1	×1	15.756
B2	×1	×0.85	×1	×1	×1	×1	9.034
C1	×1	×1	×1.15	×1	×1	×1	11.63
C2	×1	×1	×0.85	×1	×1	×1	11.958
D1	×1	×1	×1	×1.15	×1	×1	11.947
D2	×1	×1	×1	×0.85	×1	×1	11.688
E1.1	×1	×1	×1	×1	0.085	×1	11.663
E1.2	×1	×1	×1	×1	0.045	×1	11.837
E2.1	×1	×1	×1	×1	×1	0.045	11.871
E2.2	×1	×1	×1	×1	×1	0.015	12.873

It can be observed from the model results that, the model is sensitive to most of the model parameters, where it is highly sensitive to the value of Curve Number (CN) and Time to Peak (T_p). Besides, the model is also sensitive to other parameters as well.

5.8 FLOW RESULT

In order to assess the drainage condition and identify the areas which are susceptible to flooding, a 1D hydraulic Model has been developed using HEC-RAS software. The HEC-RAS model makes use of discharge as upstream boundary condition. The SWMHYMO model has been used to generate flows for several inlets/junctions within the watershed at this stage. illustrates the locations identified inlets and junctions for the flow generation. The value of the generated flow (cms) at each inlet and junction have been demonstrated in **Figure 6-9.** Although the flow direction location has been observed at a certain location, the SWMHYMO model assumes all the flows in a single direction, i.e., from upstream towards downstream. However, based on the topography and other site features, the flow that was obtained from the SWMHYMO model, has been diverted into two directions, one towards the west and other towards the east in the HEC-RAS model.

6 DEVELOPMENT OF HYDRAULIC MODEL

To assess the drainage condition near the Farr, Webber, and River Road at the Town of Pelham; AHYDTECH has developed a 1D hydraulic model of the watercourses through HEC-RAS. The hydraulic model results were analyzed to assess the potential drainage issues and provide suitable mitigation measures within the study area.

6.1 HYDRAULIC MODEL SELECTION

HEC-RAS is a software for one-dimension or two-dimensions simulations of the evolution of a flood, which could have a steady or an unsteady flow rate, sediment transport, change of the riverbed etc. The name 'HEC-RAS' derived from the creators of the software: Hydrologic Engineering Center, which stands as a subdivision of the Institute of Water Resources, U.S Army Corps of Engineers (HEC), and "RAS" is an acronym from "River Analysis System". The software itself, has four main river analysis possibilities: the constant flow rate at the surface of a considered river profile; simulation of an unsteady flow of water; calculations of the sediment transport and modifications of the riverbed; and analysis of the water quality (U.S. Corps of Engineers, 2003, Tate et al. 1999). It is a widely used tool to model 1D flow simulation and provides graphical results. Inundation mapping is accomplished in the HEC-RAS Mapper portion of the software and Tabular output is also available. Model results can be post-processed using SMS software to obtain and map detailed results. When the length-to-width ratio is larger than 3:1, a 1D hydraulic model can provide fairly good results (UK Environment Agency, 2009). More detailed information about HEC-RAS can be obtained from the website: http://www.hec.usace.army.mil/software/hec-ras/.

The unidimensional hydraulic modeling of the study area has been carried out using HEC-RAS version 6.1.

6.2 HEC-RAS MODEL DOMAIN

Based on the figure in the TOR, it appears that some sections of the watercourses are not connected to each other. Analyzing the tentative flow direction map, two distinct outlets (illustrated on **Figure 6-1**) were incorporated within the hydraulic model. The domain of this hydraulic model has been taken such that they begin from where the watercourses are deemed "discernible" based on NPCA definition and up until they flow into the Welland River. Watercourses were represented by 6 Rivers, 10 Reaches, 4 junctions into the model which have been further explained in **Section 6.3.1.**

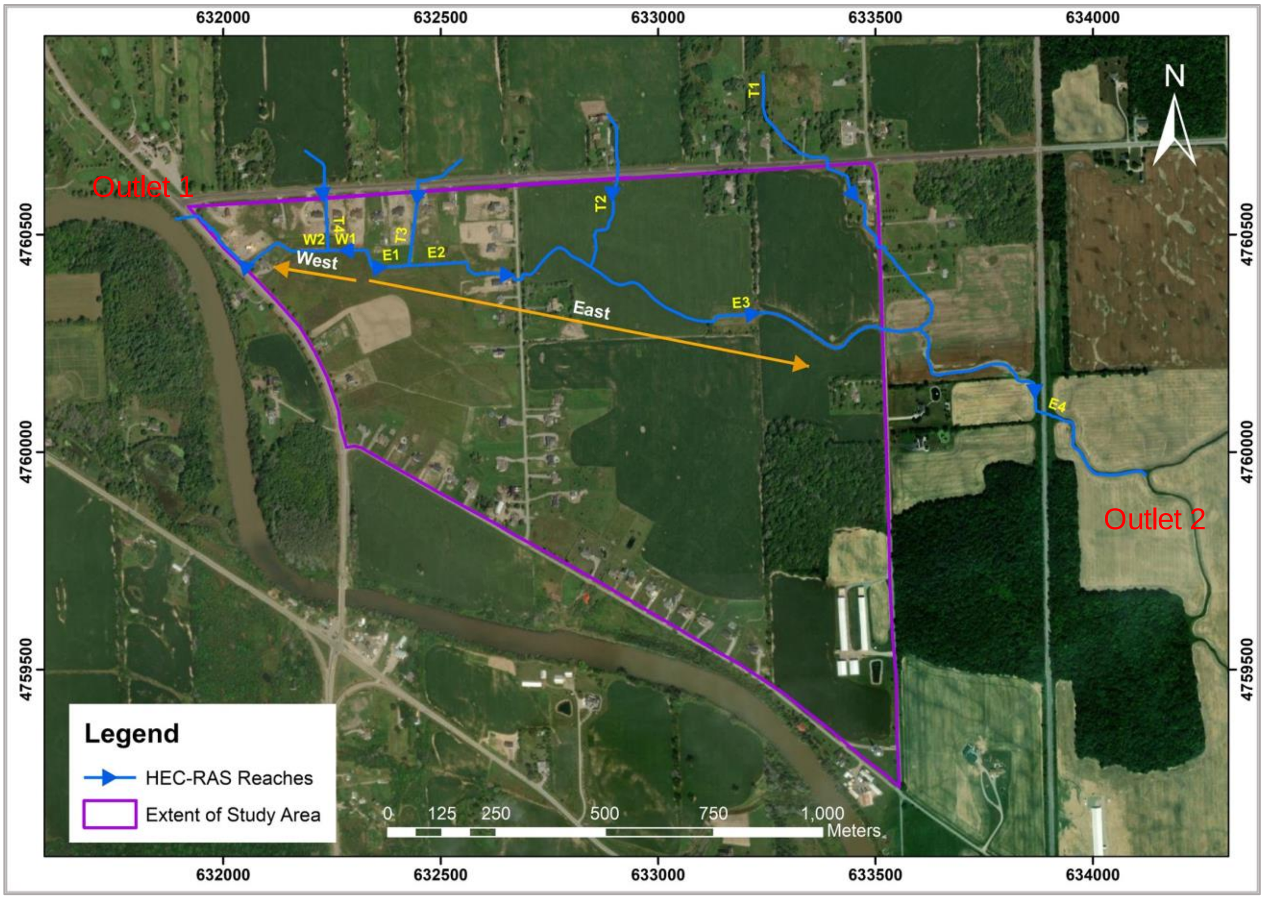


Figure 6-1:Simulated reaches in HEC-RAS Model for Existing Condition of the study area

6.3 1D HEC-RAS MODEL DEVELOPMENT

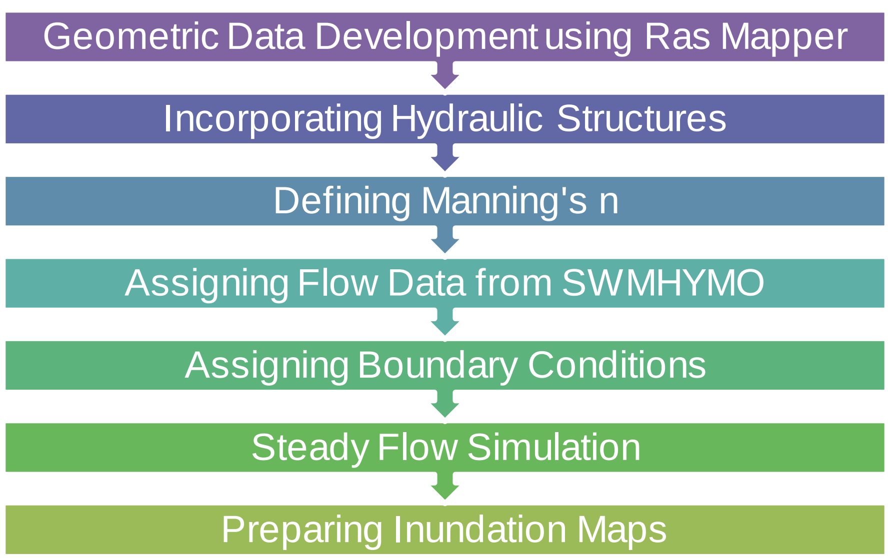


Figure 6-2: Methodology for 1D Hydraulic Model Development for the Study area

6.3.1 GEOMETRIC DATA DEVELOPMENT USING HEC-RAS MAPPER

The HEC-RAS Mapper module is an interface that can be used from the main HEC-RAS software. It offers a geospatial depiction of the HEC-RAS geometry, simulation results, and other relevant geospatial data to help users develop river hydraulic models.

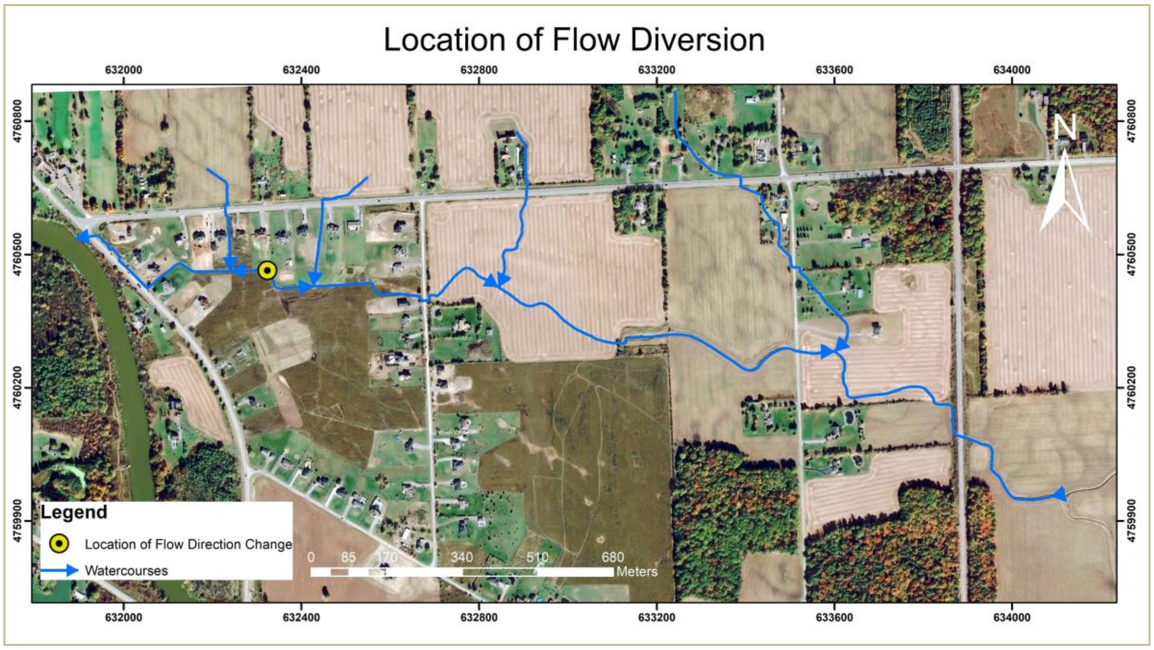


Figure 6-3: Flow Diversion Location at the Study Area

AHYDTECH used HEC-RAS 6.1 to create a 1D hydraulic geometric data into the model to analyze the drainage pattern of the study area near Farr, Webber and River Road, Town of Pelham. To determine the existing location of flow diversion, i.e., the location from which flow diverts towards east and west direction within the study area, AHYDTECH assessed the collected field survey data and Digital Elevation Model (DEM) which has been discussed in detail in **Section 4. Figure 6-3** shows the estimated flow diversion location (as obtained from the field survey assessment) within the study area based on which geometric data has been developed for the existing condition of the project site.

6.3.1.1 RIVER CENTERLINE, BANKLINES, FLOWPATHS

The centerlines of the watercourses within the project study area were delineated in the downstream direction in HEC-RAS Mapper using DEM from LiDAR data and field survey data. Two individual drainage systems were considered into the model based on the two outlets in East and West directions that were specified in RFP.

River Centerline is used to assign the river station (RS) values of the cross-sections, measured from the downstream end to upstream along the river reach and compute the main channel reach lengths between two cross sections by utilizing the HEC-RAS Mapper.

Bank lines are used to define the main channel banks for a cross section. It can be both continuous and discontinuous and can be drawn from any end of the watercourse. Though RAS Mapper enables users to auto-generate the bank lines using DEM/DTM, AHYDTECH has defined the bank lines considering the bank position data collected during the field survey since it provides more accurate and recent data. For each watercourse, left and right banks have been defined for the study area, which has been illustrated in **Figure 6-4**.

Flow path lines are used to compute reach lengths between cross-sections in the left and right overbank which were delineated towards the downstream direction. It also facilitates the user to auto-generate the lines. However, AHYDTECH used the field data to delineate the flow path lines for the study area.

6.3.1.2 CROSS-SECTION CUTLINES

Cross-sections are developed using the attributes of other layers, such as the River, Bank Lines, and Terrain layers, as well as the spatial layout of the cross-section lines. The primary data source used in characterizing cross-sections in the study area is the cross-section dataset collected during RTK Field Survey by the Senior Engineer. A total of 231 cross-sections have been taken using approximately 50m of spacing for the watercourses within the study area **(Figure 6-4)**. When each cross-section is defined, RAS Mapper automatically compute the River Name, Reach Name, River Station, Bank Station, Reach Length, and other data for the cross-section and automatically extracts the elevation data from the DEM/DTM. The RAS Mapper enables the user to manually insert the ground elevation point data which assist users in updating cross-sections where field data and/or bathymetric data are available. Since, AHYDTECH has completed the field survey and collected the elevation data of the drainage lines and all other relevant data required for the model, those data were applied in RAS Mapper when developing cross-sections for the study area.

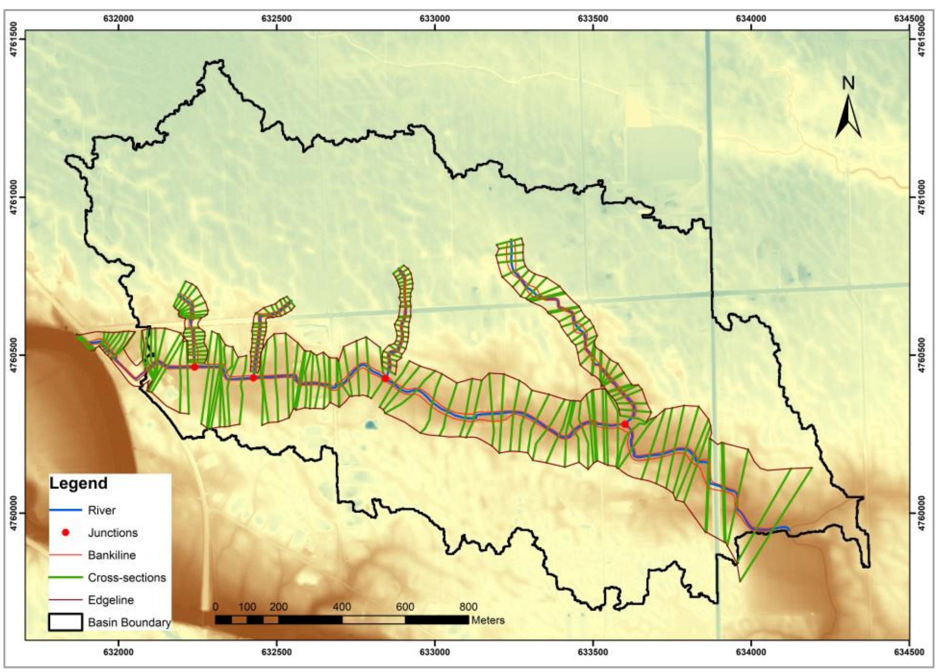


Figure 6-4: Geometry Data Developed in RAS Mapper

6.3.1.3 DEM MODIFICATION

The basis for any accurate hydraulics is a good representation of ground surface elevations for the watercourses and floodplain areas. A good DEM/DTM accurately describes the elevations of the channels and floodplains by incorporating important features that regulate water movement, such as the channel bottom and channel banks and high grounds such as the roadways and levees.

AHYDTECH has collected all the relevant data required for the hydraulic analysis of the study area including cross-sections of the channels passing through the study area, left and right bank of the channels and floodplains. When geometric data has been developed in RAS Mapper, it automatically used the elevation data derived from the 2021 LiDAR-based DEM data for the 231 cross-sections. Given that the LiDAR-based DEM data is insufficient to represent the bathymetry of the main channels, the elevation data derived from the survey was employed using geometry editor to substitute the main channel portion of the LiDAR-based cross-figure geometric data, for example the cross-section at RS 146 as presented in **Figure 6-5** below. The 'Hybrid' cross section geometric data were generated by making minor adjustments to station elevation data points and other parameters.

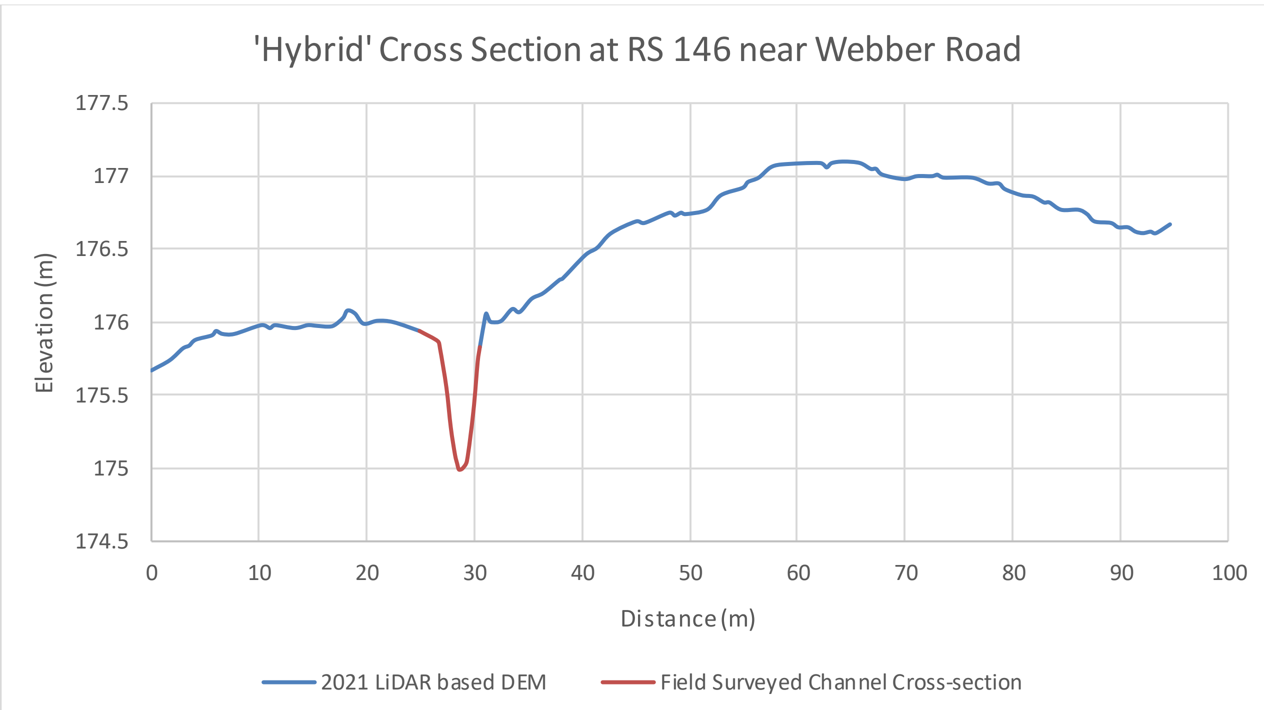


Figure 6-5: Modified Cross Section at RS 146 using LIDAR DEM and Surveyed Data

6.3.1.4 CROSS-SECTION INTERPOLATION

Cross-section interpolation is necessary to supplement surveyed cross-section data in between two surveyed cross-sections, since the LiDAR DEM data isn't sufficient to represent all the points/sections along a channel within the study area and field survey data of the channel weren't taken at constant interval. During the survey data collection, there were some locations where it was difficult to access due to dense and high vegetation as well as landfills. Since the surveyed cross-section data have been incorporated into the DEM using terrain modification tool in RAS Mapper, cross-section interpolation surface can be computed using RAS Mapper. Interpolation surface in RAS Mapper is constructed from the river centerline, cross- section cutlines, bank lines and edge lines, which is used for interpolating hydraulic model results. **Figure 6-6** shows the cross-section interpolation surface in the RAS Mapper.

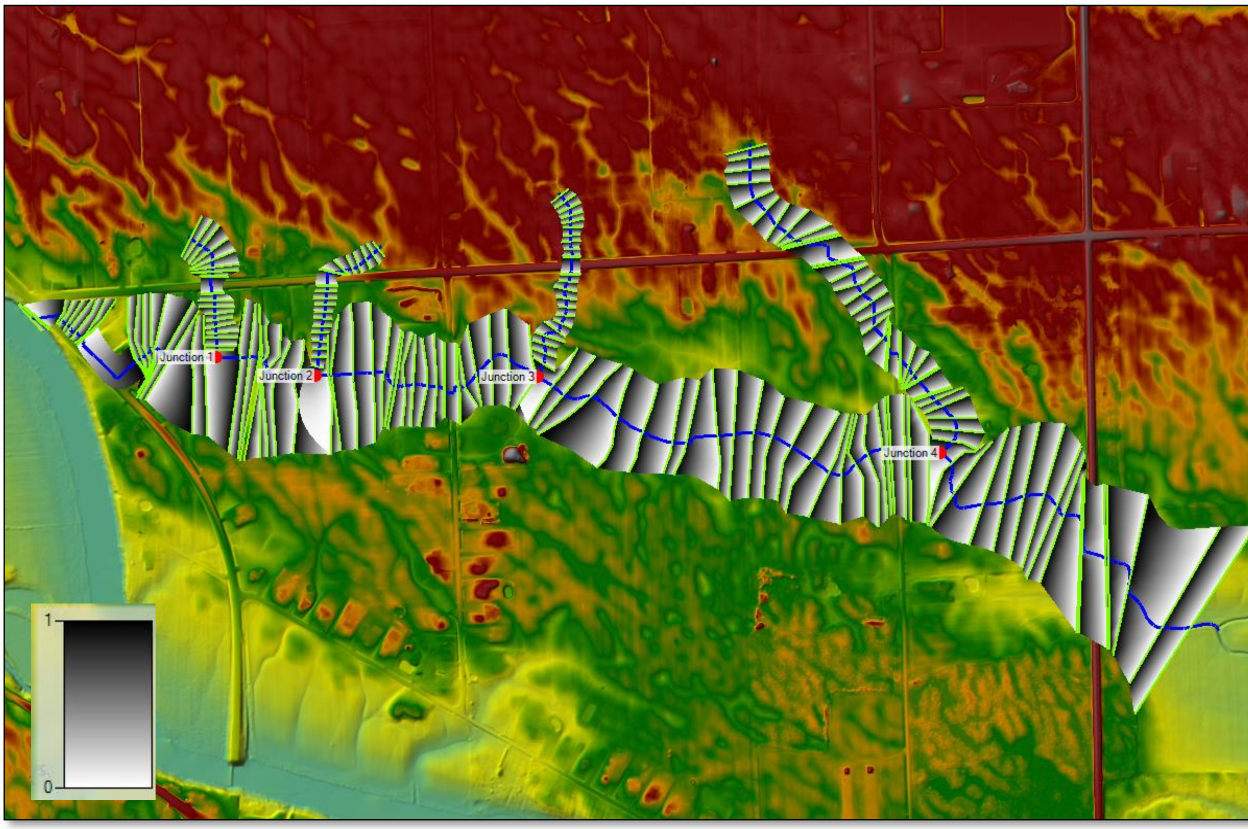


Figure 6-6: Cross-section Interpolation Surface in RAS Mapper

6.3.1.5 MODELING HYDRAULIC STRUCTURES

After entering all the necessary cross-section data, culverts within the study area were added along the channels using 1D hydraulic elements. The study area covers 11 culverts which have been incorporated and simulated as 1D culvert in the model. Dimensions, upstream and downstream conditions, and elevation data of the culverts were collected during the field survey. Culvert barrels were made of Corrugated Steel Pipe (CSP) and most of these are circular in shape. Culvert and road embankments were also included in the model, which were used to refer any area blocking the stream and the roadway. **Figure 6-7** illustrates typical cross-sections of a culvert modeled in HEC-RAS from both upstream and downstream of the culvert.

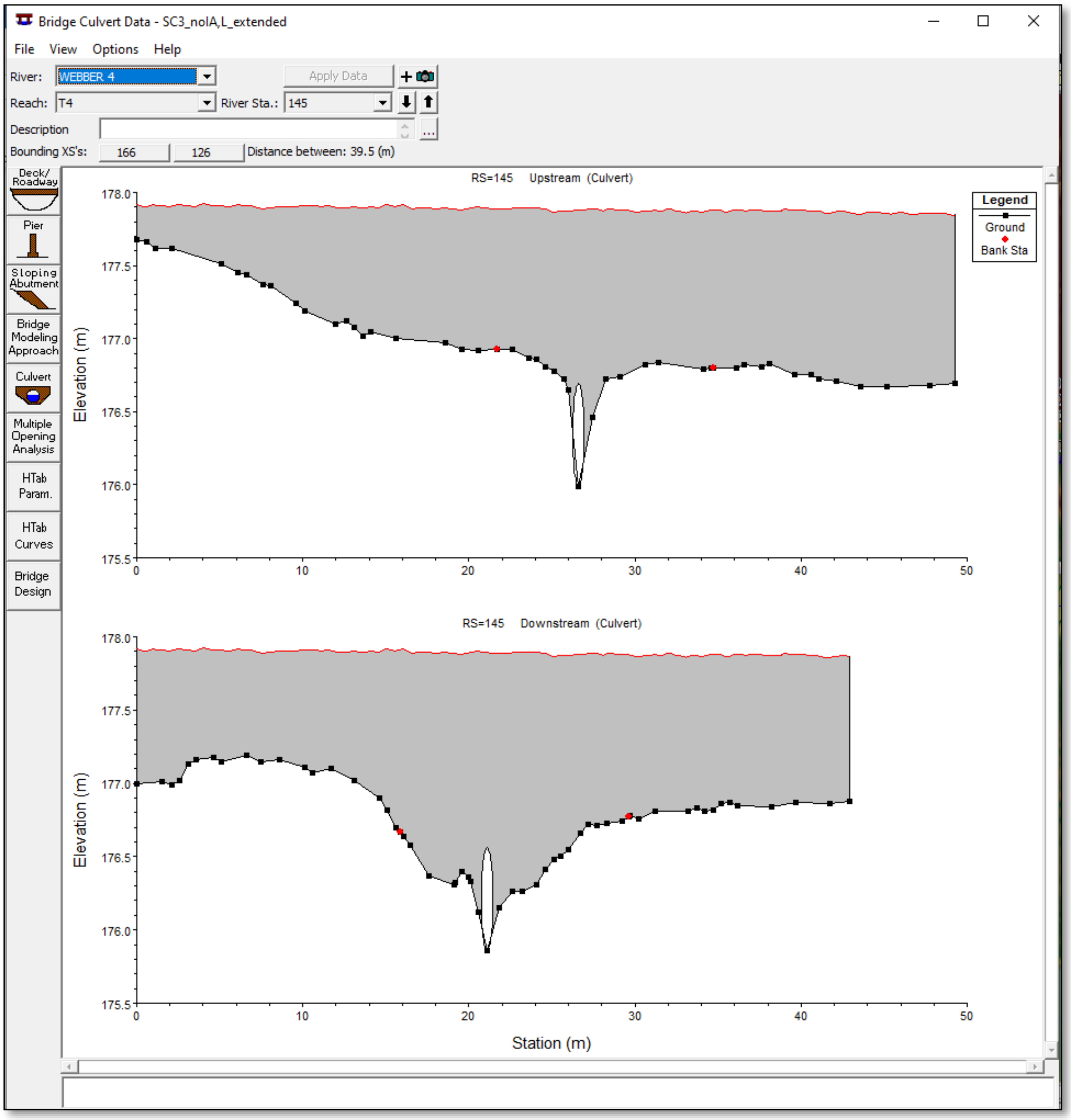


Figure 6-7: Sample profile of a Culvert at Webber Road between RS 126 and RS 166

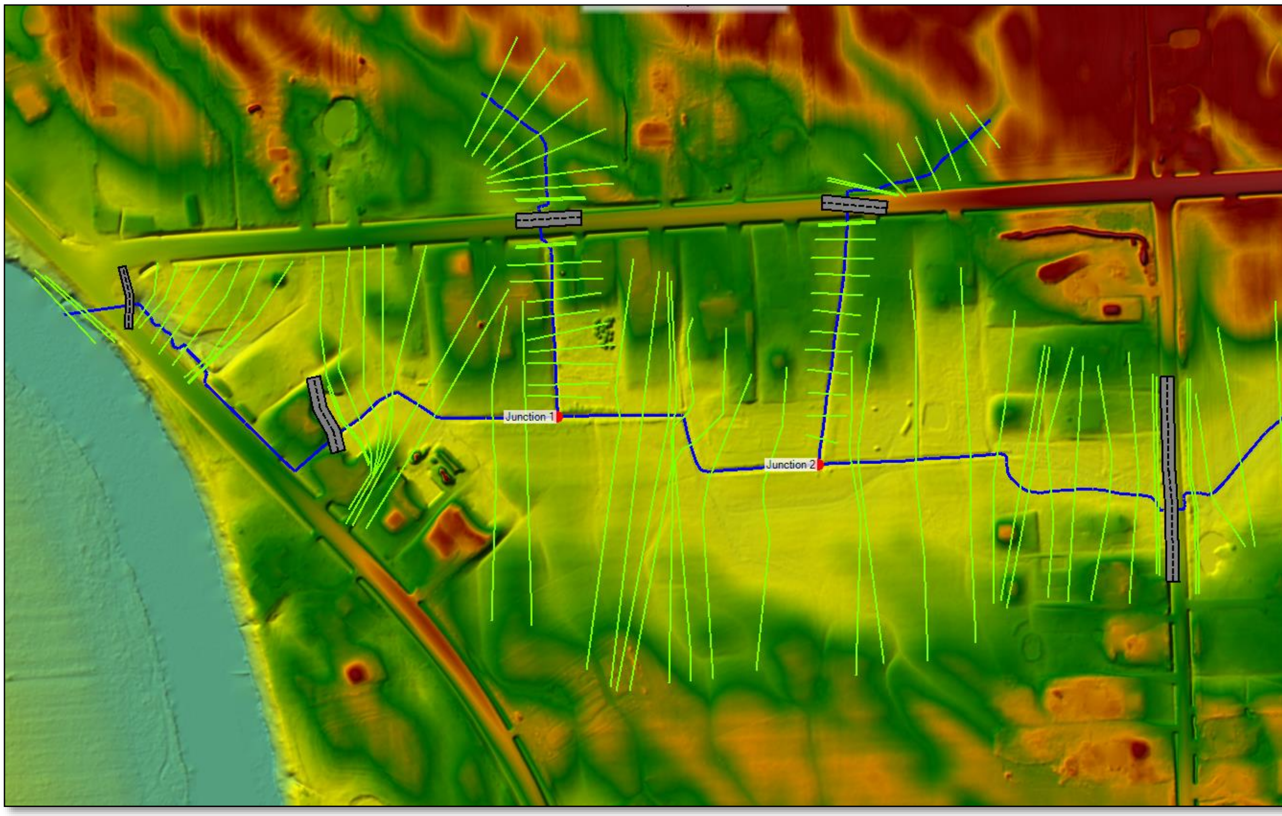


Figure 6-8: Some of the Culverts with Road Decks at the study area

6.3.1.6 MANNING'S ROUGHNESS COEFFICIENT

The accuracy of the simulated water surface levels in hydraulic modeling depends on the parameterization of Manning's roughness coefficient (n). It reflects hydraulic resistance for flow in a river and flood plain. Roughness in the main channel is usually lower than that in the flood plain. It also varies in the flood plain depending on the type of land use/land cover.

Manning's roughness coefficients (n) were determined based on land use and land cover of the study area following the *Technical Guidelines for Flood Hazard Mapping (EWRG, March 2017)*. Manning's n for Corrugated Steel Pipe and Corrugated Plastic Blade Culverts were also determined following the guidelines mentioned. The dominant land use/cover along the left bank and the right bank have been determined and the Manning's n values have been assigned accordingly. **Table 6-1** shows the range of values of Manning's "n" for different land use/cover as per the guidelines.

Table 6-1: Manning's Roughness Coefficient

Open Channel	'n' Standards	Calibration Range	
		Minimum	Maximum
Overbank			
Woods	0.08	0.04	0.12
Meadows	0.055	0.035	0.07
Lawns	0.045	0.03	0.055
Channel			
Natural	0.035	0.025	0.045
Grass	0.03	0.025	0.035
Natural Rock	0.035	0.025	0.045
Armour Stone	0.025	0.017	0.03
Concrete	0.015	0.011	0.017
Articulated Block	0.02	0.019	0.032
Gabions	0.025	0.02	0.03
Wood	0.012	0.011	0.013
Culvert			
Corrugated Steel Pipe (CSP)	0.024	0.021	0.027
Corrugated Plastic Pipe (CPP)	0.013	0.011	0.015

6.4 STEADY FLOW DATA

6.4.1 FLOW VALUES

After inserting all the geometric data into the model, 1D steady flow data were incorporated. Flow values were placed at inlets and flow change locations for 1D steady flow simulation. **Figure 6-9** shows the locations of the Inlet and Junctions for flow simulation and **Table 6-2** shows the simulated flow developed for the 1D Steady Flow Analysis.

Table 6-2: Flow Developed for 1D Steady Flow Analysis

HEC-RAS Points	RS at HEC-RAS Model	2-Year Flow (cms)	5-Year Flow (cms)	25-Year Flow (cms)	100-Year Flow (cms)
I1	259	0.214	0.367	0.637	0.882
I2	308	0.235	0.403	0.701	0.972
I3	383	0.247	0.431	0.757	1.054
I4	790	0.204	0.35	0.605	0.836
J1	420	0.429	0.691	1.136	1.527
J2	2096	0.943	1.437	2.249	2.949
J3	1598	1.832	2.984	4.904	6.632
J4	754	2.921	4.839	8.093	10.533
W1	519	0.431	0.649	0.997	1.291
E1	2250	0.216	0.324	0.499	0.645

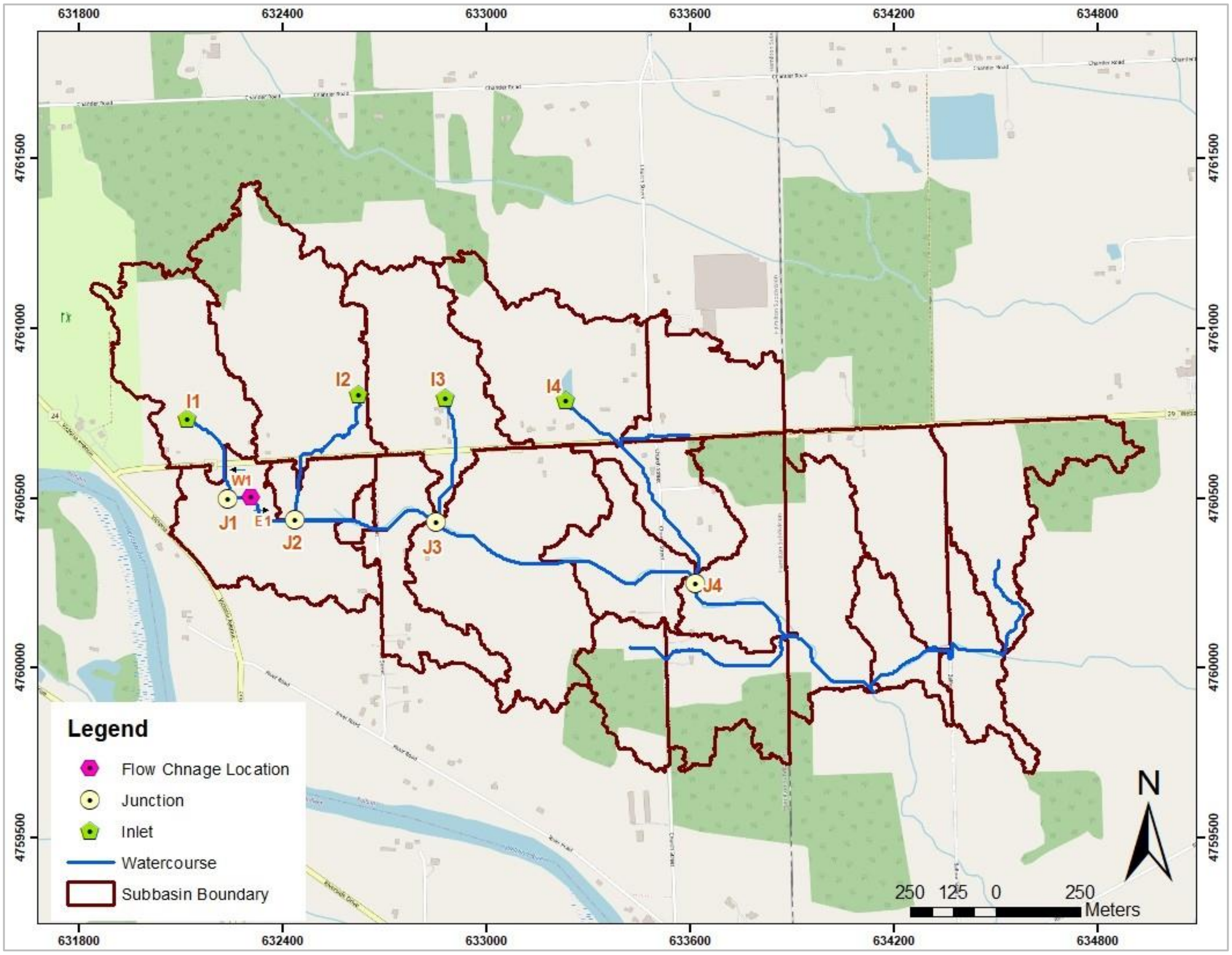


Figure 6-9: Flow Change Locations in HEC-RAS

6.4.2 UPSTREAM BOUNDARY CONDITION

One of the key input parameters of a hydraulic model is the flow at upstream boundaries. The flow of 2, 5, 25 and 100-year return periods were obtained from the event based SWMHYMO model. Normal Depths of the channels were also calculated, and corresponding channel slopes were incorporated into the model. At the junctions, HEC-RAS automatically makes use of flow data (assigned) as upstream boundary conditions.

6.4.3 DOWNSTREAM BOUNDARY CONDITION

It is important that the downstream boundary is far from the project study area to eliminate boundary effect on the hydraulic analysis. When performing hydraulic analysis, Water Surface Elevation is selected as the downstream boundary conditions at the outlets. Since the two outlets of the watercourses are at the Welland River, the known water surface elevation of the river for 100-year return period, 175.06 meters has been used as downstream boundary condition for 100-year return period. For 2-, 5- and 25-year return periods, Normal Depth has been employed as the downstream boundary condition for steady state in this study since the water surface elevation is unknown. Hydrometric gage station was not available within the acceptable limit of the study area. Hence, it was not possible to incorporate any observed or historical flow or water level data into the model for 2, 5 and 25-Year Return periods.

6.5 STEADY FLOW SIMULATION

1D Steady flow simulation has been performed by AHYDTECH to delineate the extent of flooding within the study area and analyze the water surface profiles for different flood scenarios (2, 5, 25 and 100-year) to identify the drainage issues within the study area. Simulations were also performed for different sub-scenarios which were developed for different geometric conditions. The model was run for the existing condition of the study area using RTK data and hydraulic structure data collected during the field survey program.

6.5.1 PROFILE PLOTS

Flow profiles were generated to illustrate the water levels at different river stations of the East and West channels. **Figure 6-10** to **Figure 6-17** show the depth of inundation along the main channels and their associated tributary channels. It has been observed that throughout the West channel the depth increases up to 1.74m for 2-year and 5m for the 100-year flood event near the Welland river over the Victoria Road. The depth is comparatively low at the East channel.

Along the East channel, maximum inundation depth was found to be 1.31m and 1.62m near the Railway Crossing between Church and Balfour Streets, for the 2-year and the 100-year flood conditions.

West Channel

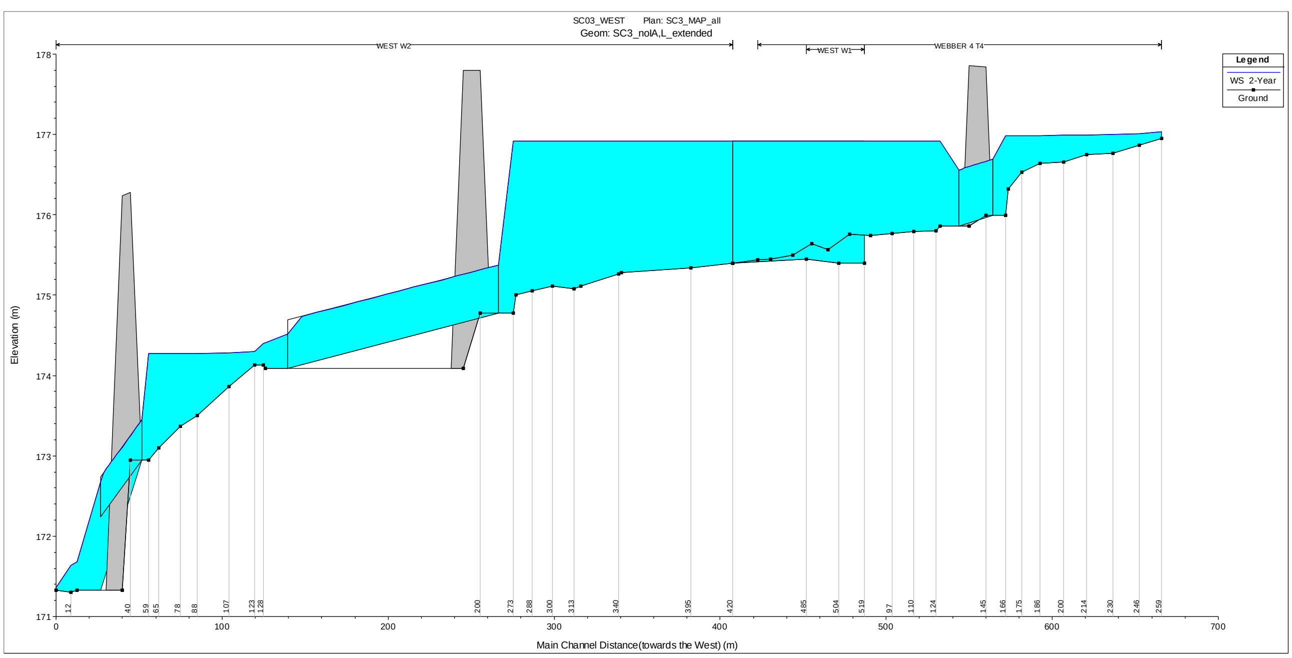


Figure 6-10: Profile Plot generated for West Channel with associated tributary channel (2-Year)

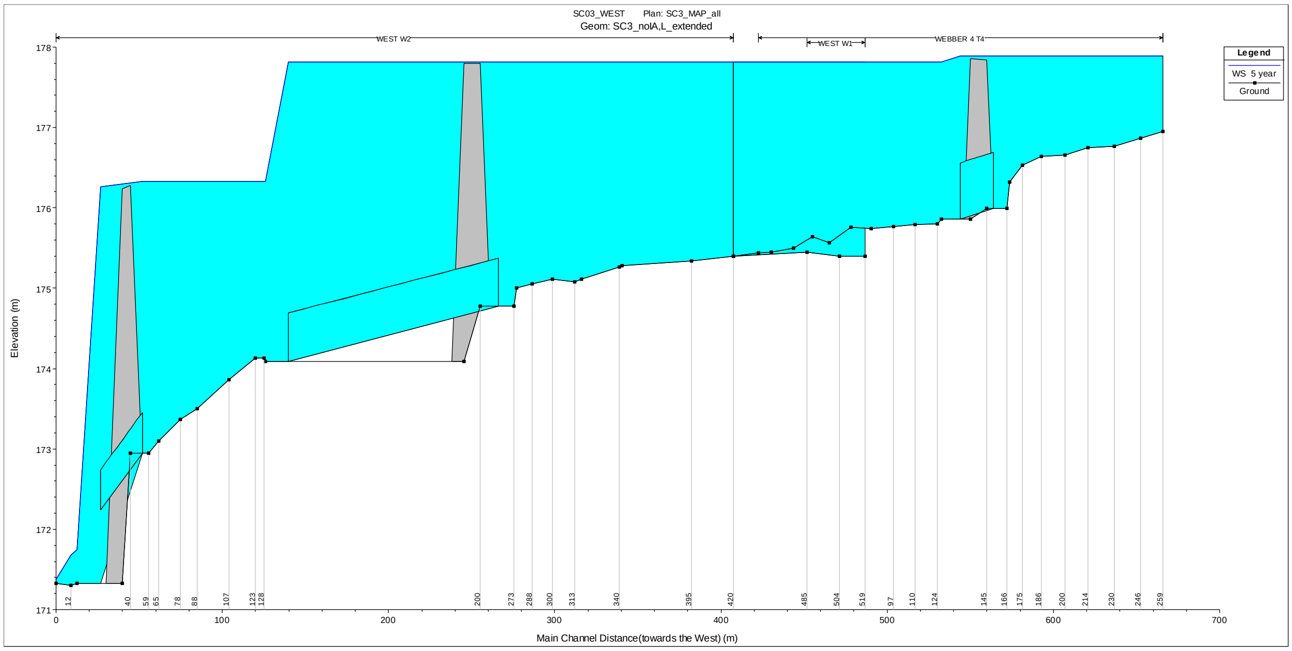


Figure 6-11: Profile Plot generated for West Channel with associated tributary channel (5-Year)

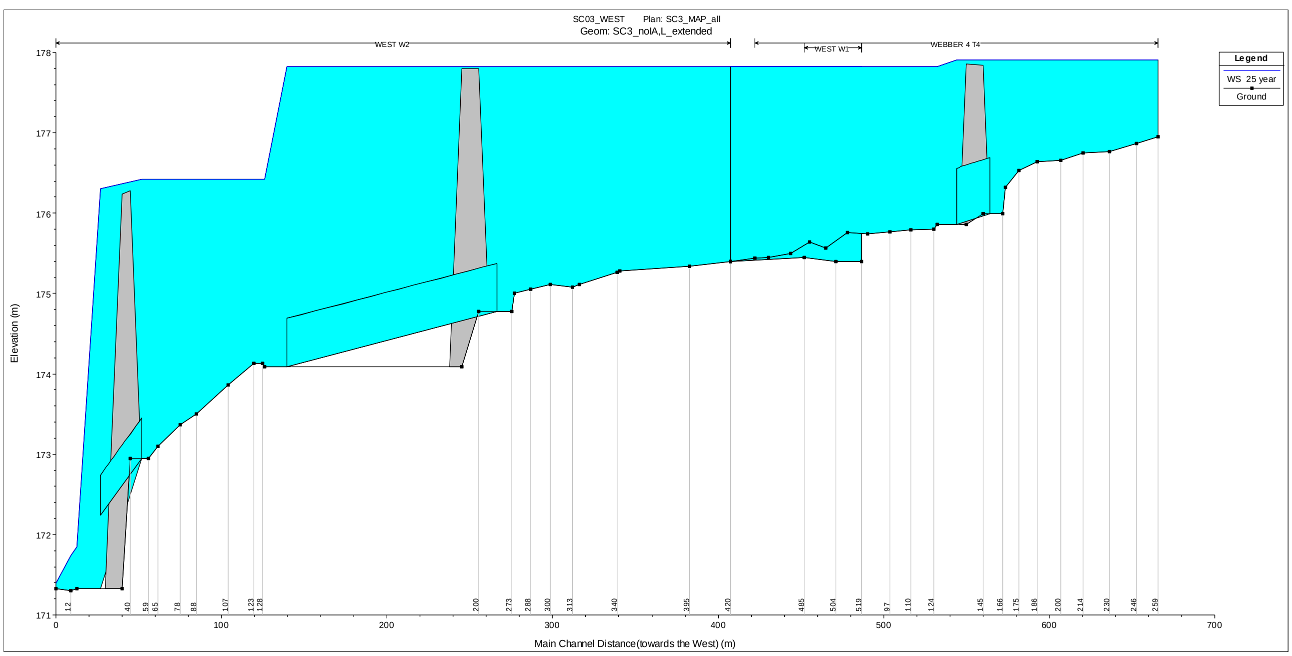


Figure 6-12: Profile Plot generated for West Channel with associated tributary channel (25-Year)

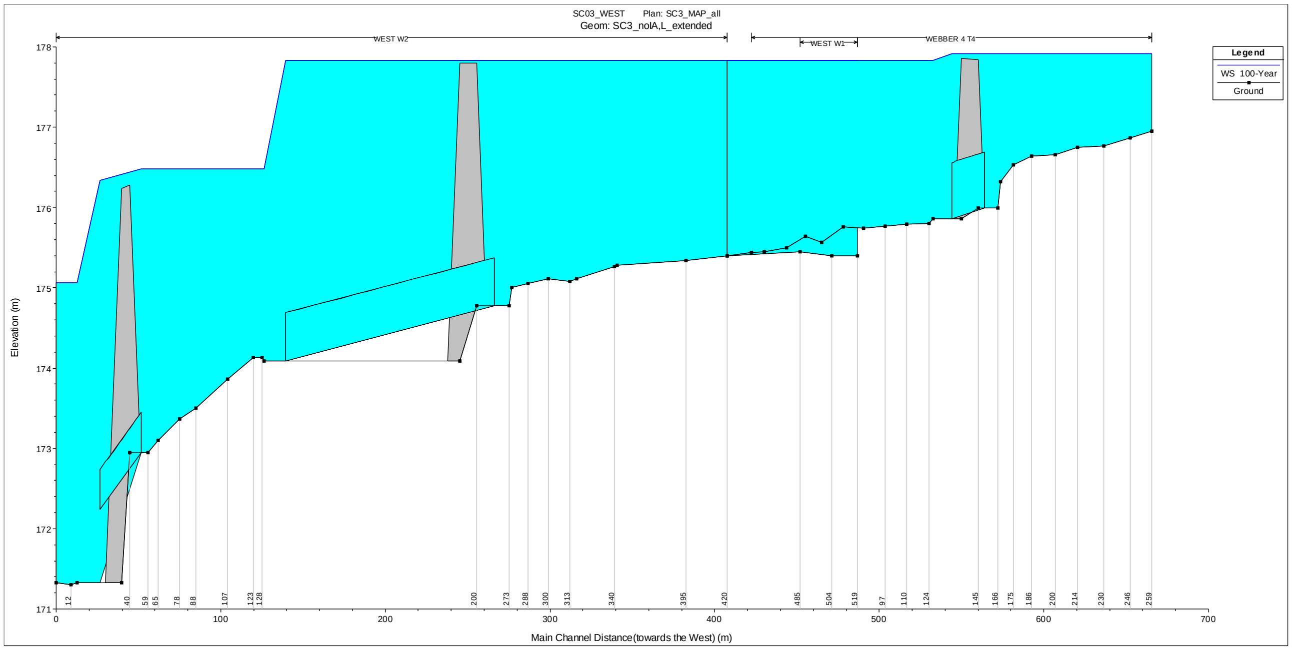


Figure 6-13: Profile Plot generated for West Channel with associated tributary channel (100-Year)

East Channel

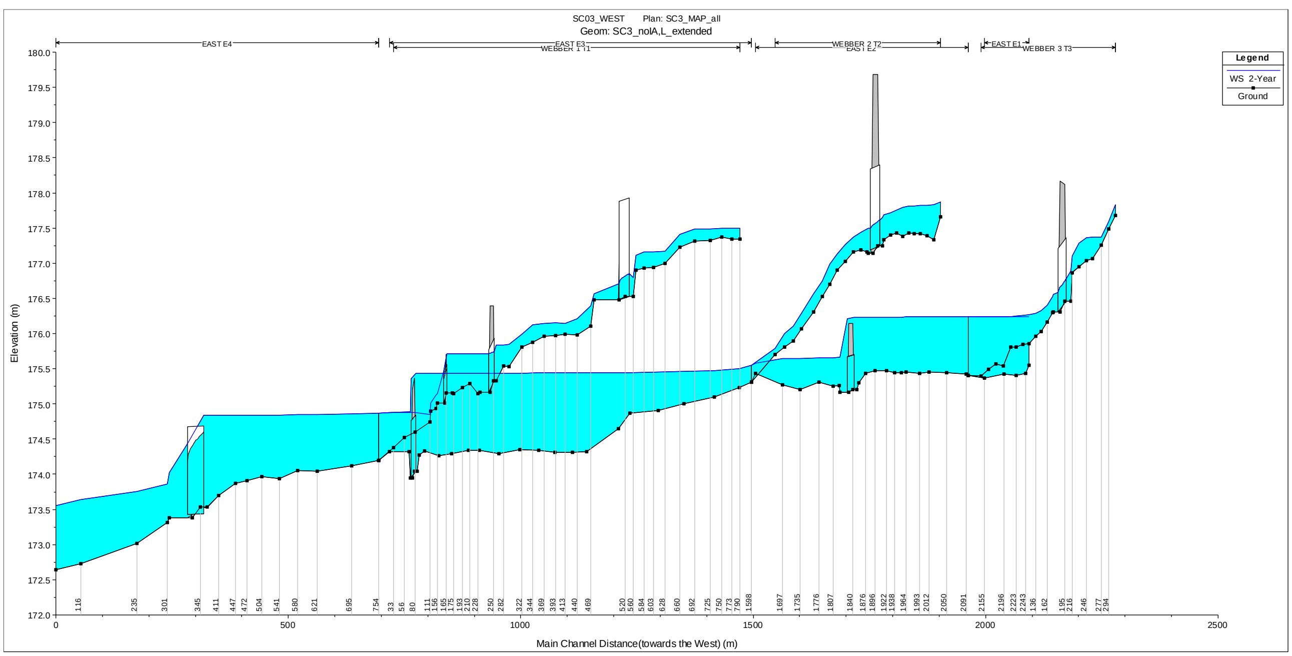


Figure 6-14: Profile Plot generated for East Channel with associated tributary channels (2-Year)

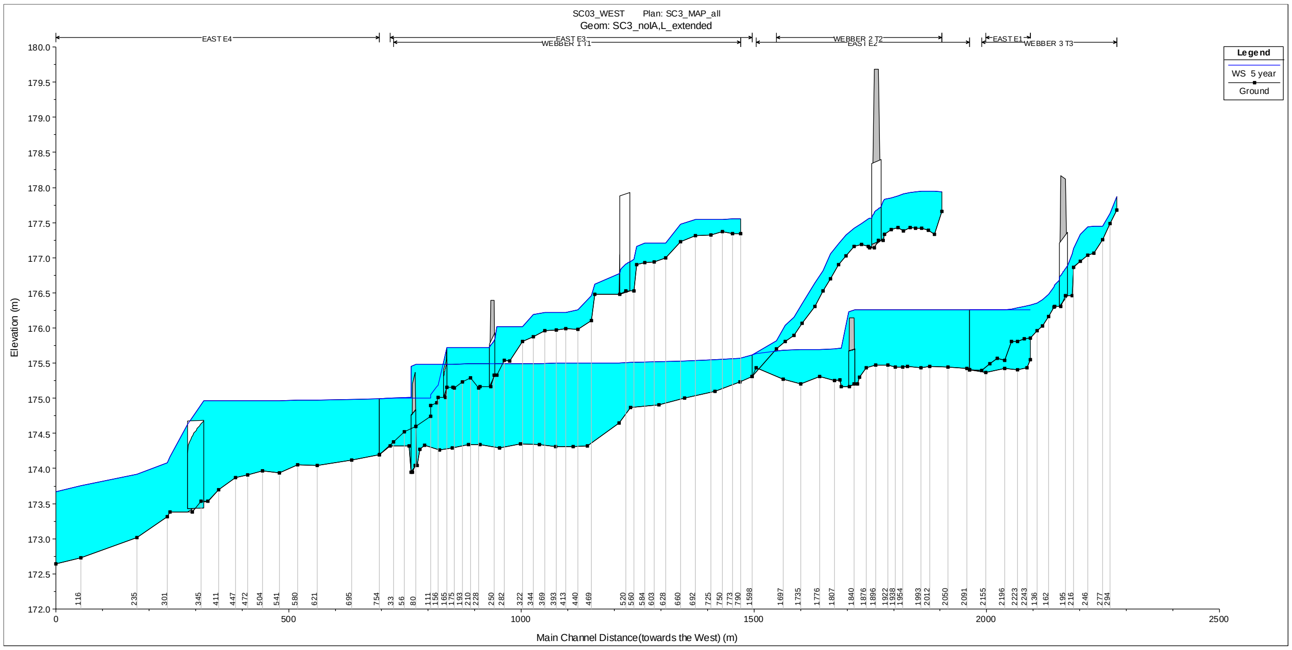


Figure 6-15: Profile Plot generated for East Channel with associated tributary channels (5-Year)

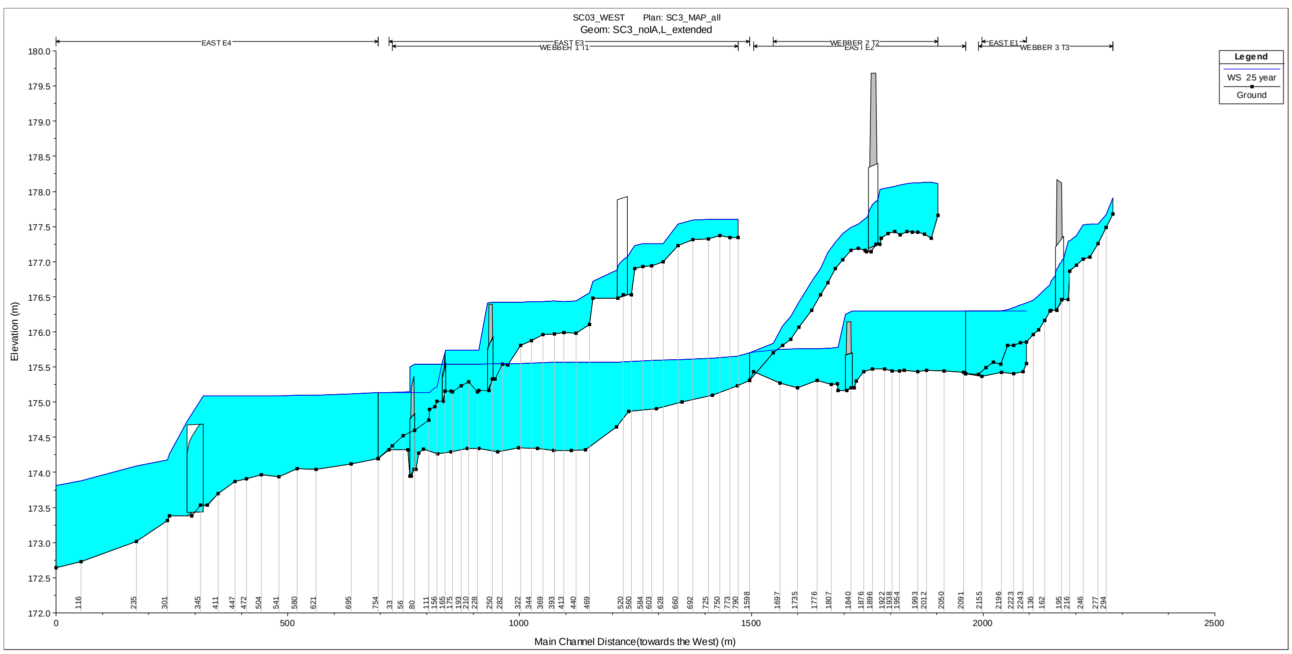


Figure 6-16: Profile Plot generated for East Channel with associated tributary channels (25-Year)

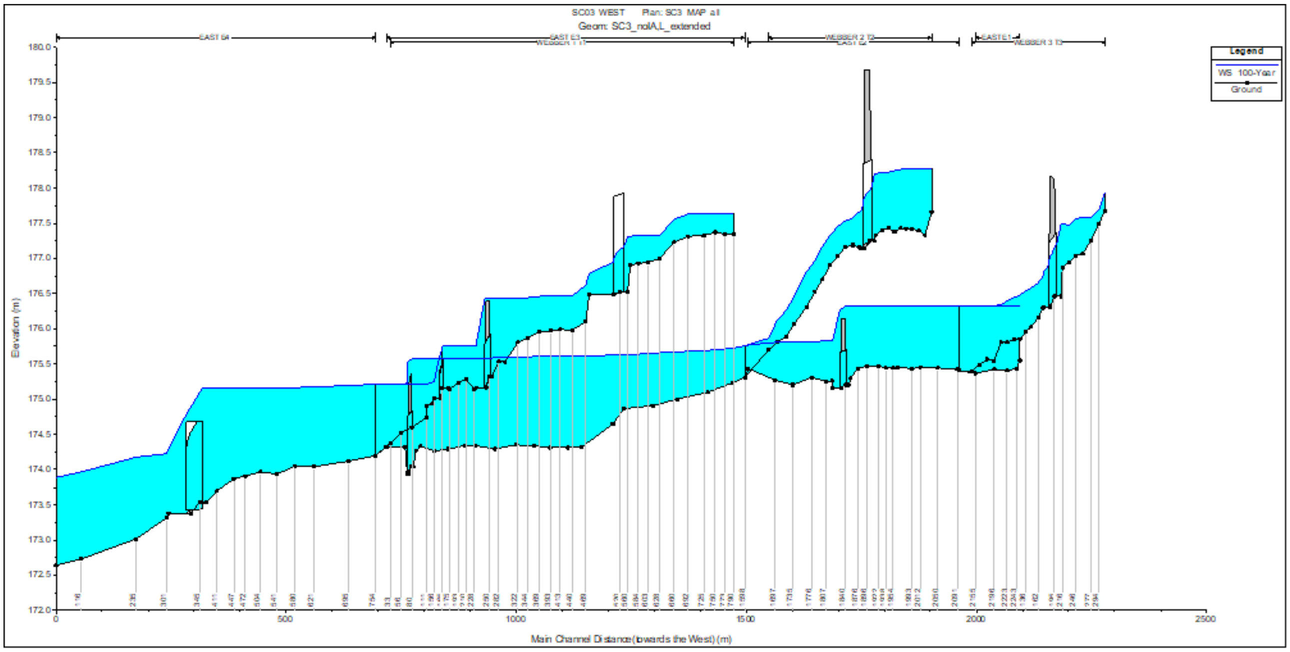


Figure 6-17:Profile Plot generated for East Channel with associated tributary channels (100-Year)

Table 6 3: Inundation Depth with respect to Existing Ground Elevation

Return Period	Maximum Inundation depth (With respect to existing channel bed elevation) West Channel (m)	Maximum Inundation depth (With respect to existing channel bed elevation) East Channel (m)
2-Year	1.74	1.31
5-Year	4.94	1.43
25-Year	4.97	1.56
100-Year	5.00	1.62

6.5.2 ESTIMATED WATER SURFACE PROFILES

The HEC-RAS model was simulated for different flood scenarios. Computed water surface profiles for various flood events with return periods (2-,5-, 25- and 100 year) have been illustrated below from **Figure 6-18** to **Figure 6-21,** for existing condition of the study area.

Analyzing the field condition and survey data, it had been estimated that the direction of flow changes from a certain point located at approximately 350m east of Victoria Road, at the east side of the tributary channel approaching from the most west culvert of the Webber Road. Depending on the change in flow direction, two main channels has been developed where one has been directed towards east and another towards the west in the hydraulic model. Hence, the estimated water surface profiles for different channels, for different flood events and different geometric conditions have been demonstrated for 2, 5, 25 and 100-year return period. Analyzing the results, it has been observed that the maximum inundated water surface elevation is 176.92 meters for 2-year, 177.81 meters for 5-year, 177.82 meters for 25-year and 177.83 for 100-year return period at the channel flowing towards the west. **Table 6-3** shows water maximum and minimum surface elevations found from the hydraulic analysis.

Table 6-3: Water Surface Elevations

Return Period	Maximum WSE (m)	Minimum WSE (m)
2-Year	176.92	171.36
5-Year	177.81	173.70
25-Year	177.82	173.85
100-Year	177.83	173.94

The following **Figure 6-18** to **Figure 6-21** illustrates gradual inundation and water surface elevation change for 2, 5, 25 and 100-year return periods.

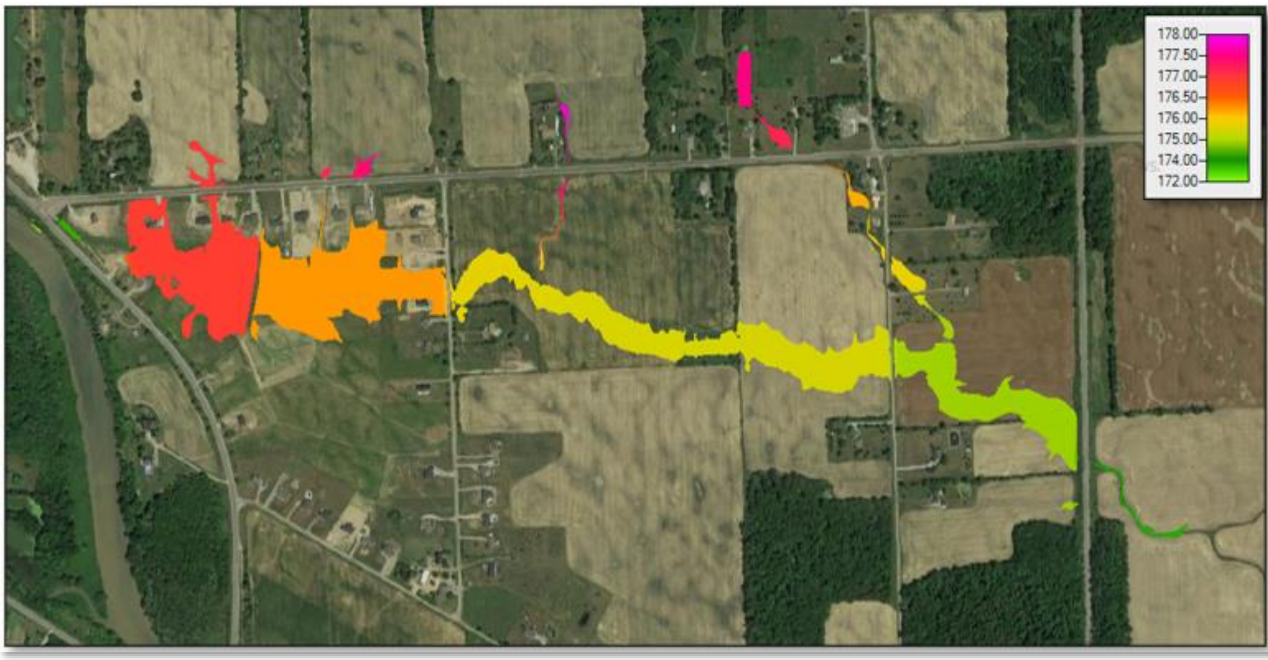


Figure 6-18: Inundated Water Surface Elevation map generated using simulated flow for 2-year return period

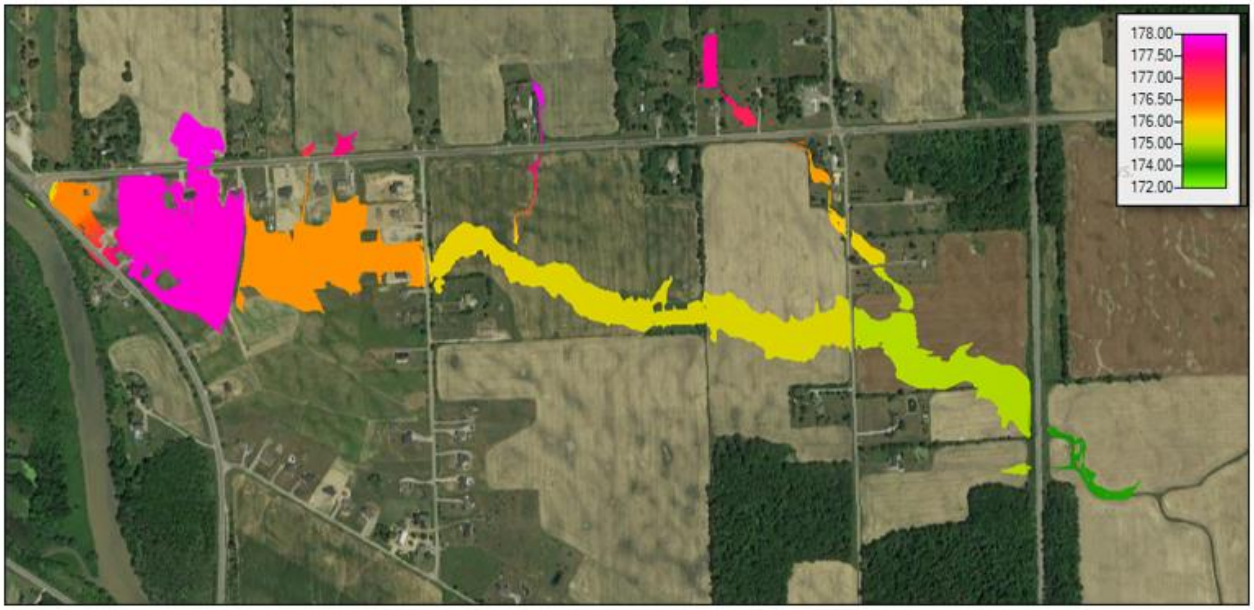


Figure 6-19: Inundated Water Surface Elevation map generated using simulated flow for 5-year return period

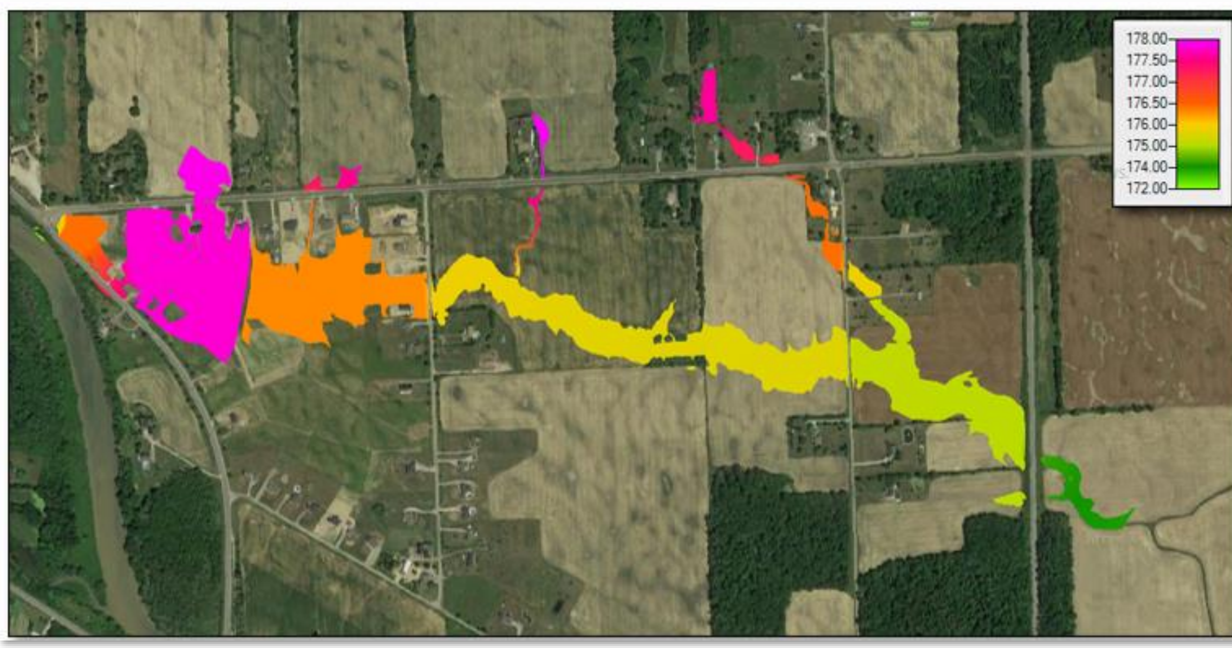


Figure 6-20: Inundated Water Surface Elevation map generated using simulated flow for 25-year return period

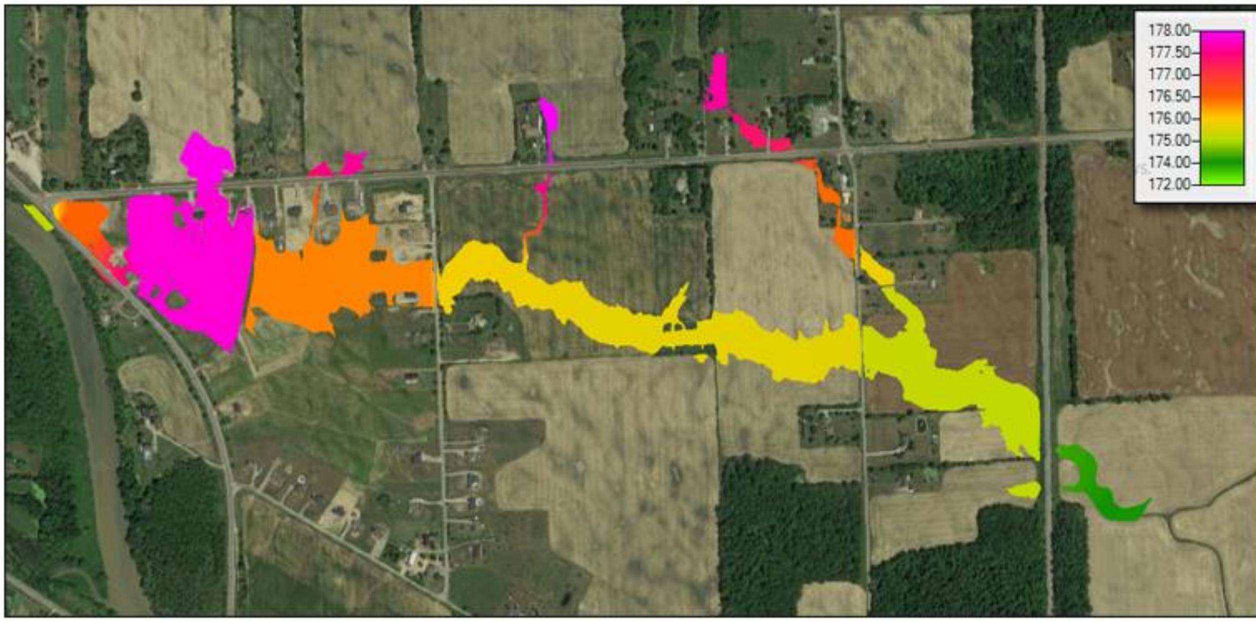


Figure 6-21: Inundated Water Surface Elevation map generated using simulated flow for 100-year return period

6.5.3 DIFFERENCE IN FLOODLINES

Flood lines were computed using RAS Mapper for 2-, 5-, 25- and 100-year flood events. Inundation area for each flood condition have been tabulated below in **Table 6-4**

Table 6-4: Inundation Area and Depth for Different Return Periods

Return Period	Inundated Area (Existing Condition) (sq.km)
2-Year	0.17
5-Year	0.22
25-Year	0.24
100-Year	0.26

6.5.4 BUILDINGS WITHIN THE FLOODPLAIN

AHYDTECH collected Building Footprint from the [ODB (Open Database of Buildings)](). The layer contains information regarding the buildings collected over a few years throughout the Ontario region. AHYDTECH used the footprints to identify the most vulnerable properties that are under risk of flooding. Model results show that the bounding area of Farr, Webber, Victoria, and River Road has the maximum extent of flooding and inundated water surface elevation is also very high at this area compared to the others segment of the study area. **Figure 6-22** demonstrates the inundation caused by 2-Year and 100-Year flood events including the buildings that fall within those flood extents.

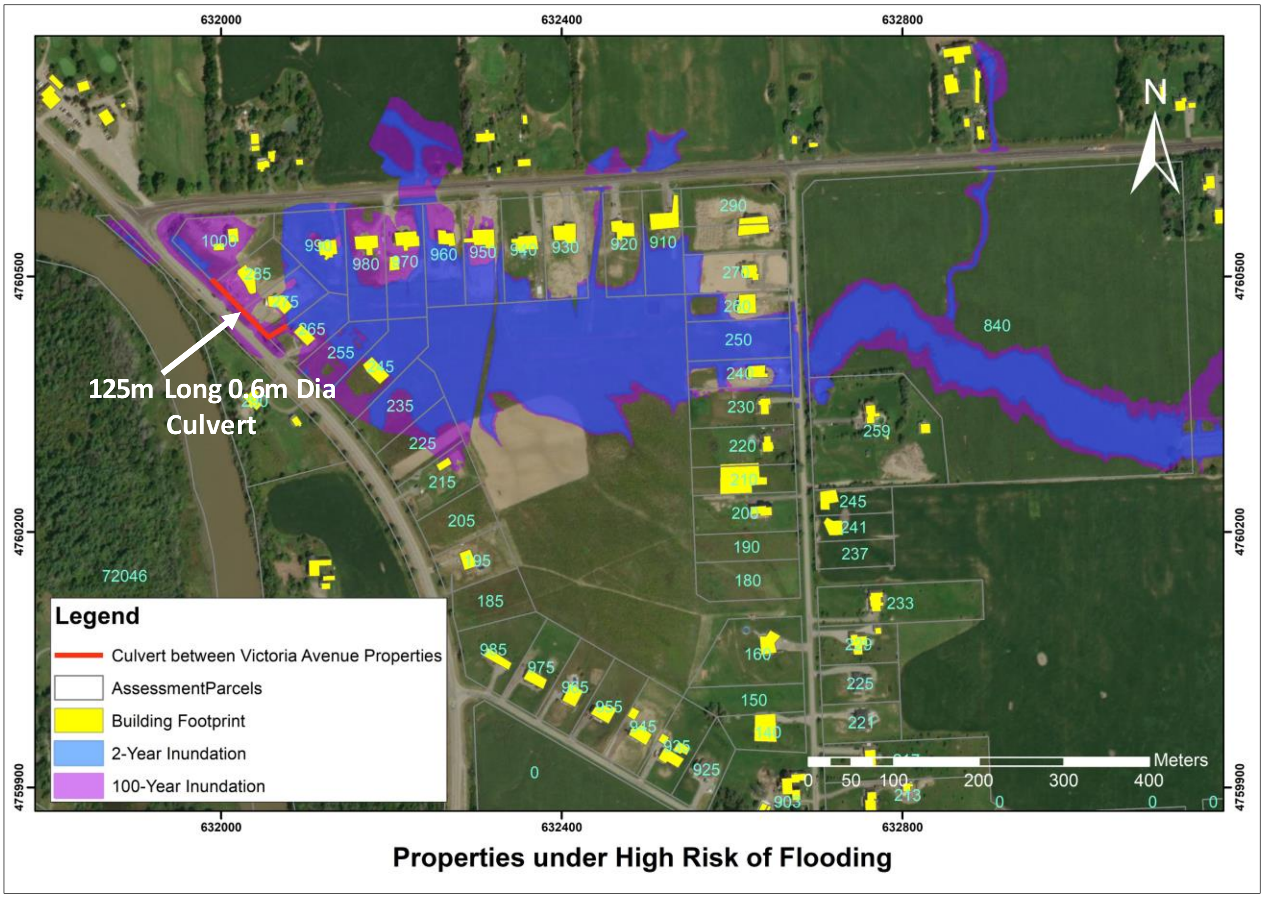


Figure 6-22: Properties within the floodplain near Farr, Webber, and Victoria Road

6.6 HYDRAULIC ANALYSIS AND DRAINAGE AREA MAPPING

AHYDTECH followed applicable technical guidelines to do hydraulic analysis and to produce drainage area mapping products. Simulated inundation map near the Farr, Webber and River Road in the Town of Pelham using HEC-RAS model has been illustrated in **Figure 6-23.** Observing the inundation maps for 2-, 5-, 25- and 100-year flood generated using HEC-RAS model, distinct flooding conditions throughout the study area were noticed. Near the Victoria Road and Webber Road, the extent of inundation appears to be higher, and the residential buildings have been observed to be within the inundation zones. The extent of flooding near the Church Street appears to be moderate. High inundation has also been observed near the Farr Road. At the east side of the Webber Road, there seems to be a minimal degree of inundation. Overall, it can be observed that the 5, 25 and 100-Year flood result causes significant overflow within the study area. In the West channel, there is a 0.6m diameter culvert in between 265 Victoria Avenue and 275 Victoria Avenue properties. This culvert is 125 m long, which runs 35 m in between the two properties and then turns 90 degrees under the ditch along Victoria Avenue the West Channel downstream. This culvert is under sized, which provides backwater effect even for the 2-year flow and flooding in both sides of the watercourse in the West channel.

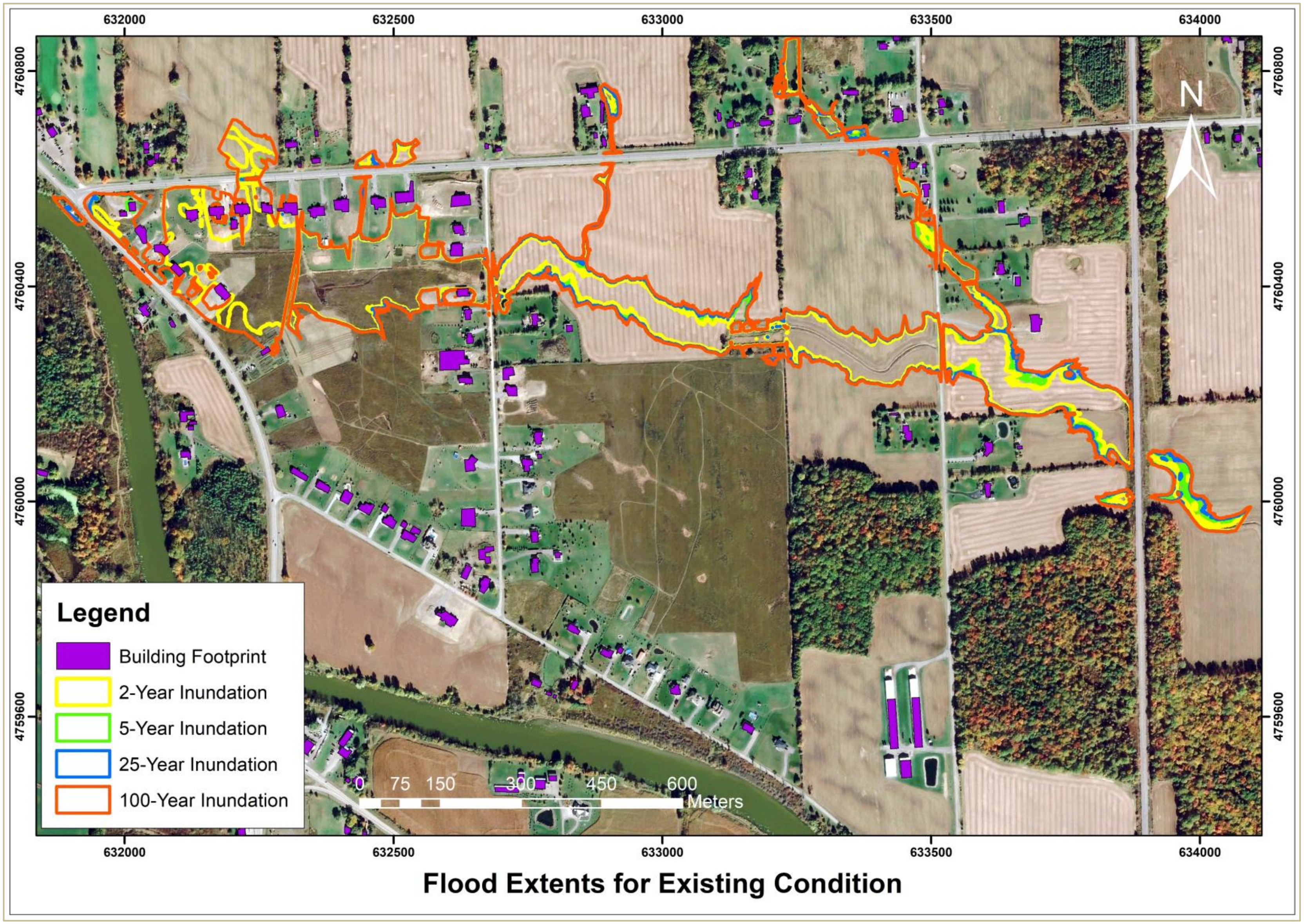


Figure 6-23: Inundation Extent Map for different return periods generated using HEC-RAS

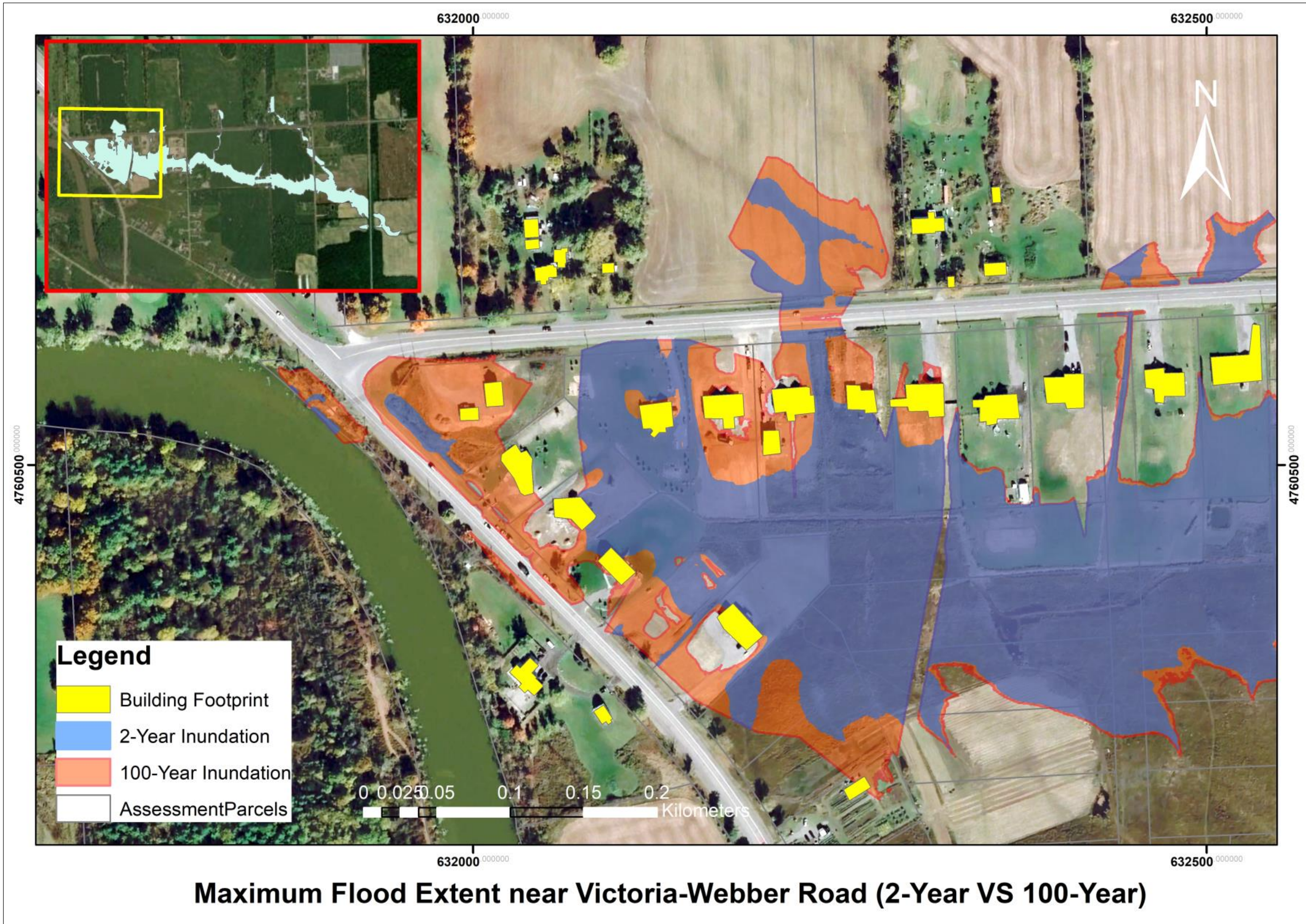


Figure 6-24: Maximum Inundation area delineated using RAS Mapper near Victoria-Webber Road

7 CONCLUSION

An Open House Meeting was held on January 26, 2023, with the presence of stakeholders from the study area, Town, NPCA and AHYDTECH. After analyzing the model results, observations from field assessment, desktop analysis and stakeholder meeting for the study area, it can be stated that the area has extremely poor drainage capacity. Furthermore, the topography of the land is nearly flat; hence, when a rainfall event occurs, stormwater gets entrapped in this area due to poor drainage facilities. Some of the major causes of flooding have been listed below:

- Less capacity of the culverts which causes backwater effects
- Lack of water conveying structures (e.g., Bridges/ Culverts)
- High elevated deck/road with poor drainage facilities
- Alteration of the natural watercourses
- Landfilling
- High and dense vegetation along the channels

AHYDTECH has performed comprehensive analysis on the main drainage issues and findings from the hydrologic and hydraulic analysis has been utilized to find possible measures to mitigate drainage issues based on which a list of recommendations has been offered.

8 RECOMMENDATIONS

Analyzing the results of 1D steady flow simulation and inundation maps, it has been noticed that a considerable number of properties falls within the area of extreme flooding near Victoria Road. The *NPCA Policy Document: Policies for the Administration of Ontario Regulation 155/06 and the Planning Act*, specifies several development setbacks associated with flooding and alteration of watercourses. Following those guidelines before planning and construction a new building or any advancement of land near the floodplain, can eliminate further extension of flood risk within the study area. Primarily assessing the drainage issues, some mitigation options have been discussed below:

- Repair and reconstruction of the existing culverts
- Increased number of barrels within the culverts as well as increased size of the culverts
- Construction of new stormwater conveying structures
- Removal of unwanted plants from the watercourses, inlets, and outlets of the structures

Since, the study area is situated very close to the bank of the Welland River, new and enlarged hydraulic structures and channels can be constructed in future to convey stormwater from the study area to the river. AHYDTECH has conducted detail analysis on the study area and specifically recommended some mitigation alternatives to eliminate flooding near the properties situated close to the intersection of Victoria and Webber Road where the channel meets the Welland River. This portion of the study area experiences frequent inundation, and the area is very prone to flooding for future extreme flood events which has been observed from the hydraulic analysis.

8.1 PROPOSED MITIGATION ALTERNATIVES

According to the aforementioned recommended mitigation options, detail analyses for some alternative options have been performed to assess the proposed specific alternative mitigation options. Assessing the existing model results it has been observed that the bounding area of Farr, Webber, Victoria, and River Road at the west side of project site falls within the high flood risk zone. AHYDTECH assessed three alternative options to eliminate flooding issues which have been discussed below:

8.1.1 ALTERNATIVE OPTION 1

The existing culvert under Victoria Road should be sized for the 50-year storm event to minimize flooding because of backwater under major storm events.

AHYDTECH performed a detailed analysis on water depth and water surface elevation as well as inundation extent increasing the size of the culvert under Victoria Road that conveys water from the west channel to Welland River. Assessing the field survey data, the measured diameter of the culvert at inlet was found to be 0.5m. The culvert size does not allow the water to flow through the culvert and backwater effect is observed. Besides, model results showed that water accumulates at the inlet of the culvert and causes inundation and flow stagnation in the study area. Backwater effect in the surrounding area is also flooding the nearby properties for different flood scenarios.

Different culvert diameter (0.6m, 0.8m and 1m) were used to check and assess the flooding extents. When the size of the culvert has been increased to 1m diameter, no flow stagnation and inundation occurs in the properties which had been flooded previously. The hydraulic analysis shows that the culvert has capacity to convey up to 25-year return period storm events. The culvert cannot convey the 100-year flow, which can cause flooding in the area. Since, the 100-year water surface elevation of Welland River is 175.06m, which is higher than the culvert upstream invert elevation of 173.5m, it is very natural to anticipate that the culvert will be submerged under water for the 100-year storm event for the Welland river watershed. Therefore, 100-Year flood scenario has not been incorporated in the map since it causes same inundation for every scenario.

8.1.2 ALTERNATIVE OPTION 2

An overland flow channel should be constructed passing in between 285 Victoria Ave. and 990 Webber Road properties, along the historical/original channel from the existing channel. The overflow channel will by-pass/away from any septic tank. The channel should be designed for the major event storms. A detailed natural channel design should be prepared for the overland flow channel including for the channel in the study area to provide a positive flow gradient from Farr Road to Victoria Avenue outlet.

Two flow scenarios have been developed which is illustrated below, where F1 and F2 denotes the contributing flows and the area considered for the purpose.

- **F1:** Flow from the north of Webber Road (i.e., flow from two subbasins namely SB01_1 and SB02_1) flows through a channel (or a roadside ditch). It is assumed that the flow coming from these tributaries will discharge directly into the Welland River.
- **F2:** Flow generated by an area bounded by Victoria Avenue, Webber Road, and Farr Street excluding the contributing flows from north of Webber Road (i.e., flow from two subbasins namely SB01_1 and SB02_1). The flow gradient from Farr Road to Victoria Road where the channel has been redesigned with improved conveyance capacity, which will accommodate the flow to pass gently along the channel to Welland River without causing significant flooding.

A new channel has been added with the existing channel at the upstream of Victoria Road where the existing 125m culvert runs perpendicularly from the property 265 to 285 Victoria Avenue. The new channel will run between the properties at 1000 and 285 Victoria Avenue. Flows from F2 scenario has been considered here and is assumed to run entirely towards the west direction. A bifurcation has been created at 200m east of Victoria Road to incorporate the overland flow channel which has been designed to carry flow to the downstream culvert under Victoria Road to minimize flooding/ drainage issues. The new channel meets again the existing channel with 125m culvert at a junction immediate upstream of the Victoria Road culvert. This scenario has been developed based on the fact that the size of the culvert under Victoria Road is 1m. This is because the existing size (currently 0.5m diameter) of the culvert is acting as an obstruction and causing backwater pressure than conveying flow to Welland River caused by 2-, 5-, 25- and 100-Year flood scenarios which has been discussed in **Section 8.1.1**.

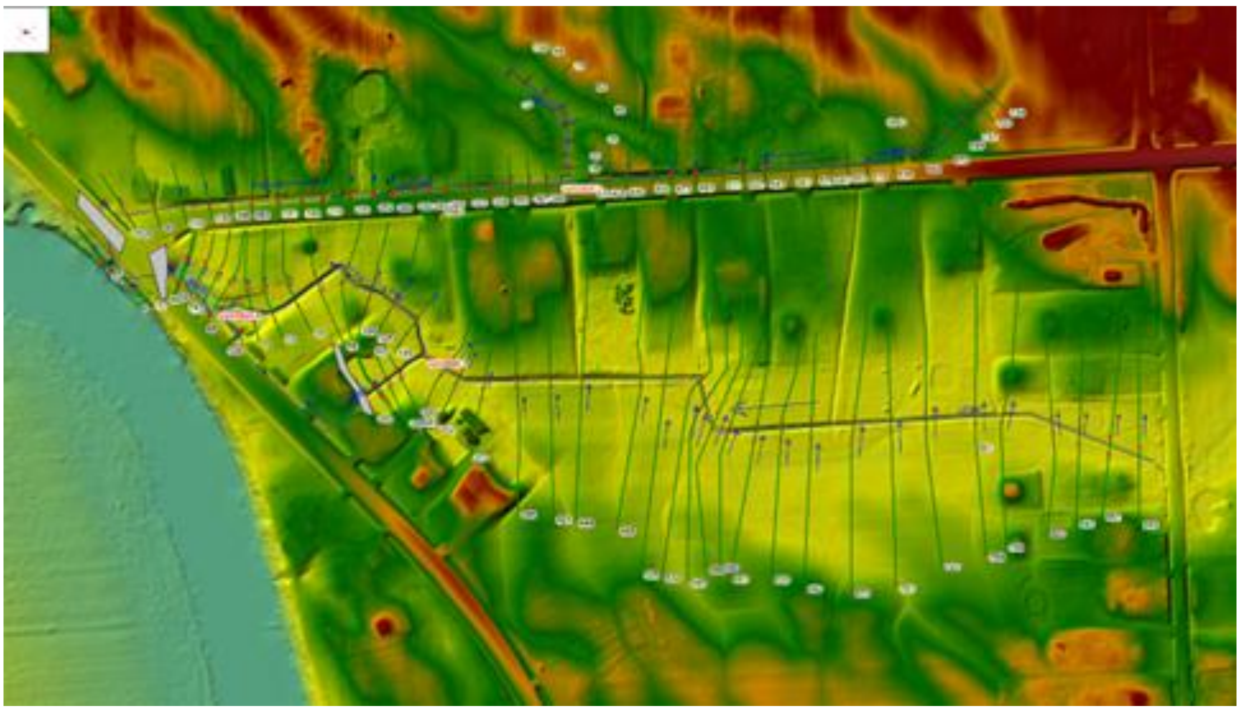


Figure 8-1: Schematic of the Geometry developed in HEC-RAS for the above scenario

The distribution of flow along the two channels incorporated through the bifurcation were estimated based on a trial-and-error process. Two channels, one with the existing 125 m culvert and other which has been designed to convey the flow that exceeds the 125 m culvert capacity. Moreover, it has also been examined whether the channel with culvert has the capacity to convey maximum portion of the generated flow or not. It has been observed that the culvert has the capacity to convey less than 20% of flow generated for the corresponding catchment where the newly generated channel can capacitate 100% of the flow without causing any significant flooding in the surrounding properties. Based on this observation, two different sub-scenarios have been discussed below according to the flow distribution and capacity of those two channels.

It should be noted that, inundation caused by the 100-Year return period has not been incorporated into the scenarios. This is because, the water surface elevation of the Welland River for 100-Year return period was found to be 175.06m, which is higher compared to the existing upstream invert elevation of the culvert under Victoria Road. Therefore, the entire channel faces inundation due to backwater caused by the 100-Year water level of the Welland River.

SCENARIO 1: 20% OF THE FLOW CONVEYED BY THE CHANNEL WITH 125M CULVERT

When the existing portion of the bifurcated channel with the 125m culvert is subjected to convey 20% of the flow obtained in F2 scenario, it has been observed that the culvert causes backwater effect throughout the entire channel up until the Farr Road for 25-Year return period. As a result, the entire channel seems to become flooded, and the depth of water also increases despite having a very minimum flow for the 25-Year return period. For the 2 and 5-Year return period flood scenarios, no backwater effect has been observed. The following **Figure 8-2** illustrates the extents of inundation for the 2, 5, 25 and 50-year return period for this scenario. The severity of flooding increases when more than 20% of the flow is directed to this channel and the entire area gets inundated due to the backwater caused by the 125m culvert.

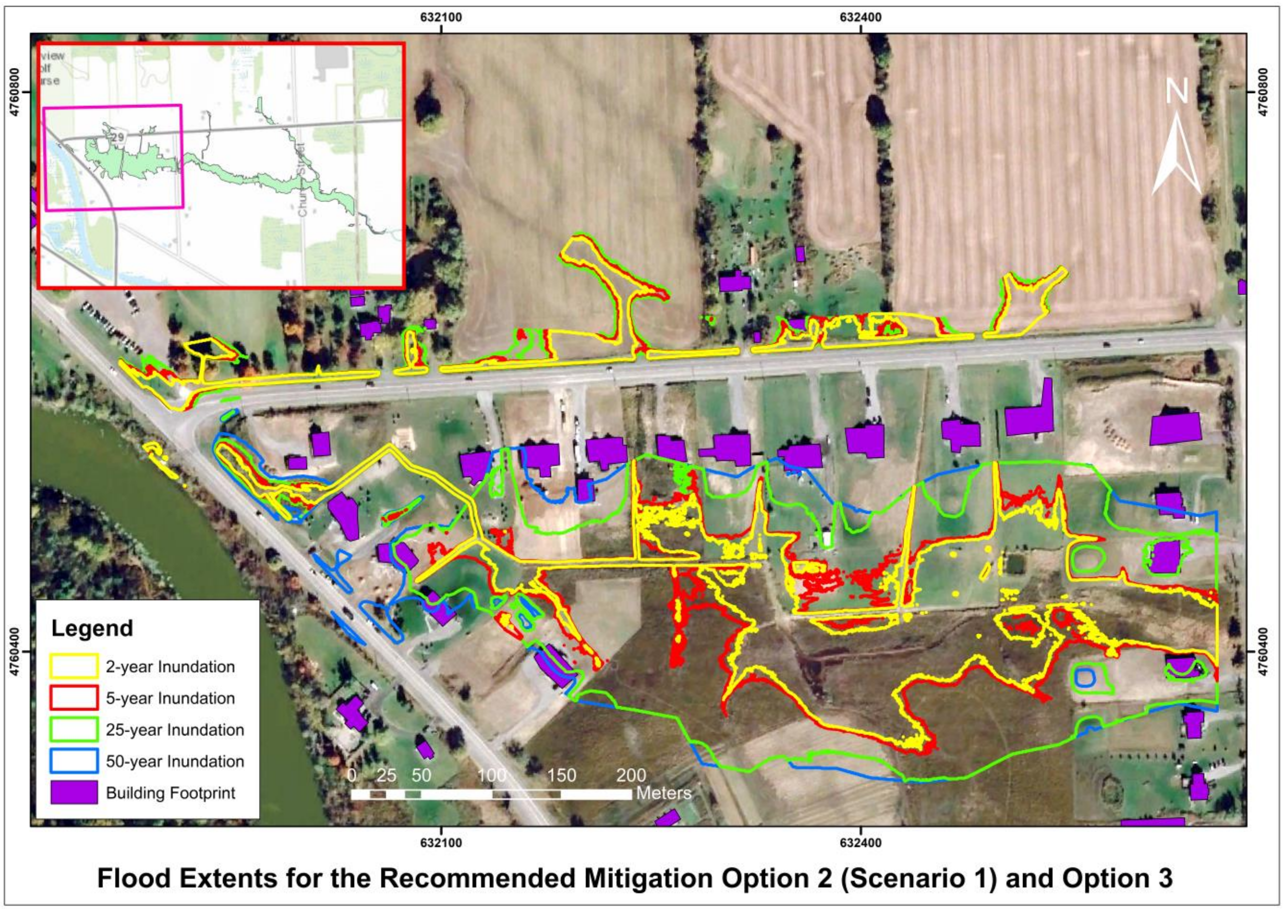
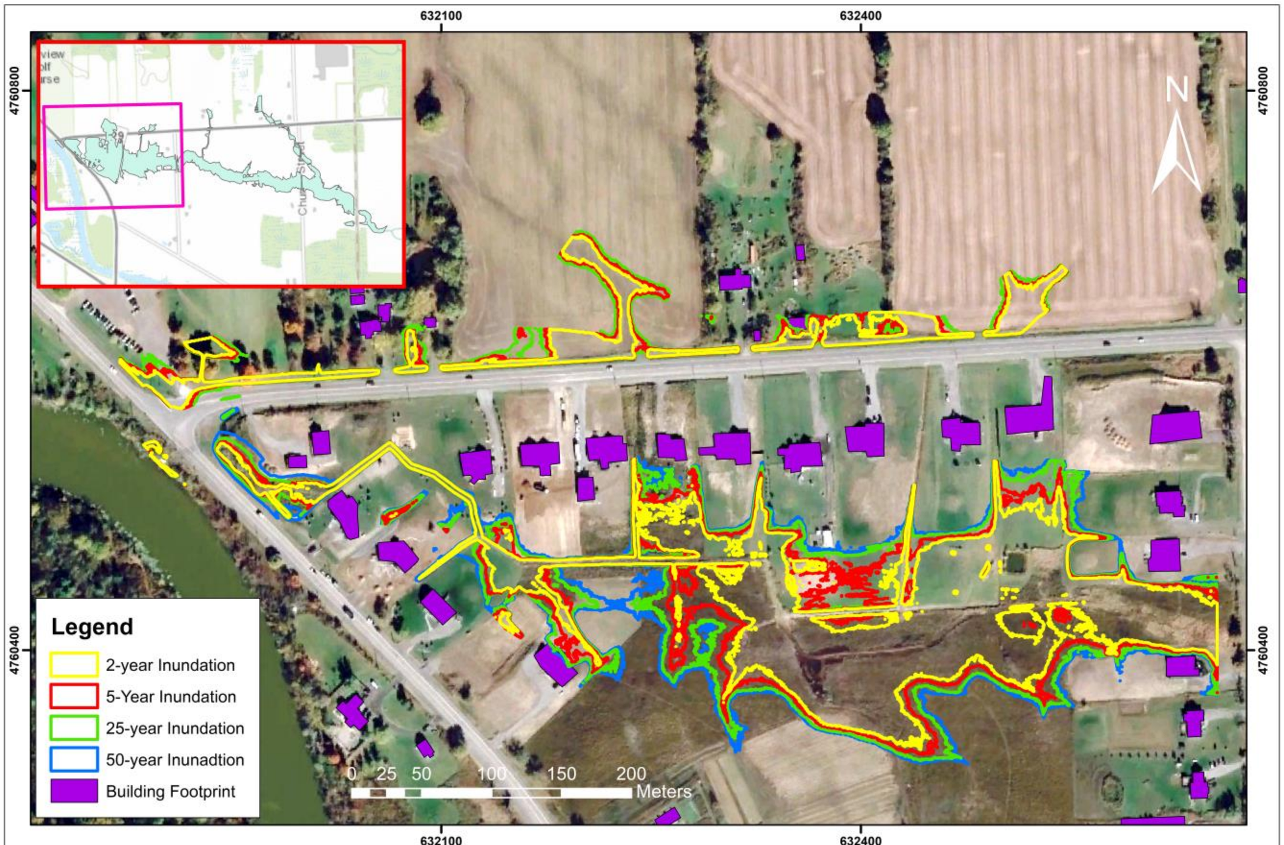


Figure 8-2: Flood Extents for the Recommended Mitigation Option 2(Scenario 1) and Option 3

SCENARIO 2: NO FLOW CONVEYED BY THE CHANNEL OF 125M CULVERT

This scenario considers that no flow is conveyed by the channel with 125 m culvert, and instead the entire flow is directed through the new proposed channel. Through this scenario it has been seen that, no inundation occurs within the area for 2, 5, 25 and 50-year flood scenarios. The new channel which runs between the properties at 1000 and 285 Victoria Avenue has the capacity to convey 100%

of the flow without causing any flooding issues. This indicates that the backwater caused by the 125m culvert has the most contribution while causing inundation upstream, and the 125m culvert significantly increases the depth of water along the inundated channel. **Figure 8-3** illustrates the extent of flooding when 125m culvert does not convey any flow. It can be observed comparing **Figure 8-2** and **Figure 8-3** that there is substantial reduction in inundation extent when the 125m culvert is excluded. Hence, it is recommended either to close the channel passing between 265 to 285 Victoria Avenue and diverting the entire flow to the new channel passing through the 1000 and 285 Victoria Avenue or to increase the diameter of the 125m culvert such that it conveys a considerable portion of flow without causing any backwater effect throughout the channel.



Flood Extents for the Recommended Mitigation Option 2 (Scenario 2) and Option 3

Figure 8-3: Flood Extents for the Recommended Mitigation Option 2(Scenario 2) and Option 3

8.1.3 ALTERNATIVE OPTION 3

At final detailed design, the Region and Town should consider a diversion of some or all of the stormwater from the tributaries that originate north of Webber Road and convey the flows easterly along the Webber Road ditches to Victoria Ave and the Welland River. The design would need to confirm that the ditch system has the capacity to convey the design flow without impacting the existing property owners.

AHYDTECH performed hydraulic analysis for both the Alternative Option 2 (Scenario 1 and Scenario 2) and Alternative Option 3 within a single model. In this alternative option, it has been assumed that two of the tributaries which were originated from the north of Webber Road and conveyed flow to the main channel (**Figure 6-3)** south of Webber Road, will be diverted to Welland River through a roadside ditch north of Webber Road. The main reason of this flow diversion is to reduce flooding in the study area of Farr, Webber, Victoria, and River Road. **Figure 8-2** and **Figure 8-3** demonstrates the diversion of the tributaries north of Webber Road. The figures also demonstrate extents of flooding along the north ditch for different flood events. This alternative option notably eliminates flooding and reduce the extent of inundation within the project study area.

This study would like to forward following recommendations to mitigate the flooding in the project area:

1. Since the recommendations for improved drainage impacts more than one property, the Municipality and the Region should be responsible for future maintenance of the drainage system
2. The existing 125 m long culvert can be kept providing drainage for more frequent events. Proper maintenance of this culvert through visual inspection can confirm the integrity and add a manhole at the turn to facilitate cleaning.
3. The proposed diversion channel needs to be constructed by the Municipality as a capital works project as they permitted earthworks to fill in the original watercourse through the development process.
4. Two options are possible for the municipality to maintain the proposed new channel and the 125 m culvert. One, the municipality would take an easement over the lands for future maintenance of the 125m culvert. Two, the proposed channel and culvert will be incorporated into a drain under the Drainage Act. The property owners need to provide access/easement of the lands. However, if future maintenance is required the costs would be assessed to all upstream landowners.
5. The channel should be kept fence and structure free to permit infrequent storms to pass.

9 LIMITATIONS

Though a detail study has been performed to investigate the main drainage issues for the study area, some limitations were found which affected on the precisions of the model outputs such as:

- **Downstream Boundary Conditions**
  The downstream boundary condition used in the hydraulic model was the slope of the channel for 2, 5 and 25-Year return periods except for the 100-Year return period. Known water surface elevation of the Welland River for 100-Year return period (provided by the Town) has been used as downstream boundary condition. For 2-, 5- and 25-year, normal depth was used because, the Welland River near Farr, Webber and River Road do not have any stream flow gauge stations.
- **Model calibration**
  Both the hydrologic and hydraulic model requires gage station data for the purpose of calibration. Since no gage station have been found within the acceptable limit outside of the study area, it was not possible to calibrate the models. Thus, the hydrologic model developed is an uncalibrated one.
- **2D Hydraulic Model**
  2D hydraulic model could give more precise results regarding the flow diversion location, channel direction at different locations. Even if a flow path changes during an event, 2D hydraulic model can capture and handle flow diversion and flow change location.

10 REFERENCES

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