The Odan/Detech Group Inc.
P: (905) 632-3811
F: (905) 632-3363
5230, SOUTH SERVICE ROAD, UNIT 107
BURLINGTON, ONTARIO, L7L 5K2
www.odandetech.com

MAIN STREET TO KILLALY AND ELIZABETH STREET TO LORAINE STREET, PORT COLBORNE

PROPOSED RESIDENTIAL PLAN OF SUBDIVISION
CITY OF PORT COLBORNE

PROJECT No.: 21247

FUNCTIONAL SERVICING & STORMWATER MANAGEMENT REPORT

Prepared For:

Elite Cap Inds Holdings Inc.,
Elite Cap PC Holdings Inc.,
Elite P.C.M. Holdings Inc.,
Elite P.C.V. Holdings Inc.,
705 Main P.C. Holdings Inc.,
Elite Capital P.C. Developments Inc., and
Elite 869 Killaly Holdings Inc

Prepared By:
The Odan/Detech Group Inc.

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EXECUTIVE SUMMARY

This Functional Servicing and Stormwater Management Report has been prepared in support of an Official Plan Amendment (OPA), Zoning By-law Amendment, and Draft Plan of Subdivision Application for a proposed 158.24 hectare (391 acre) residential and commercial development in Port Colborne. The property is generally bounded by Elizabeth Street (west), Main Street East (north), Lorraine Street (east), and Killaly Street East (south), and is currently vacant agricultural land.

The purpose of the study was to assess the feasibility of servicing the development with municipal sanitary and water infrastructure, identify constraints, and establish preliminary servicing strategies to support the proposed build-out.

Sanitary Servicing

The servicing strategy for the subject site consists of a sanitary pumping station (SPS) within the development lands and a proposed 300mm forcemain which will direct all sanitary flows from the development to the Seaway wastewater treatment plant (WWTP). The preferred SPS location is centrally situated within the development lands (north of the floodplain along the Snider Road allowance). The central location is beneficial for reduced bedrock excavation, shallower sewer and wet well depths, lower overall construction costs, and improved phasing flexibility and ability. The total SPS service area will encompass approximately 126 hectares, including external contributing lands as requested by the City and Region. The projected residential population of the development is approximately 6,289 persons (1,258 single detached units and 745 townhomes). The calculated total peak sanitary flow to the new SPS is 110.75 L/s. The Niagara Region Master Servicing Plan (2023) confirms that the Seaway WWTP has sufficient capacity to accommodate projected growth to 2051, including flows from the subject development.

Water Distribution

The development is located within Pressure District 1 (PD1) of the City's water distribution system. A hydraulic analysis was completed using the Region's INFOWATER Pro model to assess system performance under: Minimum Hour Demand (MHD), Maximum Day Demand (MDD), Peak Hour Demand (PHD), and Peak Hour plus Fire Flow scenarios.

Key findings of the hydraulic analysis include:

- System pressures remain within acceptable municipal criteria (275–690 kPa).
- Pressures exceed the minimum 140 kPa requirement during fire flow events.
- Maximum pipe velocity criteria (≤1.5 m/s) are satisfied.
- Available fire flows are adequate to support the proposed development.
- The system is well-looped and can be expanded through phased development without major off-site upgrades.

Although full build-out results in modest pressure reductions compared to existing conditions, the water system remains within operational standards and capable of supporting approximately 6,300 additional residents by the 2041 planning horizon.

Stormwater Management

The proposed development is located within two distinct systems: the Welland Canal South watershed and the Wignell Drain watershed, which ultimately outlets to Lake Erie. Stormwater management (SWM) analysis has been completed confirming that all SWM criteria (quantity, quality, water balance, erosion) can be met and will not have any negative impact the existing drains.

Six stormwater management ponds (Ponds A–F) are proposed across the development. These facilities are provided to control peak flows to pre-development quantity control targets as well as provide enhanced level 1 quality control protection (80% TSS removal). The provided quantity controls ensure that pre-development peak flows and hydraulic grade line levels are maintained at key downstream control points for all events.

Enhanced stormwater quality control (80% TSS removal) will be provided via a treatment-train approach incorporating: Wet ponds (primary quality treatment), Oil/grit separators at outlets, Low Impact Development (LID) measures.

In accordance with NPCA and MECP criteria, post-development water balance analysis identified an annual infiltration deficit. This deficit will be addressed primarily through infiltration galleries designed in accordance with Ministry of the Environment, Conservation and Parks (MECP) design criteria.

The proposed development has been evaluated using hydrodynamic modelling which confirm that:

- Post-development peak flows match pre-development targets.
- Floodplain limits and hydraulic grade lines remain essentially unchanged.
- Erosion risk is not increased.
- Emergency access and public safety criteria are satisfied.
- Enhanced water quality (80% TSS removal) will be achieved.
- Water balance requirements can be fully addressed through infiltration and LID strategies.
- High lake level scenarios do not materially affect upstream conditions.

Overall Conclusions

Based on the analyses completed, the proposed development is serviceable from both sanitary and water distribution perspectives. From a stormwater management perspective, the development does not increase flooding, erosion potential, or public safety risk. Overall, the development can proceed without negative downstream impacts. In conclusion, from a functional servicing standpoint, there are no infrastructure limitations that would prevent the proposed residential and commercial development from proceeding, subject to detailed design and continued coordination with the City and Niagara Region.

1.0 BACKGROUND

The property under study is a 158.24 ha (391.018 acre) residential development site in the City of Port Colborne. This 158.24 ha area also includes commercial lands owned by the client which are located north of Main Street East. The residential subdivision site is bounded by Elizabeth Street to the west, Main Street to the north, Lorraine Street to the east and Killaly Street East to the south. The site is east of the Welland Canal. The lands are predominately vacant and are currently being utilized as Agricultural Lands. Refer to Figure 1 for the extends of the subject property.

It is proposed to develop the lands for residential and commercial use with related streets and parking.

The purpose of this document is to provide a Functional Servicing and Storm Water Management strategy assessing the serviceability and identifying potential constraints and encumbrances that would limit the sites potential for the intended development based on available information.

For detailed topography of the existing site conditions refer to the topographic survey and for a detailed Conceptual site plan prepared by Armstrong Planning refer to Appendix A.

2.0 SCOPE OF WORK

THE ODAN/DETECH GROUP INC. was retained by the developer to review the Site, collect data, evaluate the Site for the proposed use and present the findings in a Functional Servicing and Storm Water Management Report in support of an OPA/Rezoning Application and Draft Plan of Subdivision Application. The scope of work in brief involves the following:

a) Collecting existing servicing drawings from the CITY/REGION in order to establish availability and feasibility of Site servicing.
b) Meetings/conversations with CITY/REGION Engineers, Niagara Peninsula Conservation Authority and Design Team.
c) Evaluation of the data and presentation of the findings in a FSR and Storm Water Management Report in support of the OPA/Rezoning and Draft Plan of Subdivision Application.

Figure 1 – Location of Development (Planned Development)

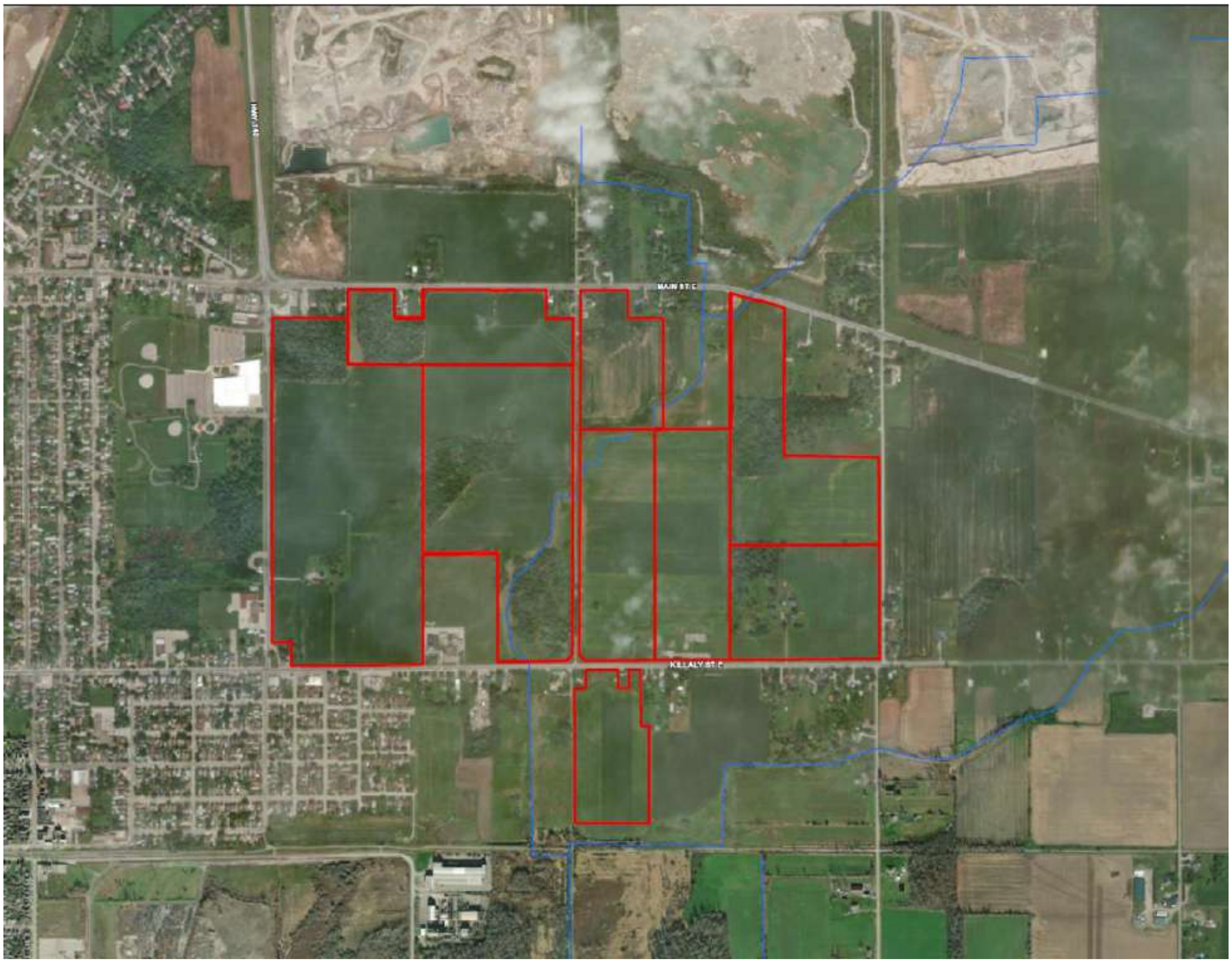


Figure 1 above shows the location of the properties to be developed.

Red lines above represent the proposed development.

3.0 SANITARY SERIVICING

Existing Infrastructure and Background

The proposed development extends from east of Elizabeth Street to Lorraine Street and from south of Main Street to Killaly Street. There are no sanitary sewers within the subject lands. There are existing sanitary sewers on Main Street East, and on Elizibeth Street near the northwest corner of the site. The City has stated that these sanitary sewers may have available capacity for Phase 1 of the development due to recent sewer lining work in the City which has removed significant infiltration flow to the sewer. The City has stated that flow monitoring and modelling results to demonstrate capacity in this sewer will be shared when available.

The existing Fretz Sanitary Pump Station (SPS), located south of the site on Johnston St between James St and Mercury Ave, was considered to be upgraded for portions of the site located near the SPS however due to existing high inflow and infiltration (I&I) contributing to the Fretz SPS, retrofitting the station would be difficult to undertake (see Figure 2 below for location of Fretz SPS relative to the subject site). Our discussions with City of Port Colborne and Niagara Region staff revealed that the Fretz SPS is relatively old and the existing forcemain outlet is near the end of its service life, which doesn't warrant an upgrade. In addition, the forcemain outlets to a gravity sewer that doesn't have the capacity to add additional flow from the subject site.

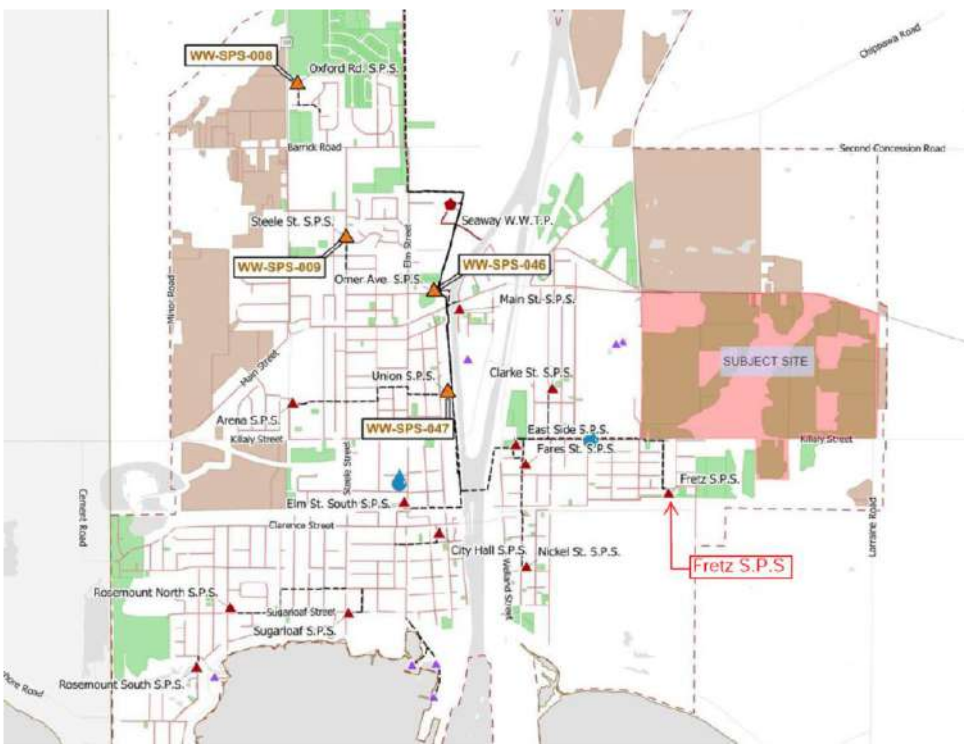


Figure 2 – Location of Existing Pump Station Relative to Subject Site

Existing Capacity at Seaway Treatment Plant

The existing Seaway Wastewater Treatment Plant (WWTP) located at 30 Prosperity Ave services the entire City of Port Colborne and is located north-west of the site on the west side of the Welland Canal. According to the Niagara Region Master Servicing Plan Update dated June 2023, the existing WWTP has the capacity to support growth up to 2051 which includes population from the subject development.

Proposed Sanitary Servicing

Sanitary Pump Station

Due to the absence of existing sanitary servicing to the site, Odan/Detech met with the City of Port Colborne and Niagara Region to discuss feasible sanitary servicing strategies for the site. Niagara Region expressed that the preferred strategy is to propose a new pumping station within the subject lands with a forcemain routed to the Seaway WWTP. The forcemain route is proposed to run north along Elizabeth Street to Second Concession Rd, turn west toward the Welland Canal and cross to Barrick Rd through a new tunnel beneath the canal being proposed, where it will connect south from Barrick Rd to the existing Seaway WWTP. The service tunnel beneath the canal is currently proposed to be constructed in 2026 according to the City and Region. Please refer to Figure S-1 in Appendix B for the proposed layout of the forcemain connection to the Seaway WWTP.

Several factors, namely site phasing, bedrock depth, sewer depth and crossing the existing floodplain within the site, led to determining suitable locations for a new sanitary pump station within the site. Two options were considered.

The first option proposes the SPS at the west end of the site fronting Elizabeth Street, south of the northwest woodlot, which is likely within the limits of the first phase of development. Although this location allows for a reduced forcemain length to the Seaway WWTP, it requires longer gravity sewers to reach the extremities of the areas to be services, thereby requiring deeper sewers and additional bedrock removal.

A second option has been considered which locates the SPS in a central location, north of the floodplain fronting the Snider Rd road allowance running north-south through the centre of the site (see Figure S-2). This location allows for comparable sewer run lengths in multiple directions servicing the required sanitary service boundaries and shallower sewers. This location of the site shows to have a deeper bedrock depth which reduces bedrock removal for both sewer excavation and the SPS wet well compared to the first option.

Comparing both above SPS locations, the second option is the preferred location from a construction cost and phasing standpoint. The reduced bedrock excavation will significantly lower the costs to service the site and being centrally located will allow for greater flexibility to delineate phasing within the site.

In discussion with the City and Region, it was requested that additional land area outside the subject site be included in the drainage area for the proposed SPS. These areas are located between the Snider Rd road allowance and Lorraine St, south of Hwy 3 and also south of Killaly Street East. Please refer to Figure S-3 in Appendix B showing the additional drainage areas to be included in the SPS service area. The total service area for the new SPS is 126 hectares (not including environmental protection areas).

The SPS details will be determined at a later date once the site-specific details regarding population horizons and SPS locations are finalized. Based on the Niagara Region Water-Wastewater Project Design Manual and anticipated flows (see following discussion), the SPS will be a single wet well with a bypass inlet maintenance hole. It will be equipped with one or more pumps with a combined capacity equal to the design flow. The lot size required for the SPS is expected to be a minimum of 25m x 25m (625m^2). A triangular shaped lot providing 2540 m^2 of area for the SPS has been provided which will be adequate.

Total Sanitary Flow Generation

To determine the total sanitary flow that the new SPS will service, the population of the subject site and external drainage boundaries were determined.

For calculating the population of the subject site, the criteria below was used for determining the population of different unit types.

- 2.7 persons/unit for town homes
- 3.4 persons/unit for single detached

Using the above criteria, the following population is derived for the subject site.

Unit Type	Number of Units	People per Unit	Total Population
Single Detached	1,258	3.4	4,277
Townhomes	745	2.7	2,012
		Total Residential Population	**6,289**

For sanitary drainage areas outside the subject development, the number of estimated units is unknown. However, it is assumed that the housing density will be similar to the subject site. To estimate an appropriate people per hectare (pph) value for external areas, the subject site's pph value was determined using the total net developable area of the site (excluding SWM ponds and regulated areas), less the assigned commercial area yielding a total area of 97.18ha. The pph value for the subject site based on the above residential total is 70pph. This population density will be used to estimate the population of external areas contributing to the new SPS within the development.

The criteria used to calculate the total sanitary flow is based on the following Niagara Region parameters.

- Residential flow rate of 255 L/person/day
- 5.0 L/day/m^2 of commercial floor area
- Peaking factor using Harmon formula (between 2 and 4)
- Infiltration – 0.286 L/s/ha (for new developments)

Based on the above parameters, the total peak sanitary flow contributing to the new SPS was calculated to be **110.75 l/s**. It should be noted that this peak flow does not include the future commercial property north of Main Street east. Refer to the following calculation breakdown for further details

SANITARY FLOW CALCULATIONS

This spread sheet calculates the sanitary discharge from various land use as per the Niagara Region Guideline

Total Site Area (ha)	96.38
Total External Area (ha)	29.33

LAND USE	NUMBER OF UNITS	AREA (ha)	GROSS FLOOR AREA, m^2	TOTAL POPULATION	TOTAL DAILY FLOW (LITERS)	AVERAGE DAILY FLOW l/sec	PEAKING FACTOR, M	TOTAL FLOW FROM LAND USE, l/sec
RESIDENTIAL Single Detached 3.40 person/unit	1,258			4,277	1,090,686	12.62		
RESIDENTIAL Townhomes 2.70 person/unit	745			2,012	512,933	5.94		
External Future Residential 70pph		29.33		2,053	523,541	6.06		
Total Residential	2,003			**8,342**	**2,127,159**	**24.62**	**3.03**	**74.66**
COMMERCIAL 5 L/DAY/m^2			2460		12300	0.14		
Total Commercial					**12300**	**0.14**	**1.00**	**0.14**
Infiltration		125.71						**35.95**
Total Flow (l/s)								**110.75**

Q = (M*q*P/86400) + A * I (L/sec)

Q1= total flow from Residential Land Use (L/sec)
Q2= total flow from Commercial Land Use (L/sec)
Qinfil = total flow from infiltration (L/sec)
Qtot = total flow (Q1 + Q2 + Qinfil)

where : P is population
q(res) = 255 L/person/day for residential
A = gross site area
i = 0.286 L/sec/ha (infiltration rate)
Peaking Factor M = 1 + [14 / (4 + (P/1000,1/2))]
(Between 2 and 4)
q(comm)=5.0 L/d /m2 total floor area for Commercial Retail

Forcemain Sizing

The proposed forcemain outletting to the Seaway WWTP shall be designed in accordance with MECP Guidelines and Niagara Region standards which requires the flow velocity to range from 1.0 m/s to 2.0 m/s. The design and construction for a 300mm sanitary forcemain crossing the canal from the Seaway WWTP to Second Concession Road is currently underway. The future forcemain between the subject site and the Second Concession 300mm forcemain will be designed at the detailed design stage when SPS locations and populations are finalized. Maintaining the minimum velocity will ensure that sediment does not accumulate within the forcemain therefore the sanitary flow rates for each phase of development and corresponding pump size will be evaluated during the detailed design phase.

Sanitary Sewer Distribution

Schematic sanitary sewer distribution concepts have been prepared for the subject development for two options discussed above which slightly vary depending on the proposed SPS location. Please refer to Figure S-2 in Appendix B for details of the conceptual layouts for the preferred option.

The main difference between both options is the depth of the sewer in the north-west quadrant of the site which is due to the placement of the SPS as mentioned previously. An additional 3m of sewer depth is required to run the sewer to the SPS located along Elizabeth St as opposed to centrally locating it adjacent to the Snider Rd road alliance north of the floodplain.

It is anticipated that phasing of the site will progress from west to east therefore should the SPS be centrally located it will need to be constructed in isolation from the first phase of development. Temporary access to service the SPS in Option 2 would need to be considered.

To service the areas of the subject site east of the floodplain, a sanitary sewer crossing beneath the floodplain along the Snider Rd road allowance will be required. The method of crossing the flood plan will be determined at the detailed design stage. This crossing is relatively deep (approximately 10m) and does not vary between both schematic sewer options. The need for the significant sewer depth is to be able to reach the sewer to the south-east extremities of the site, including allocated external development areas, while meeting City standard sewer depths at the far reaches of the sewer.

The Region has stated concerns of operational and odour concerns of the SPS in the early phases of the development until the full build-out. During detailed design, options will be assessed to direct phase 1 sanitary flow to the existing gravity sewers. As stated earlier, there are existing sanitary sewers on Main Street East, and on Elizibeth Street near the northwest corner of the site which may have available capacity for Phase 1 of the development. The City has stated that flow monitoring and modelling results to demonstrate capacity in this sewer will be shared when available.

Due to the size of the development and substantial rock removal that will be required, it is expected that main sewer runs to the furthest points of the development leading from the SPS will be relatively low percentage slopes to minimize the depth of the SPS and rock removal along the sewers. However please note that in the final design stage of the sewers, MECP and City standards will be followed to ensure adequate pipe sizes and slopes are proposed to provide self-cleansing velocities within the sewer.

4.0 WATER DISTRIBUTION

Existing Infrastructure and Background

The proposed development is located in PD1 (only pressure district) of the City of Port Colborne water distribution network and will be serviced by connecting to and extending pipes from the existing network through the proposed development. To support the proposed developments, this report presents and discusses the results of a hydraulic investigation and hydraulic modelling analysis of the existing City water model. The analysis assesses whether the existing water infrastructure has adequate capacity to support the proposed development, and what system expansions and upgrades (if necessary) are required. It should be noted that the modelling was completed based on the prior site layout and statistics. The current overall servicing configuration aligns with the previously modelled layout and maintains connections to the same existing external watermains. Also, populations have now decreased and are calculated conservatively as discussed later in this section. Therefore, the modelling results are conservative for the OPA stage and will be refined at the detailed design stage.

The Region of Niagara created a INFOWATER Pro (715023 - Port Colborne Water Model - MSP.aprx) distribution model which included the entire City. Odan/Detech procured a copy of the Model from the Region of Niagara.

We are assuming the following about the City water model:

- City water system model data, including mains (pipes), junctions, valves, chambers, hydrants and pressure district boundaries represents the existing conditions in-situ.
- Model is calibrated and represents the pressures and flows as they are now with existing demands.
- The Model can be relied upon for predicting future conditions.
- Development details for the Development (e.g., unit counts, commercial area breakdown, development statistics, etc.) as set out in the Concept Plans prepared by Armstrong Planning.

In addition, we are relying on the following:

- **DESIGN GUIDELINES FOR DRINKING WATER SYSTEMS**, by the Ministry of the Environment of Ontario (now The Ministry of the Environment, Conservation, and Parks; MECP), dated 2008.

- **NIAGARA REGION WATER-WASTEWATER PROJECT DESIGN MANUAL, Revision** 3, July 2023.

- **THE CITY OF WELLAND MUNICIPAL STANDARDS DESIGN CRITERIA**, February 2013.

The analysis presented in this report will include hydraulic simulations of the

- Minimum Hour (MHD),
- Maximum Day (MDD),
- Peak Hour plus Fire (MDD + FF) and the (Niagara Region Standard)
- Peak Hour (PHD) demands.

The above will be performed for the existing conditions and the Development added. The two will be compared. The following Table 1 is a summary of the scenarios run and the hydraulic assumptions made.

Table 1. Summary of Scenarios run and assumptions made

Development Condition	Demand	ADD Multiplier	Barrick Road ET Water Level	Number of Pumps in Service	Fire Flow
Pre-Development	2041 Peak Hour	2.8	7.8 m (60%)	2	-
Post-Development	2041 Peak Hour	2.8	7.8 m (60%)	2	-
Pre-Development	2041 Peak Hour + Fire	2.8	7.8 m (60%)	2	Fire demand of 318 L/s at node 33001115 as per Region's hydraulic model
Post-Development	2041 Peak Hour + Fire	2.8	7.8 m (60%)	2	Fire demand of 318 L/s at node 33001115 as per Region's hydraulic model
Post-Development	2041 Peak Hour	2.8	7.8 m (60%)	2	Available fire flow assessed for subject development subject to maintaining 140 kPa
Pre-Development	2041 Minimum Hour	0.16	12.4 m (96%)	1	-
Post-Development	2041 Minimum Hour	0.16	12.4 m (96%)	1	-

It should be noted that the Development will take the City to the 2041 timeline, which will add approximately 6300 persons. The following Table 2 are the calculated water demands for the subject site.

Table 2. Development Demand Calculations

TYPE	DESCRIPTION OF DEVELOPMENT	NUMBER OF UNITS	ICI (m2)	POPULATION	Average Day (RESIDENTIAL + ICI) (L/sec)	Peak Day (L/sec)	Peak Hour (L/sec)	Fire flow required (L/sec)	Total Flow required (Fire + max day) (L/sec)
RESIDENTIAL	SINGLE FAMILY	1258	0	4277	15.84	28.20	44.36	75	103.2
RESIDENTIAL	TOWNHOMES	745	0	2012	7.45	13.26	20.86	120	133.3
					23.29				
COMMERCIAL	COM-1		2460	0	0.23	0.41	0.64	200	200.4
					0.23				
TOTALS		**2003**	**2460**	**6289**	**23.52**	**41.86**	**65.85**	-	-
PEAK DAY FACTOR		1.78							
PEAK HOUR FACTOR		2.8							
SINGLE FAMILY		3.4	PPU						
TOWNHOME UNITS		2.7	PPU						
ICI DEMAND IS		8	L/Day/m2						
AVERAGE DAY		320	L/CAP/DAY						

The above includes the subject property. We do not have any knowledge of density and style of development for the other infill properties not owned by the client. The infill properties not owned by the client will be included. The following standards will be used to make the analysis conservative thus including the infill properties in the analysis. The standards used above were from the City of Welland. The Welland standards were utilized to be conservative at the OPA level due to any potential changes to unit counts and populations. The domestic demand calculations will be refined at detailed design using the Region's per capita flow rate. Note the actual peak day and Peak hour multipliers used in the model were 2.8 for both. This is conservative. The City of Welland uses 320 L/cap/day. The Region of Niagara uses 255 L/cap/day.

The Welland demand is (320 - 255 ÷ 255) = 25% more demand. Coupled with the Peak Day being 2.8 which makes the analysis very conservative.

Refer to the Concept Site Plan by Armstrong Planning in Appendix A for details of the development and site statistics.

Models

The Region of Niagara created a INFOWATER Pro (715023 - Port Colborne Water Model - MSP.aprx) distribution model which included the entire City. As mentioned above this model will be used as a base. The addition of the Development will be added to the base model. The scenarios as mentioned in Table 1 above will be run and results obtained and compared.

For detailed model input/output refer to the digital INFOWATER Pro and info in appendix C for the following:

- Model Output – Nodes
- Model Output – Pipes
- Model Output – Fire Flows
- Pressure Distribution – Data

Refer to Figure 3 for the Region of Niagara Water Model (existing) and to Figure 4 for Region of Niagara Water Model (Site added).

Results

In addition to the above detailed output the following Results will be shown:

Figure 5 Pre-Development with 2041 Peak Hour Demand
Figure 6 Post-Development with 2041 Peak Hour Demand
Figure 7 Pre-Development with 2041 Peak Hour Demand + Fire
Figure 8 Post-Development with 2041 Peak Hour Demand + Fire
Figure 9 Post-Development Available Fire Flow with 2041 PH Demand (detail of Site)
Figure 10 Pre-Development with 2041 Minimum Hour Demand
Figure 11 Post-Development with 2041 Minimum Hour Demand

Note, all but Figure 9 are nodal pressure and pipe velocity plots.

Figure 9 shows the pipe sizes within the development. Note, only the major distribution pipes were modelled (200mm, 250mm and 300mm). The numbers in the box near the nodes represent the available fire flows in L/sec.

Figure 12 represents in graphical format the predeveloped and post developed sites.

Table 3 below summarizes the range of pressure operations under the various demands.

Table 3. Developments demand calculations

	Pre-Development 2041 PHD	Post-Development 2041 PHD	Pre-Development 2041 PHD+Fire	Post-Development 2041 PHD+Fire	Pre-Development 2041 MHD	Post-Development 2041 MHD
System Pressure (Entire System)						
Max (kPa)	427	421	396	395	477	477
Min (kPa)	158	151	128	116	333	332
System Pressure (Proposed Development)						
Max (kPa)	-	379	-	327	-	440
Min (kPa)	-	330	-	278	-	391

Figure 3 – Region of Niagara Water Model (existing)

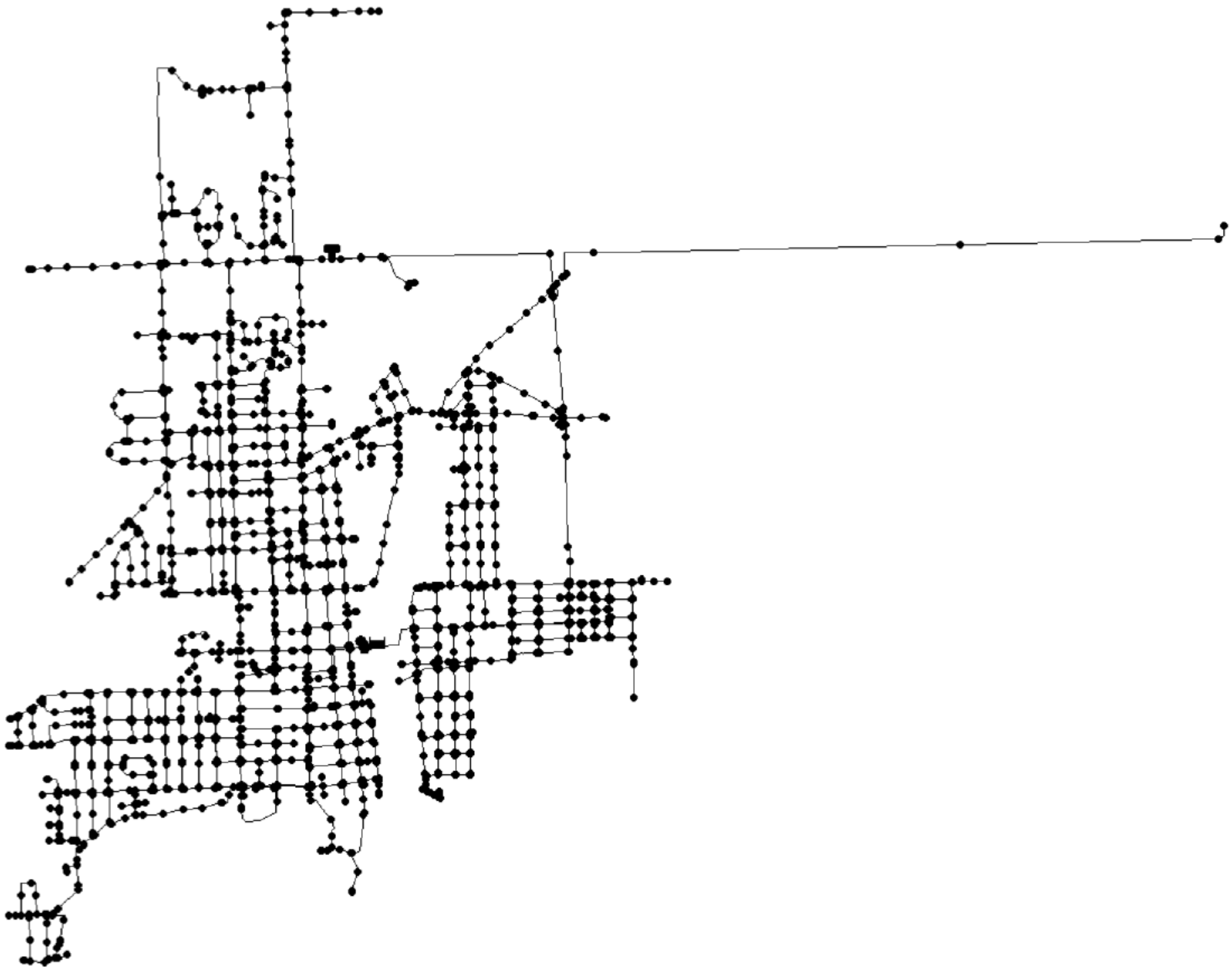


Figure 4 – Region of Niagara Water Model (Site Added)

Figure 5 – Pre-Development with 2041 Peak Hour Demand

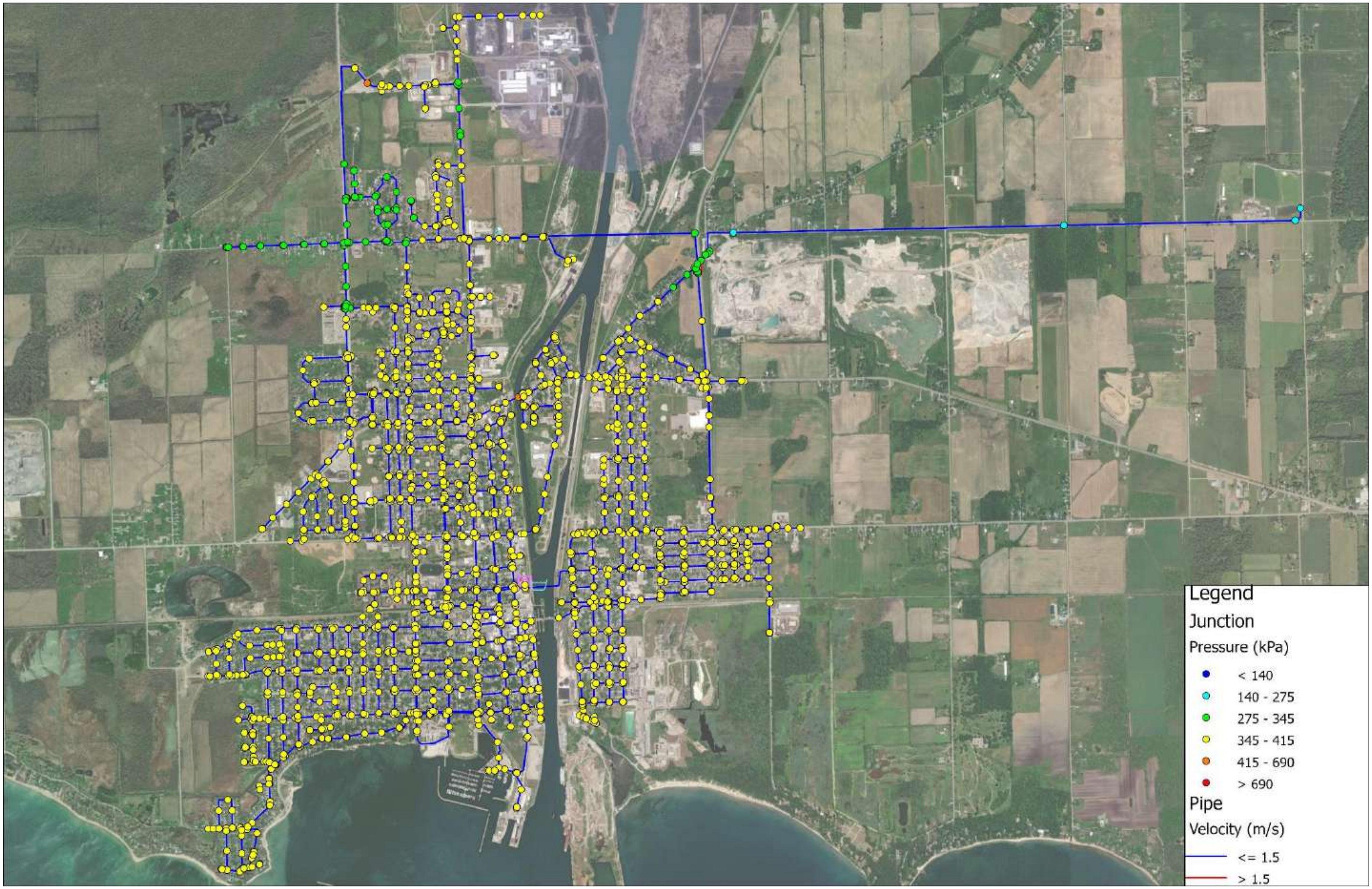


Figure 6 – Post-Development with 2041 Peak Hour Demand

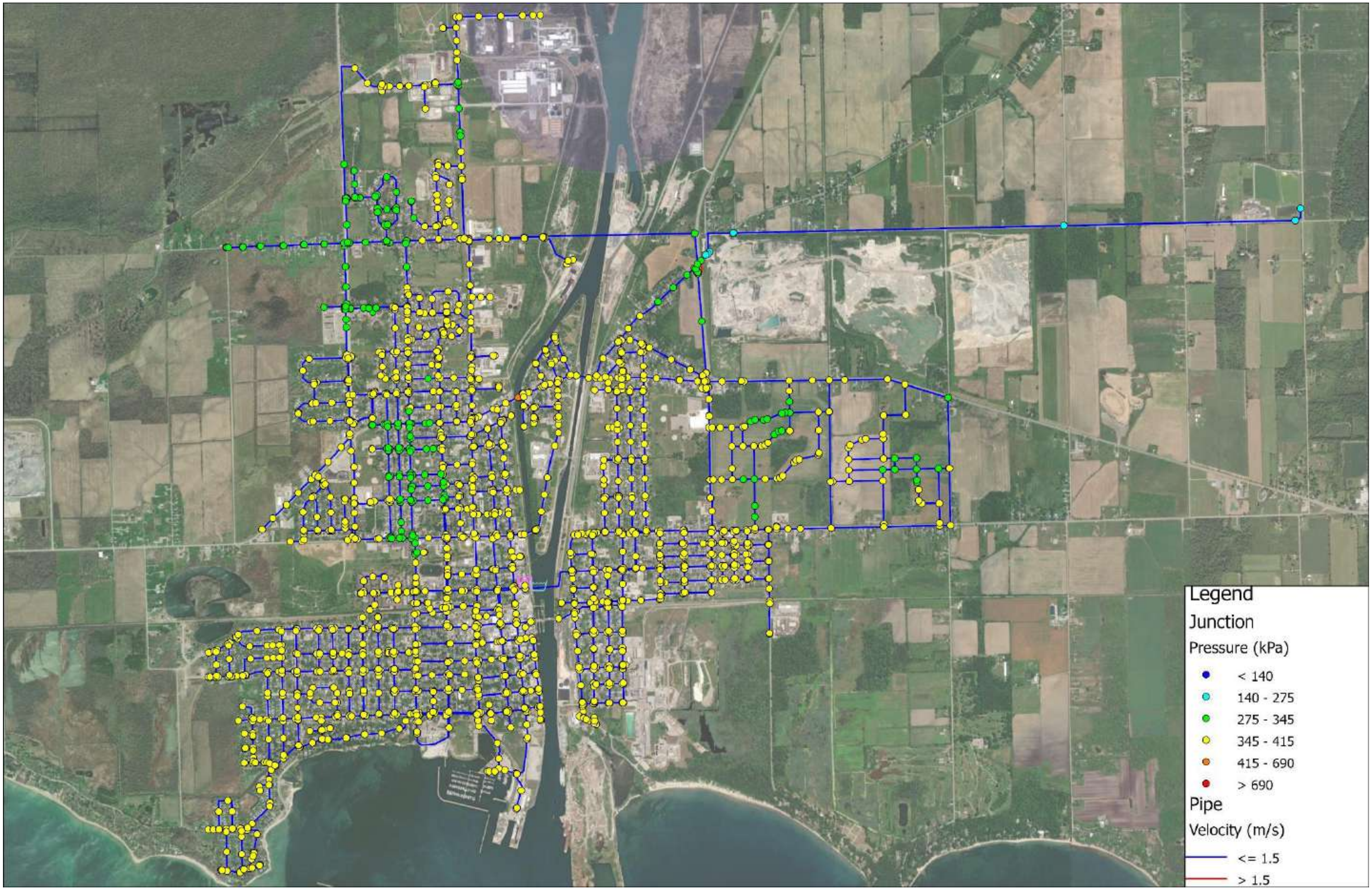


Figure 7 – Pre-Development with 2041 Peak Hour Demand + Fire

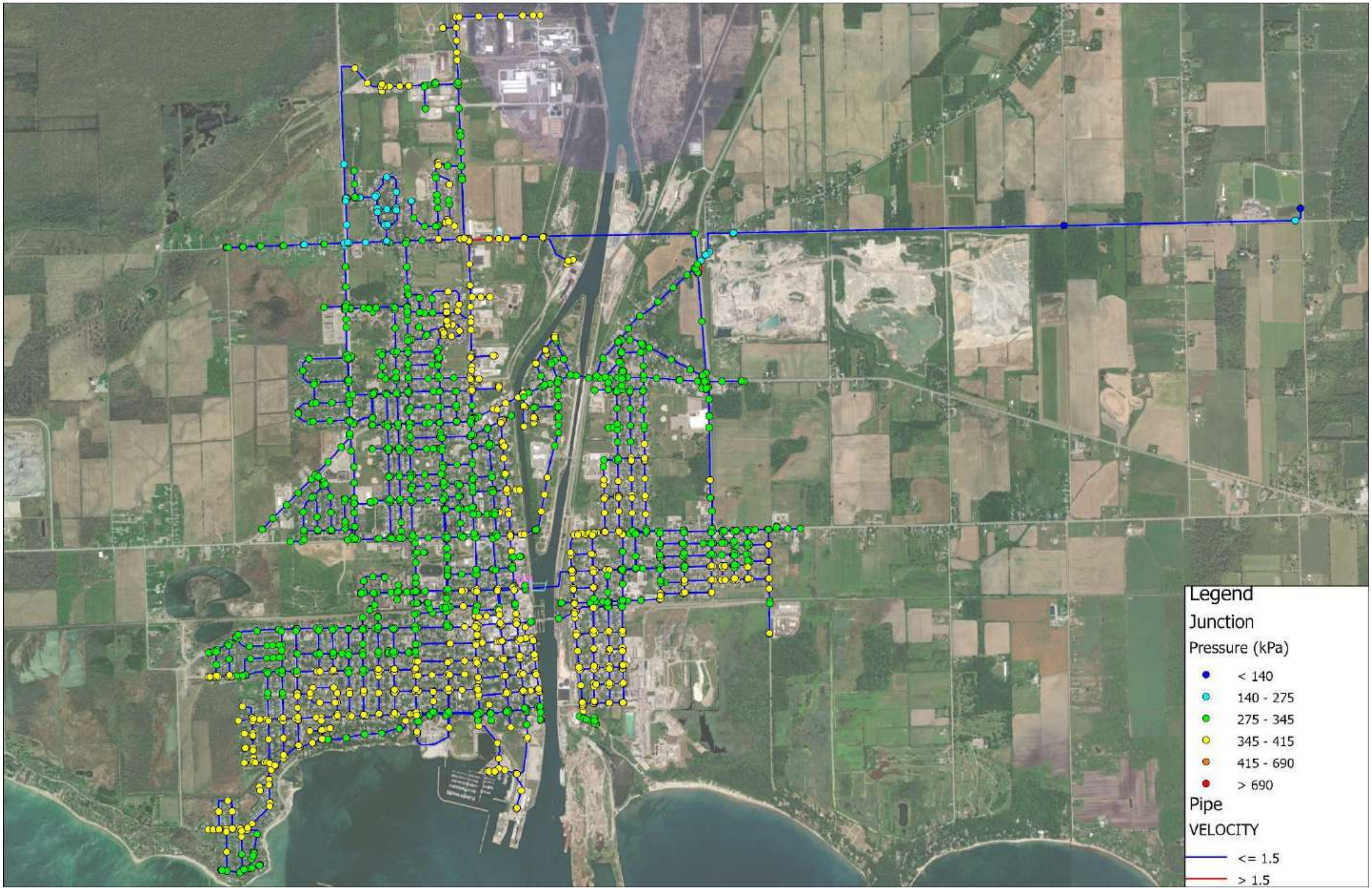


Figure 8 – Post-Development with 2041 Peak Hour Demand + Fire

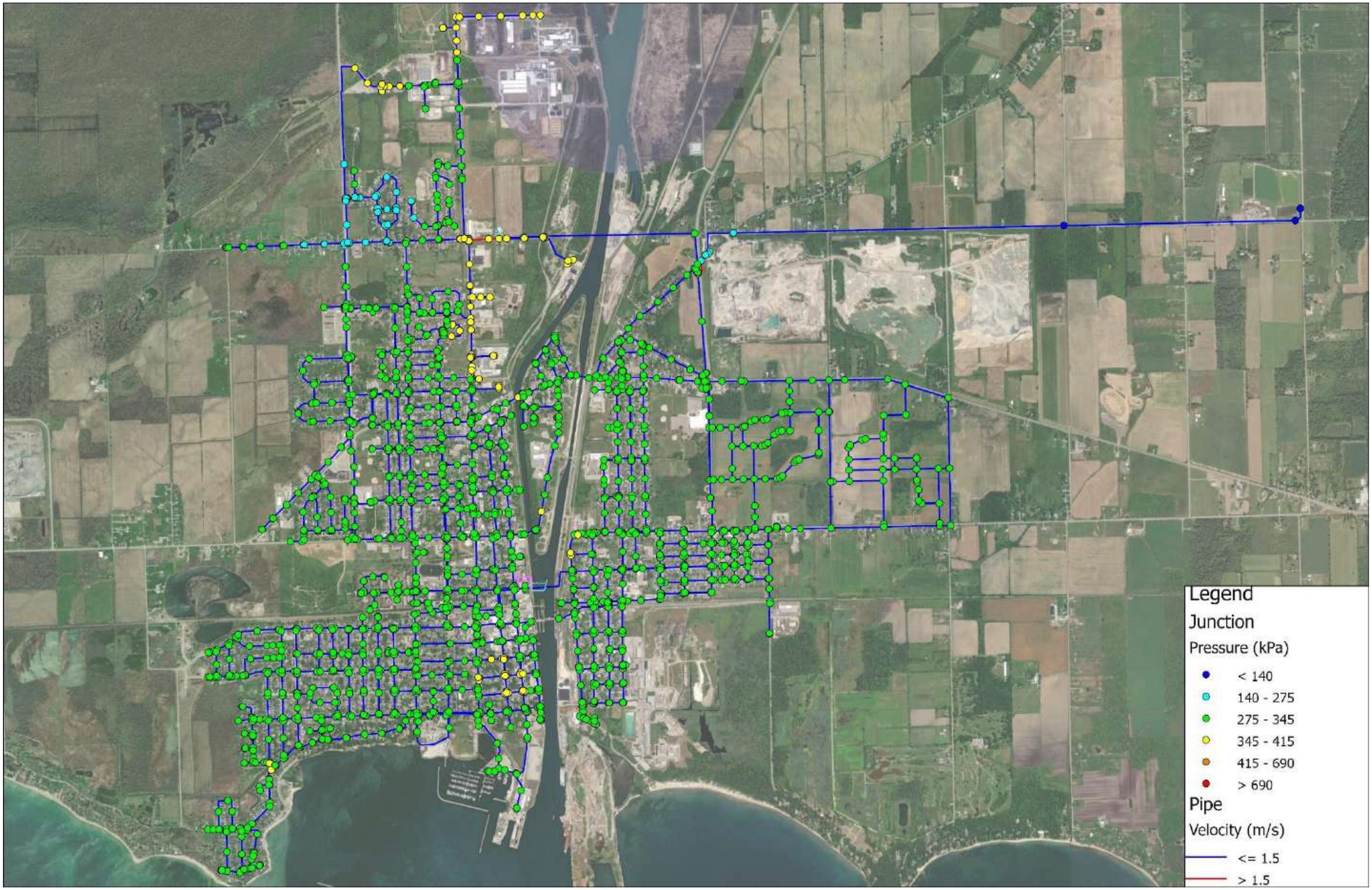


Figure 9 – Post-Development Available Fire Flow with 2041 PH Demand (detail of Site)

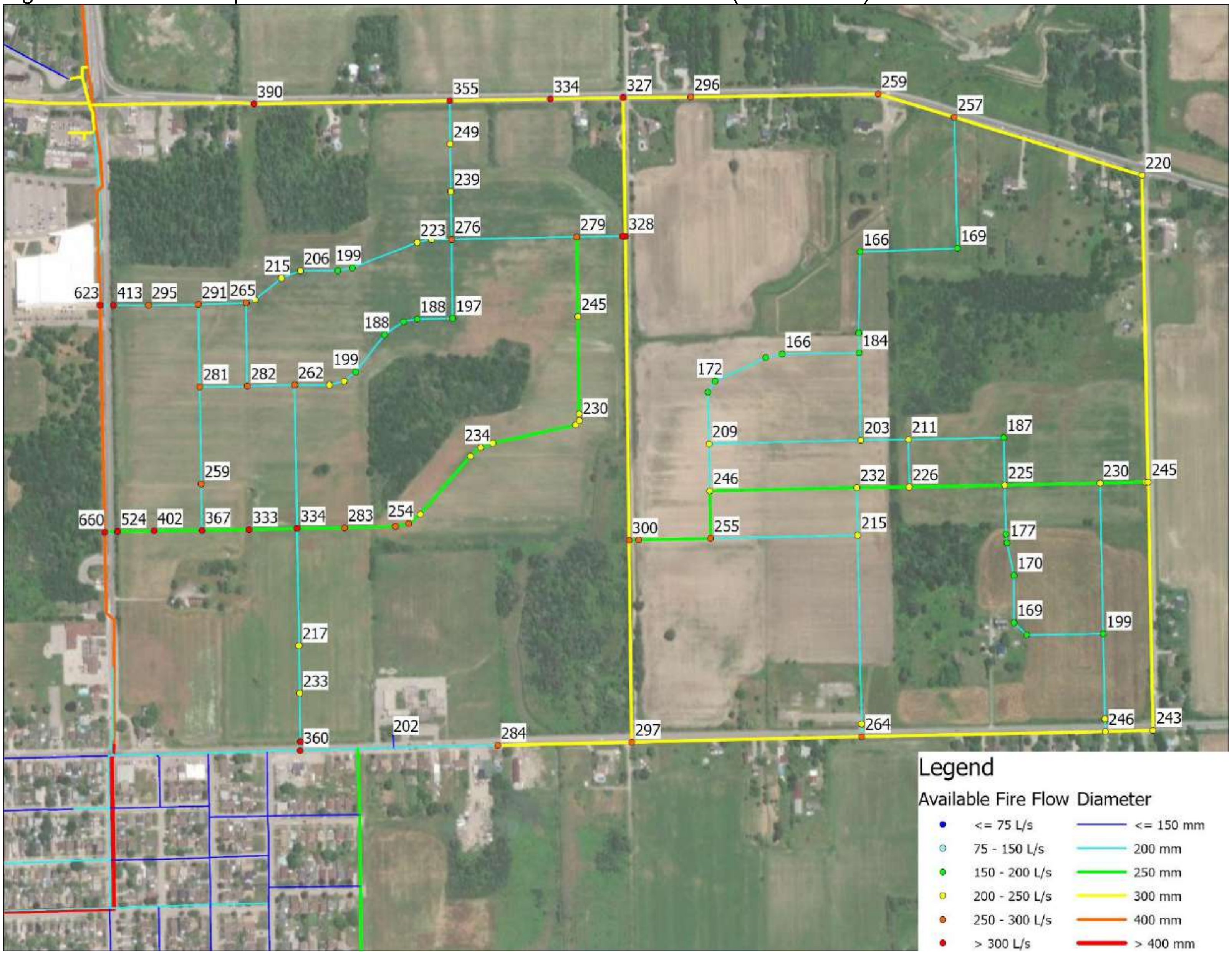


Figure 10 – Pre-Development with 2041 Minimum Hour Demand

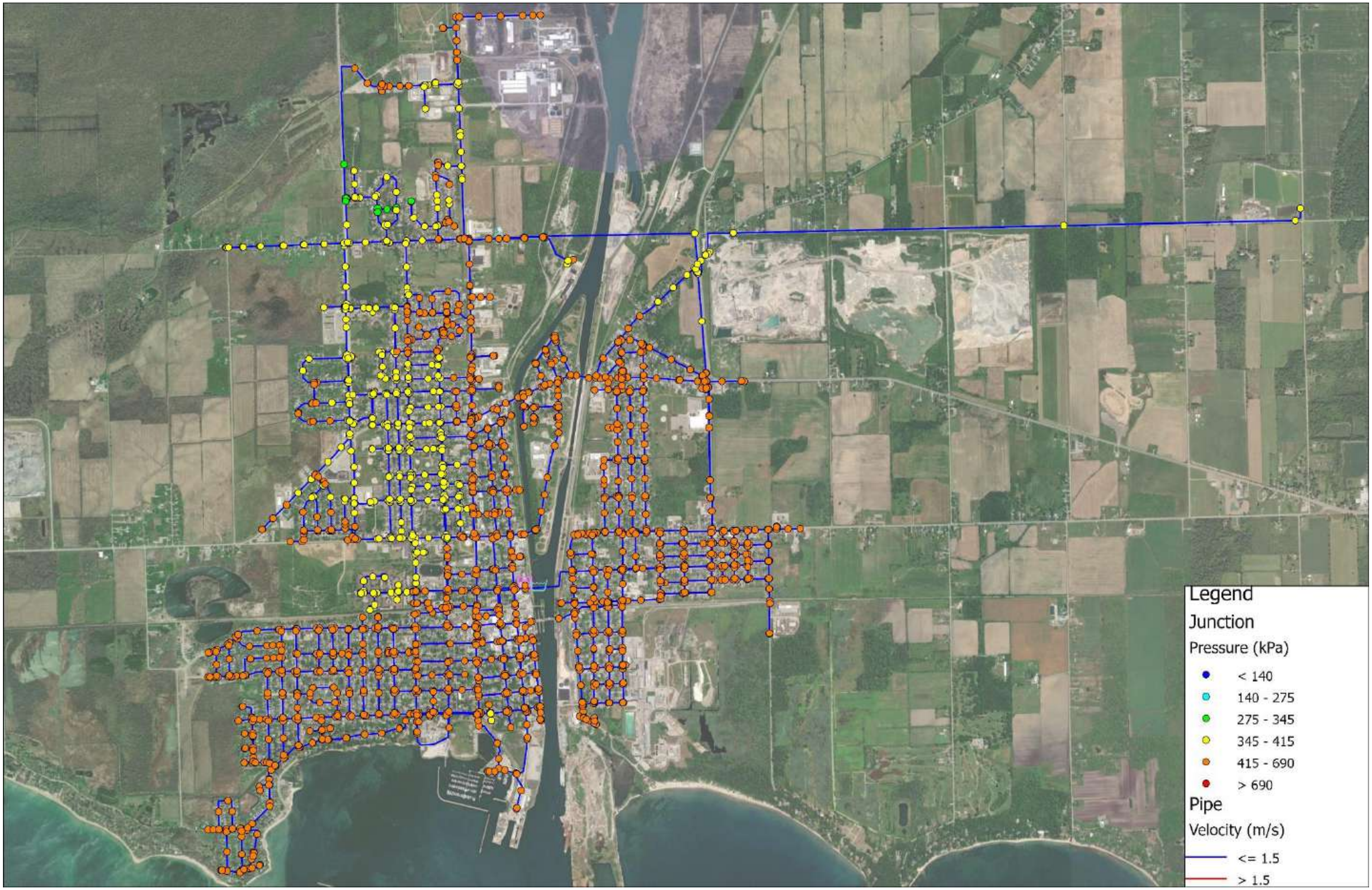


Figure 11 – Post-Development with 2041 Minimum Hour Demand

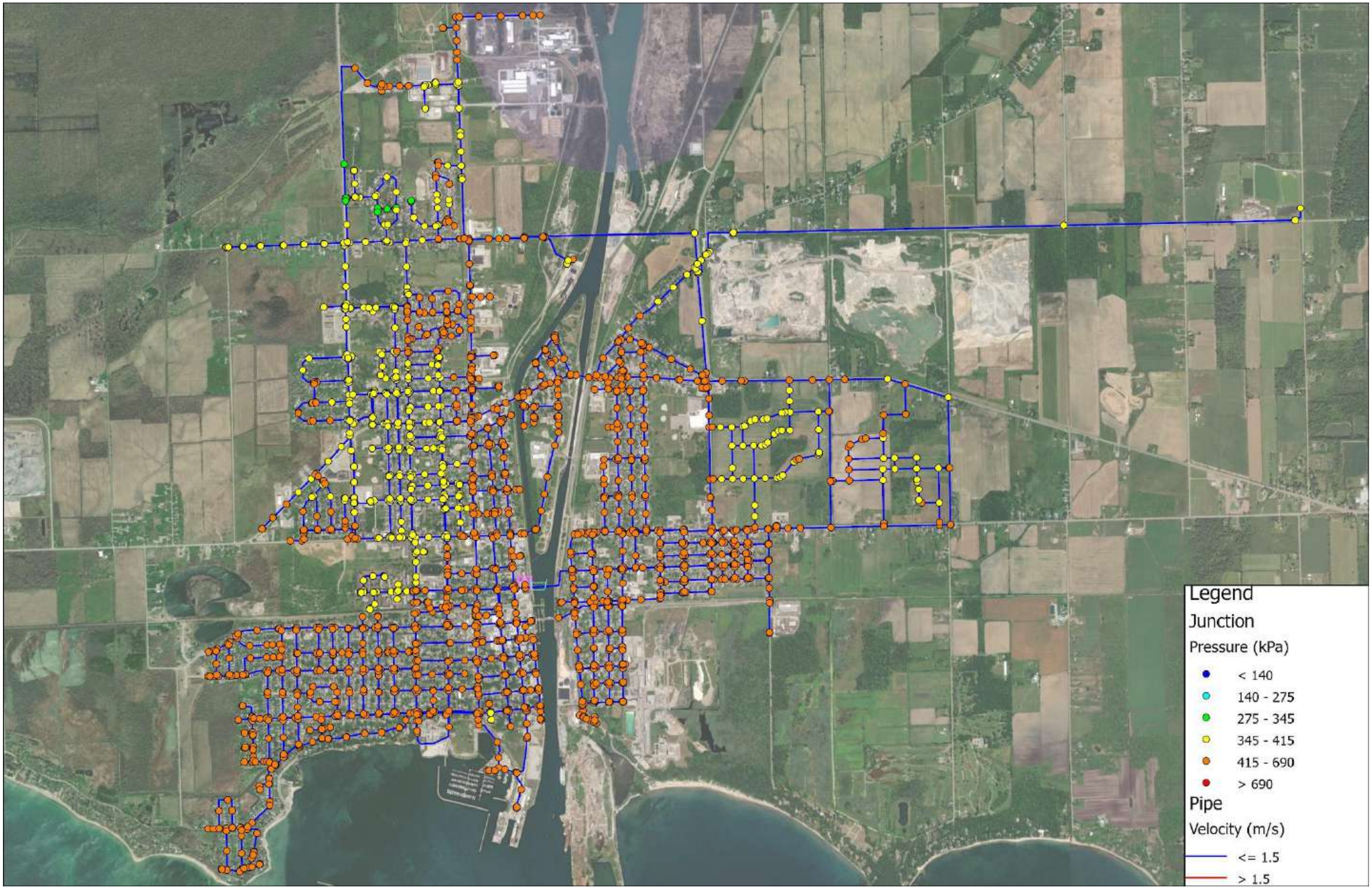
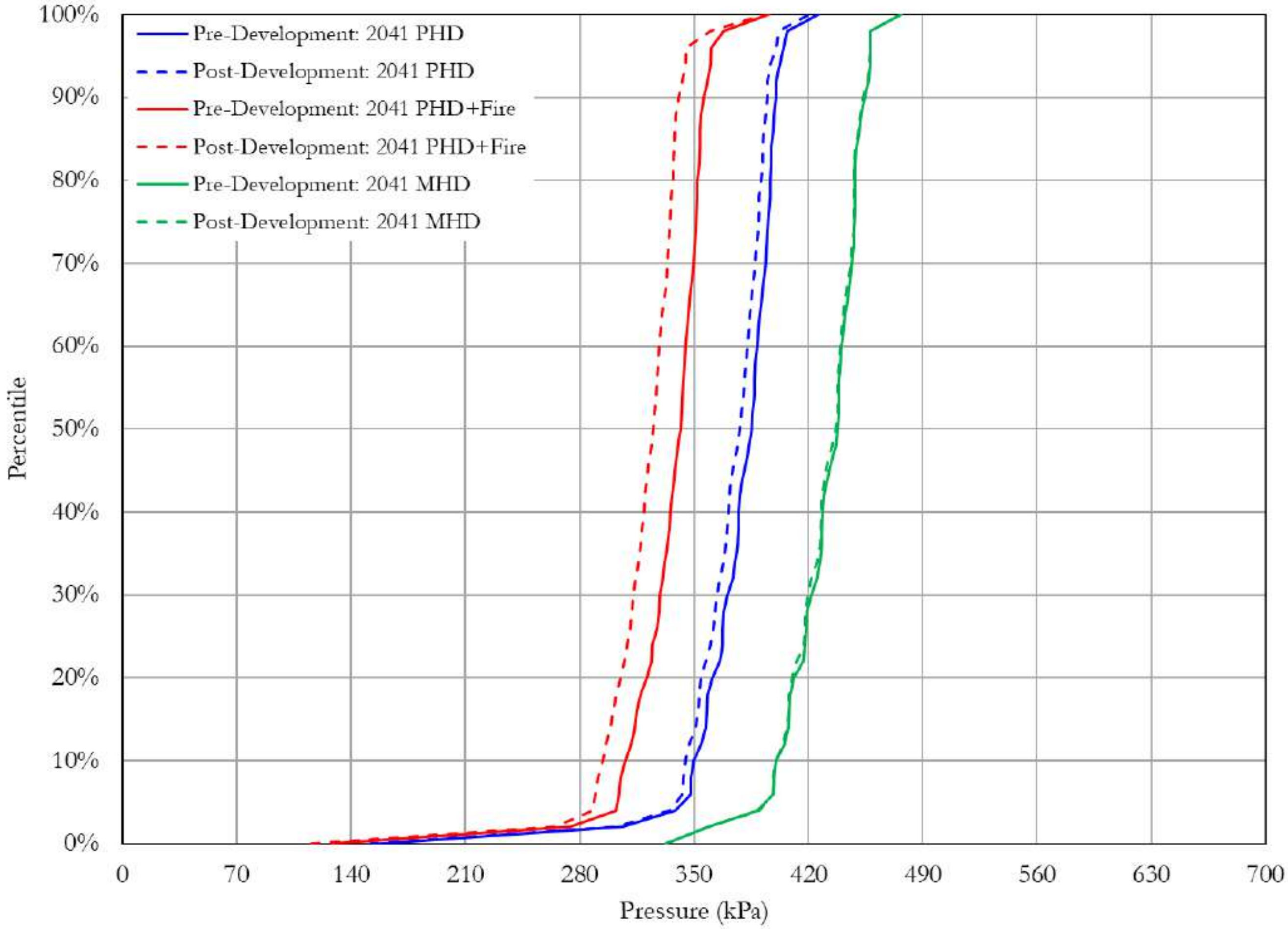


Figure 12 – Pre and Post Pressure Distribution Plot



The above plot is an encompassing result plot showing and comparing many things. For example, under full buildout, and MHD (minimum hour demand) 100 % of the area has pressure less than 475 kPa. In addition, 0% of the area has a pressure less than 340 kPa under MHD. Note, in the beginning of the development all the pre/post curves will be close to each other. As demand (development) increases the higher demands will show a more pronounced deviation in post/pre as shown above. In essence the top and bottom of the graph is the operating range of the system for that demand. Note, it is the long dead-end pipe at the north end of the system that shows poor pressure at the dead end. It is the same in the existing condition. It shows up in the above graph in the PHD and Fire flow demand (red and blue graphs).

Discussion of water distribution

- While distribution system pressures are predicted to decrease at full build-out of the study area, these are still within the City's acceptable pressure range.
- City Criterion normal Pressure Range 275 kPa (40 psi) to 690 kPa (100 psi) has been met.
- City Criterion Maximum Velocity 1.5 m/s has been met.
- Anticipate fire flows for the Site has been met.
- Pressures through out the system is greater than 140 kPa (20 psi) during fire flows has been met.
- Exact fire demand can not be determined at this early stage. The values given in Table 2 should be sufficient.
- The system analysis assumes the supply of water to meet the demands will be available. The system supply side was not analysed.
- The development pipe routing is well looped. Note, only the major distribution pipes were modelled (200mm, 250mm and 300mm). The lesser pipes will be looped and routed in accordance with City standards.
- It is recommended that in the phasing of the development, that all pipes are looped, with only continuation stubs (+/- 1 m) left or a hydrant placed at the dead end.
- Note, it is the long dead-end pipe at the north end of the system that shows poor pressure at the dead end. It is our understanding that the Region will rectify this in the future.

5.0 STORMWATER MANAGEMENT CONSIDERATIONS

INTRODUCTION

The proposed development is located within the Wignell Creek Sub watershed. Refer to Figure 13 for location of the proposed development.

Prior to embarking on the Storm Water Management (SWM), we reviewed the following reference reports:

1. "**Wignelll Watershed Hydrology and Hydraulics Report**", EWA Engineers Inc., August 31, 2021.

2. **"NIAGARA PENINSULA CONSERVATION AUTHORITY FLOOD PLAIN MAPPING WIGNELLL DRAIN CITY OF PORT COLBORNE"**, NIAGARA PENINSULA CONSERVATION AUTHORITY, August 2011.

Report 1 above is The Wignelll Drain Engineer's Report is prepared as follows:

a. Baseline Drainage Report.
b. Wignelll Watershed Report.

Report 2 above was undertaken by (NPCA) to generate the regulatory 100-year flood plain mapping for the Wignelll Drain Watershed in the City of Port Colborne. The generated flood plain extents will be used by the Niagara Peninsula Conservation Authority to regulate development within the 100-year flood plain, as mandated by the Conservation Authorities Act.

The subject report will be a hybrid of both reports, showing the drain capacities along with the flood HGL by incorporating the Culverts and their effects on the drains.

GOAL OF THE STORM WATER MANAGEMENT DESIGN

Stormwater management for the proposed development will set out the following goals:

1. Protect and manage quantity and quality of surface water and groundwater resources

2. Mitigate or minimize the risk of flooding and erosion in the Sub watershed

3. Preserve natural hydrological and hydrogeological systems

4. Identify the aquatic, wetland and terrestrial resources that should be protected or enhanced

5. Produce an implementation plan for Development of the Sub watershed Study area

6. Provide recommendations for the responsible storm water management Plan on a sub watershed level, if applicable

7. Develop an adaptive management guide for future activities in the sub watershed

Figure 13 – Development location relative to existing drainage sheds

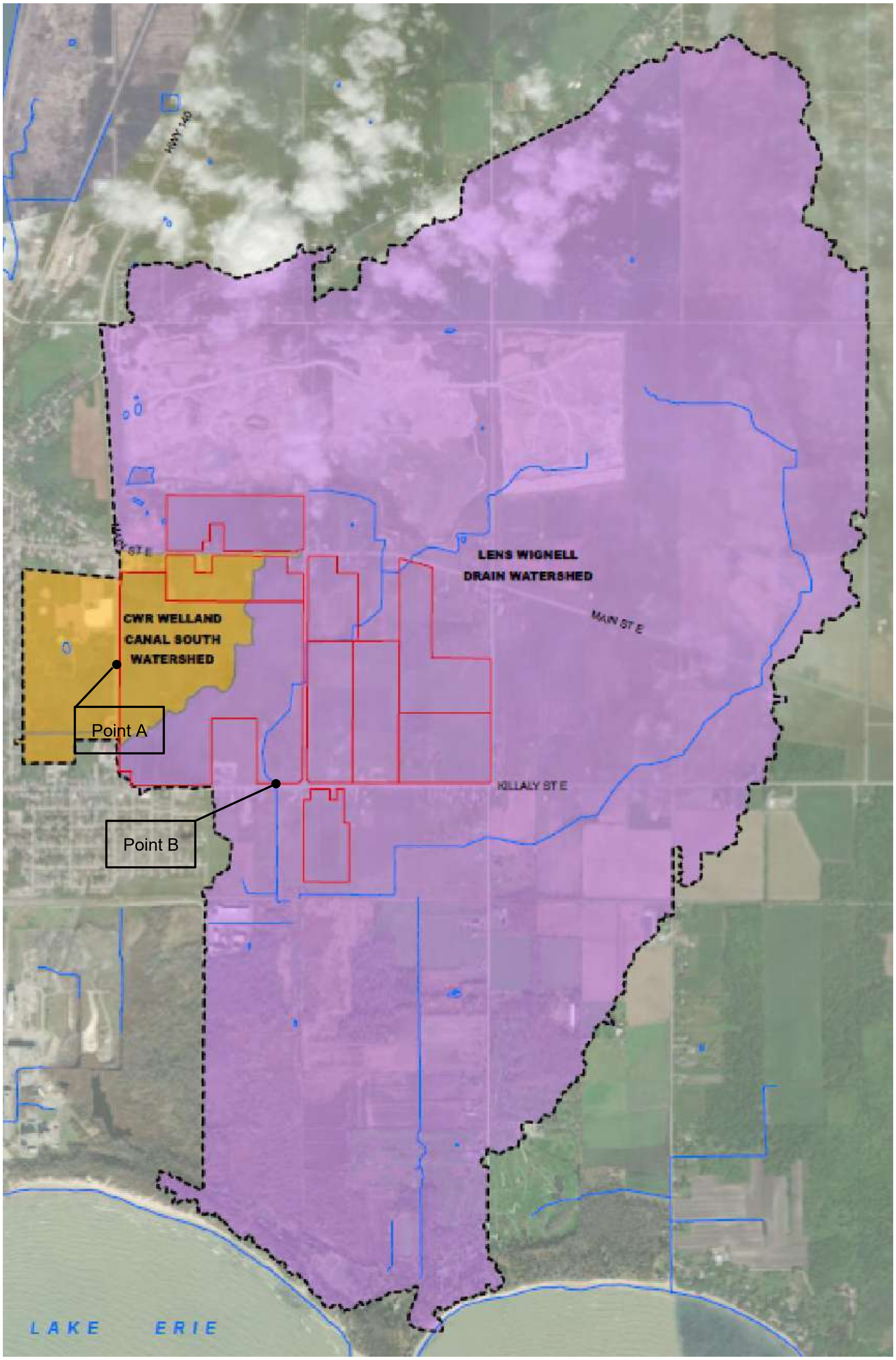


Figure 13 above shows that the development will drain to two distinct drainage systems:

1. Welland Canal south watershed.
2. Wignelll Drain watershed.

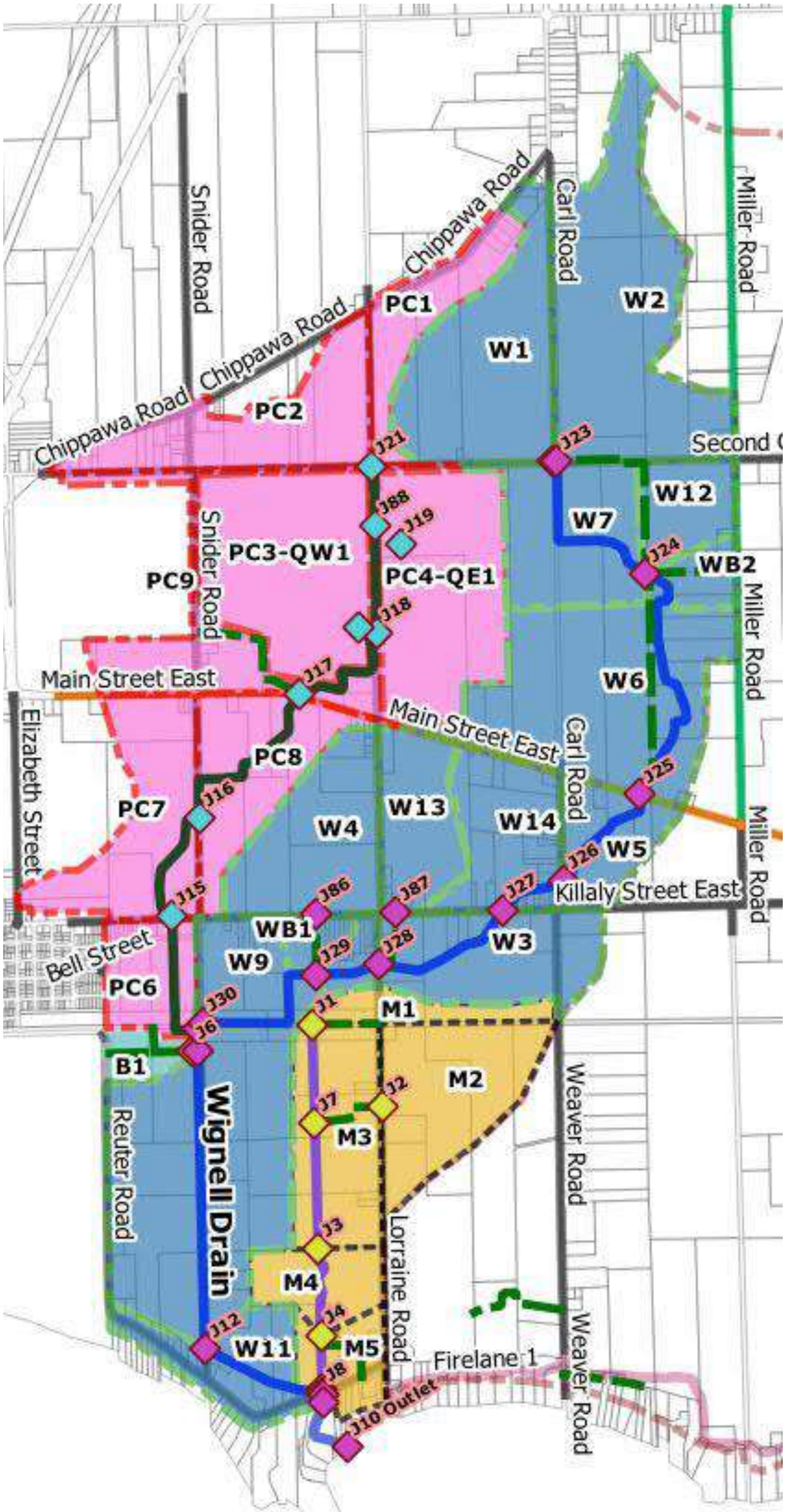
Figure 14 below is a schematic of the PCSWM model used by EWA Engineers to evaluate the Wignell Drain. The schematic shows the drains in yellow and the dashed red lines are the hydrologic connectivity from the subcatchments to the hydrologic nodes, shown in green.

The following Figure 15 and 16 are the EWA color coded model showing the branches of the Wignell Drain with key crossing numbers and the Wignelll Drain showing node (junction) numbering respectively.

Figure 14 – Wignelll Drain Study Area Hydrological Subcatchments Existing Conditions

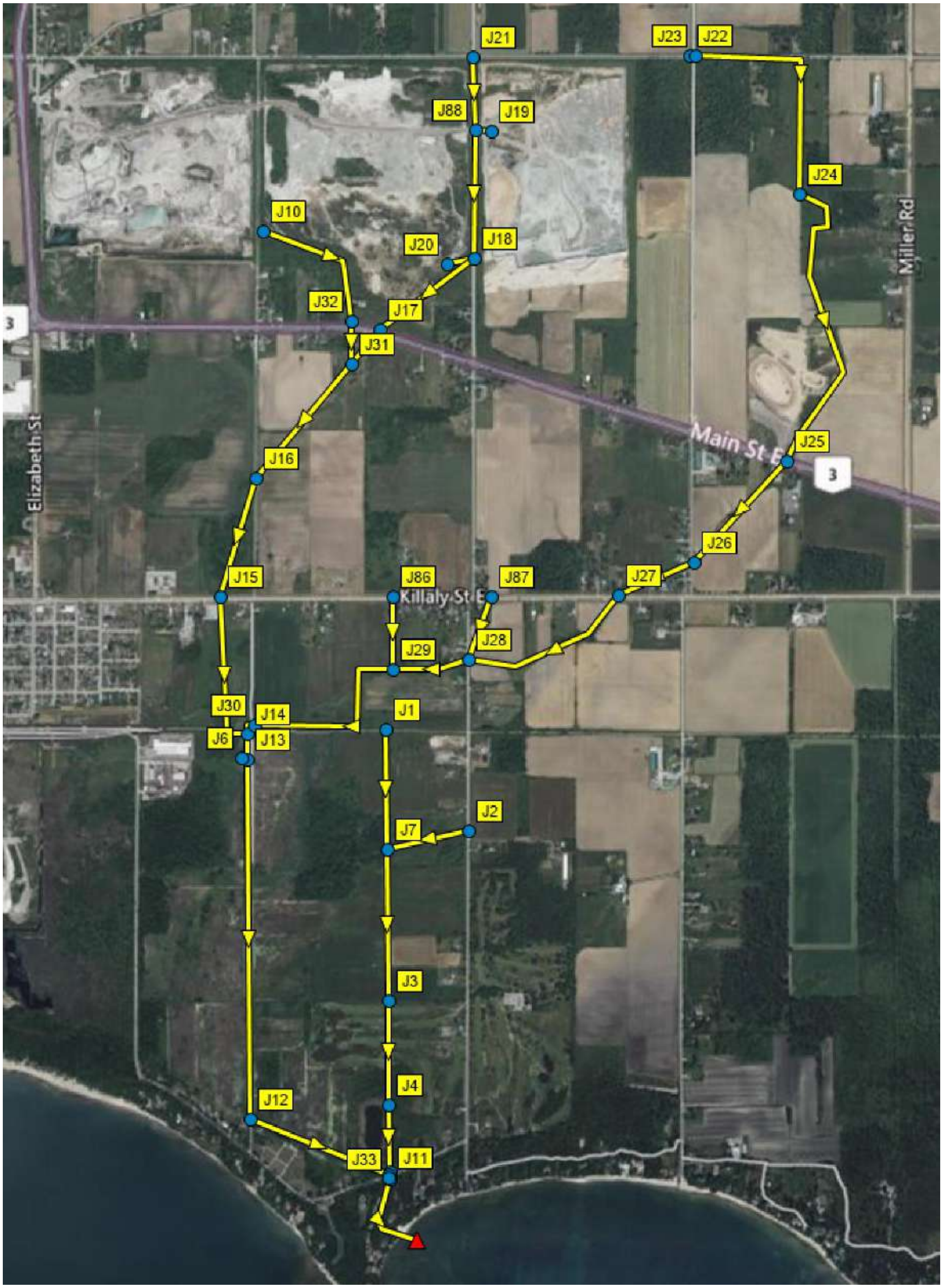
Total area = 1089.58 ha

Figure 15 – PCSWMM model of Wignelll Drain Study Area Existing Conditions



The above Figure is from the EWA PCSWMM model for the Wignell watershed report.

Figure 16 – PCSWMM model of WigneIll Drain showing node (junction) numbering



We will utilize the same node numbering shown above in the XPSWMM (2D) model for existing and proposed conditions.

EXISTING SUBCATCHMENT AREAS.

The subject site is located within the tributaries of the Central Park Drain, the Port Colborne Drain, and the Wignell Drain. The Central Park Drain drains west towards the Welland Canal and the Wignell Drain which drains into Lake Erie. The majority of the site is located within the Port Colborne Drain tributary which drains into the Wignell Drain south of the site and is therefore part of the Wignell Drain tributary.

The Wignelll drain total drainage area is 1089.58 ha. (from EWA PCSWMM model)

The site as mentioned above has two main drainage sheds as follows:

Total area to Welland Canal south watershed	46.28 ha
Total Wignelll Drain watershed	1089.58 ha (same as above)

NOTES ON EWA REPORT and MODEL:

1. We have adapted the same hydrology as EWA (refer to Table 1 from the EWA report).

2. We will have to create a new subcatchment for our predevelopment and post development model for the site area draining to the Welland Canal south watershed. The following will be used – existing - % impervious 1.0%, slope 0.1 %, width 480 m, CN =83, Area by DEM which is 46.28 ha.

3. PCSWMM model shows an area for sub-catchments PC7 of 54.01 ha, table 1 shows 46.29 ha.

4. PCSWMM model shows flow from the quarry areas plus the pump rates. The report table 2 shows the pump rate flow only. On page 21 of the report EWA states in the last paragraph on that page, the groundwater pump rate at the quarries were considered as the flow generated from the respective catchments and discharged into Wignelll Drain.

5. EWA set Quarry contributing areas with a static outflow to model pumping set at two rates:
   West Quarry = 0.057 m3/s, and
   East Quarry = 0.118 m3/s.
   We have adapted the same flow rates in the XPSWMM model.

TARGET FLOW

The above PCSWMM model was used to establish predevelopment hydrology target values to compare to the post developed conditions with the development added. The following Table 4 summarizes the location where existing target flows will be compared to post developed conditions.

Table 4 Target flow locations for the Wignell Drain

No.	Location description or crossing	Notes
S-1	Hwy # 3 culvert crossing west branch	Should be similar to PCSWMM
S-2	Hwy # 3 culvert crossing east branch	Should be similar to PCSWMM
S-3	Killaly culvert crossing west of Snider Road	Should be similar to PCSWMM
S-4	Killaly culvert crossing east of Lorraine Road	
S-5	Snider road culvert crossing just north of Friendship Trail (former CNR)	
S-6	Friendship trail culvert crossing adjacent to Snider Road west side	
S-7	Outlet to Lake	

Table 5 below summarizes the allowable (target flows) for the existing outlets per storm event.

The comparison will be made with XPSWMM (2D) existing conditions and XPSWMM (2D) with the development added. It is the only reliable way to compare as the original PCSWMM model is not 2D and does not include the culverts in real time. The NPCA HEC-RAS model maintains continuity of flow and thus has no attenuation at culverts. We will include the flows of PCSWMM at the above locations for comparison only.

Table 5. Pre-Development Existing Flow Targets

Storm Event	Storm Type	**Target Peak Flow Rate (m^3/s)**						
		S-1	S-2	S-3	S-4	S-5	S-6	S7
		Existing	Existing	Existing	Existing	Existing	Existing	Existing
2 Year	24Hr SCS	0.439	0.730	1.092	0.685	1.025	1.026	0.480
5 Year	24Hr SCS	0.794	1.215	1.815	1.554	1.948	1.956	1.049
10 Year	24Hr SCS	1.054	1.592	2.413	2.325	2.515	2.541	1.475
25 Year	24Hr SCS	1.402	2.164	3.225	3.415	2.654	3.374	1.770
50 Year	24Hr SCS	1.651	2.656	3.937	4.230	2.684	3.886	1.955
100 Year	24Hr SCS	1.916	3.159	4.689	4.979	2.687	4.374	2.164
100 Year	12Hr AES	0.606	1.976	4.105	4.619	2.694	4.048	1.766

A development such as the subject site requires a team approach with many disciplines. Planners, Environmental Consultants, Hydrogeologist/Soil Consultants and Civil Engineering Consultants. The development fabric was determined based on the Survey/Topographical Plan, Provincial Lidar derived DEM and the natural heritage features as determined by SLR. Armstrong is the Planner, SLR is the Environmental Consultants, EXP is the Hydrogeologist/Soil Consultant and Odan/Detech Group Inc. are the Civil Consultants.

The development fabric (Draft Plan of Sub-Division) was assembled by Armstrong through input by the Developer, SLR, EXP and Odan/Detech group inc. Refer to Appendix A for the reduced version of the Draft Plan of Subdivision by Armstrong. It is recommended to utilize 6 (six) storm water management (SWM) ponds for the development of the Port Colborne site as shown on drawing S-6 in Appendix D and on the Site Conceptual Plan in Appendix A.

The ponds outlet to the Port Colborne Drain, the Central Park Drain, and the Wignell Drain at various locations. Pond A will direct the minor flows and emergency overflows to the Elizibeth Street Ditch at the location of the existing culvert crossing. The conditions at the Elizibeth Street culvert were modelled in XPSWMM and it was found that the existing culvert acts as an orifice which results in ponding upstream of the culvert. Therefore, the allowable flows at the Pond A outlet have been further restricted to the existing culvert capacity to ensure no negative impacts downstream of the existing culvert. Ponds B, C, E, and F direct minor flows and emergency overflow to the Port Colborne Drain which runs through the subject site. Pond D will direct minor flows to a proposed sewer system on Killaly St W while emergency overflows will be directed to the existing ditch system on Killaly. Culvert upgrades will be assessed at detailed design. All minor flow and emergency overflow locations match pre-development locations which is required to match post development conditions to pre-development conditions. Any flow redirection would have negative impact locally to the revised outlet location.

The preferred method to determine an allowable (target flow) is to use the predevelopment unit runoff rates. The following is the procedure:

1. Pick a point in the drainage system where there is a known flow from PCSWMM model. The site as mentioned above has two drainage sheds as follows:

   Welland Canal south watershed - Point A ditch at Elizabeth Street (see Figure 13)
   Wignelll Drain watershed - Point B culvert crossing Killaly Street, west of Snider Road.

2. For the Welland Canal south watershed - Point A, the calculated flows are based on hydrology shown on page above.
3. For the Wignell Drain – Point B, the following summarizes the procedure used to derive the target flow at Point B using tributary areas in the original PCSWMM and unit flow rate per hectare for storm ranging from the 2 to 100-year flow. The target flows were determined for each pond (A-F) as shown below. Catchment G at the southwest corner of the site will not drain to any of the stormwater management ponds and will be required to provide site level stormwater management to meet the target flows for this catchment.

   Notes: The quarry areas were removed from the allowable flow calculation because they do not contribute runoff flow. Quarry areas are included in the model as pumped steady state flows.

From PCSWMM model												unit rate
								Point B		Point B		m3/s/ha
Name		Area (ha)						area (ha)		flow (m3/sec)		
W10		8.32						274.24		6.86		0.025
W6		28.7457										
W2		26.526		Flow area to point B								
PC3-QW1		41.95										
PC4-QE1		18.79							Pond Trib		100 year	
W1		16.7049		19.3425					area (ha)		allowable flow	
PC7		19.3425		63.43								
W4		1.98		7.7				Outlet A	52.28		0.921	
M3		3.65		18.3597								
W7		36.5969		42.97								
W3		66.06		82.3056								
PC8		63.43		23.23				Outlet B	274.24		6.86	
PC2		7.7		6.88								
W14		20.8394		10.0218								
M1		54.0114						Pond A			0.921	
W13		39.1345		**274.24**	ha							
M2		8.8715						Pond B	23.33		0.584	
W11		5.4412										
W9		58.2949						Pond C	31.354		0.784	
W5		100.6										
PC6		26.23						Pond D	33.849		0.847	
PC1		18.3597										
M4		28.7148						Pond E	11.827		0.296	
W12		34.15										
M5		77.959						Pond F	8.004		0.200	
WB2		41.21										
PC9_3		42.97										
B1		22.3					example : 100 year allowable flow pond B = 23.33 x 0.025					
PC5		82.3056								=	0.584	
WB1		41.66										
W8		6.61					for Outlet A - the flows are based on existing hydrology					
PC9_4		23.23										
PC11		6.88										
PC10		10.0218										
		1089.59										

Summary of Pond Target Flows:

Pond	Pond Trib area (ha)	24 hr SCS 100 year Target flow m3/sec	24 hr SCS 50 year Target flow m3/sec	24 hr SCS 25 year Target flow m3/sec	24 hr SCS 10 year Target flow m3/sec	24 hr SCS 5 year Target flow m3/sec	24 hr SCS 2 year Target flow m3/sec	12 hr AES 100 year Target flow m3/sec
Pond A	50.17	0.751	0.603	0.467	0.299	0.192	0.085	0.728
Pond B	24.43	0.611	0.567	0.451	0.314	0.224	0.118	0.510
Pond C	31.92	0.798	0.741	0.589	0.411	0.293	0.154	0.666
Pond D	33.05	0.827	0.768	0.610	0.425	0.304	0.159	0.689
Pond E	12.72	0.318	0.296	0.235	0.164	0.117	0.061	0.265
Pond F	7.87	0.197	0.183	0.145	0.101	0.072	0.038	0.164
Catchment G	4.66	0.117	0.108	0.086	0.060	0.043	0.022	0.097

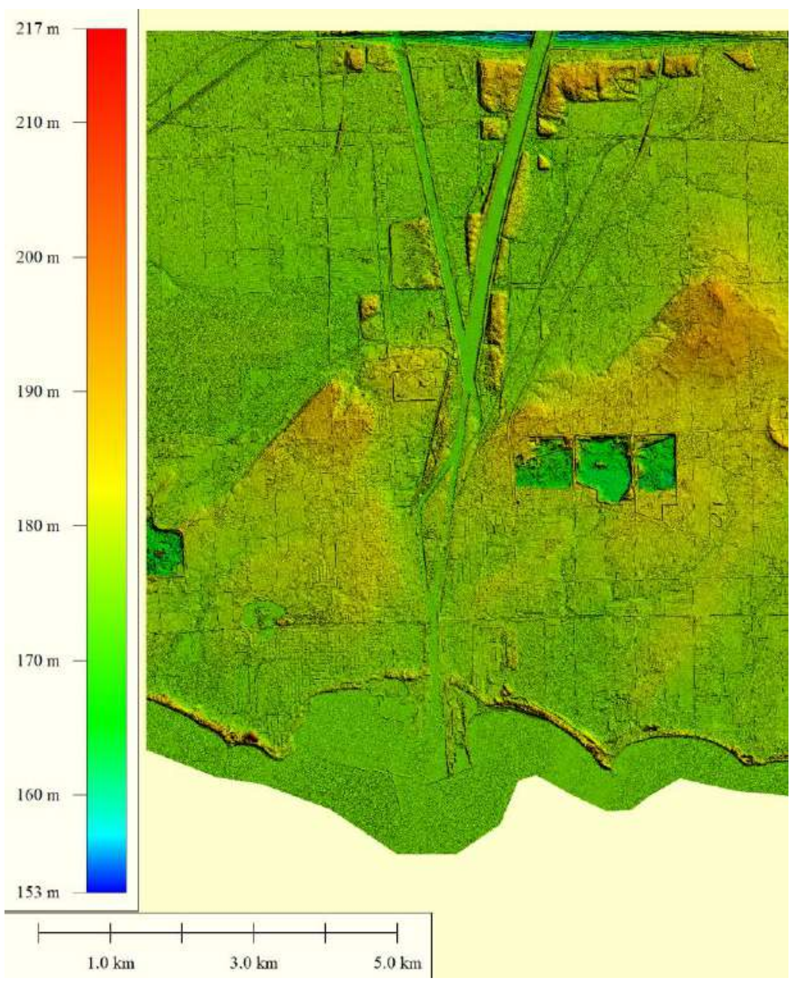
POST-DEVELOPMENT CONDITIONS

Schematic storm sewer distribution concepts have been prepared for the subject development as shown on Figure S-7 in Appendix D. Sewer layouts and elevations were established based on existing drainage patterns and below grade rock elevations. The sewer layouts on Figure S-7 are conceptual and are subject to change as the subdivision design evolve.

TOPOLOGY:

The XPSWMM model utilized the Province's 2018 DEM (Digital Elevation Model) which is Lidar derived. The grid density is 0.5m x 0.5m. We obtained and reviewed topographic survey of the existing site (existing terrain) prepared by MTE, dated February 2021, August 2021. We compared the existing MTE topographic survey to the lidar used in the hydraulic model and find them comparable. It should be noted that the Lidar and MTE data are based on the same vertical datum. The Lidar was adjusted to match the MTE topology. Odan Detech will utilize the adjusted Provincial 2018 DEM (based on Lidar) as the base line existing topology. Refer to Figure 17 for the shaded DEM for the study area.

Figure 17 – Adjusted Provincial 2018 DEM for the study area.



METHODLOLOGY

The Floodplain Analysis work will in brief include the following tasks:

1. Review City “Wignell Watershed Hydrology and Hydraulics Report”, EWA Engineers Inc., August 31, 2021.
2. Review “NIAGARA PENINSULA CONSERVATION AUTHORITY FLOOD PLAIN MAPPING WIGNELLL DRAIN CITY OF PORT COLBORNE”, NIAGARA PENINSULA CONSERVATION AUTHORITY, August 2011.
3. Compare Province DEM (Lidar derived) to MTE Surveyors Ltd. Adjust the Province DEM to the MTE topo. The adjusted Lidar DEM will become the base line.
4. Request and obtain from City the PCSWMM hydraulic models used by EWA in preparing the Wignell Watershed Hydrology and Hydraulics Report
5. Review the PCSWMM model.
6. Create a 1D/2D model (XPSWMM 2D).
7. Compare existing conditions to that of the developed site.

The following items will be evaluated via the XPSWMM 1D/2D model:

- Flood Elevations (Regulatory)
- **HGL at the subject areas.**
- Bed shear Calculations
- Velocity and depth Calculations (Hazard).

HYDRAULIC ANALYSIS:

XPSWMM 1D/2D MODEL:

The model we chose to use is XPSWMM. Many items were transferable. The boundary conditions and 1D nodes were easily copied and pasted from PCSWMM to XPSWMM.

XPSWMM was run with the following scenarios:

1. Existing 0 (Base) - The existing scenario was simulated using the hydrology in the “Wignelll Watershed Hydrology and Hydraulics Report”, EWA Engineers Inc., August 31, 2021. The storms run was SCS 24 hr -100, 50, 25, 10, 5, 2-year events and AES 12 hr – 100-year event. The Lake Erie boundary condition was considered as free flow. Please refer to Figure S-5 in Appendix D for the pre-development catchment areas that were modelled.

2. Site Developed 1 - The existing scenario was modified to simulate (add) 6 urban areas flowing to 6 SWM ponds (Site Developed). The storms run was SCS 24 hr -100, 50, 25, 10, 5, 2-year events and AES 12 hr – 100-year event. The Lake Erie boundary condition was considered as free flow. Please refer to Figure S-6 in Appendix D for the pre-development catchment areas that were modelled.

3. Existing 3 Lake boundary modified - The existing scenario was simulated using the hydrology in the “Wignelll Watershed Hydrology and Hydraulics Report”, EWA Engineers Inc., August 31, 2021. The Lake Erie boundary condition was 100-year Lake level + 10-year surface runoff event.

4. Site Developed 1 Lake boundary modified - The existing scenario was modified to simulate (add) 6 urban areas flowing to 6 SWM ponds (Site Developed). The Lake Erie boundary condition was 100-year Lake level + 10-year surface runoff event. The pond volumes were modelled based on conceptual pond designs for each pond which can be referenced on Figures S-9 to S-16 in Appendix D.

MODEL:

The Hydrodynamic 2D model utilized is XP2D by Innovyze. XPSWMM 1D is similar to and has the modified EPA SWMM 5 engine. SWMM 5 models can be imported and exported into XPSWMM. XP2D is a computer program for simulating depth-averaged, two and one-dimensional free-surface flows such as occurs from floods and tides. XP2D is based on the computational engine TUFLOW which was originally developed for modelling two-dimensional (2D) flows, and stands for Two-dimensional Unsteady FLOW. XP2D has been dynamically linked (fully integrated) with the XPSWMM 1D solution engine.

2D: TUFLOW HPC's 2D explicit formulation assures unconditional stability. Thus, a reasonable initial time step is 1 to 5 seconds. The program will use the initial time step and divide it by 10 to start the simulation. From that point on the program will adjust.

1D: Finite difference Runge-Kutta explicit scheme. Scheme solves all terms of the St. Venant equations.

1D and 2D schemes automatically switch between upstream and downstream controlled flow regimes to represent shocks.

Prior to embarking on a 2D model routine it is customary practice to create a well-planned work flow so that major items are not missed. The pillar of this work flow is a well conceived Quality Control Check List. See the following check list. It is only checked after the modelling takes place to make sure the report and models are in sync.

Quality Control Check List

Item	*Description*	*Checked*
Modeling Log	A modeling log is highly recommended and should be a requirement on all projects. The log may be in Excel, Word or other suitable software. A review of the modeling log is to be made by an experienced modeller. It should contain sufficient information to record model versions during development and calibration, along with observations from simulations. A model version naming and numbering system needs to be designed prior to the modeling. The version numbering system should be reflected in input data filenames to allow traceability and the ability to reproduce an old simulation if needed.	√
File Naming, Structure and Management	A review of the data file management should check: • files are named using a logical and appropriate system that allows easy interpretation of file purpose and content. • a logical and appropriate system of folders is used that manages the files; • relative path names to be used for input files (e.g. "..\model\geometry.tgc") so that models are easily moved from one folder to another. • documentation of the above in, for example, the projects Quality Control Document and/or Modeling Log.	√
2D Cell Size	Check whether the 2D cell size is appropriate to reproduce the topography needed to satisfactorily meet the objectives of the study.	√
Topography	The topography review should focus on: • correct interrogation of DTM; • correct datum; • modifications to the base data (eg. breaklines) have been checked. Regarding the latter, this is effectively carried out by producing a _zpt GIS check file using Write Check Files. The _zpt layer contains all modifications including any flow constriction adjustments. A DTM can be created from the Zpts using Global Mapper, or other 3D surface software, to aid in the review. Note: Reviewing the elevations in the .2dm file is not appropriate as only the ZH Zpt is represented in the .2dm file (the ZH elevation is not used in the hydrodynamic calculations).	√
Bed Resistance Values	Bed resistance values are to be reviewed by an experienced modeller. The review should focus on checking at least one of: • **Roughness Categories** in the Global Database; • the grid "Mat" or "Manning_n" values in the grd GIS check file; or • specifying weir output using the weir approach. The reviewer should be looking for: • relative consistency between different land-use (material) types; and • values are within accepted calibration values.	√
Calibration / Validation	Check that the model calibration or validation is satisfactory in regard to the study objectives. Identify any limitations or areas of potential uncertainty that should be noted when interpreting the study outcomes.	√

Mass Conservation	Standard practice is to place PO flow lines at a minimum of several locations through the model. They are typically aligned roughly perpendicular to the flow direction. The locations should include lines just inside each of the boundaries. Other suitable locations are upstream and downstream of key structures, through structures and areas of particular interest. The flows are graphed and conservation of mass checked (i.e. the amount of water entering the model equals the amount leaving allowing for any retention of water in the model). Check that any 1D flow paths crossed by a PO line are also included in the mass check. In dynamic simulations, an exact match between upstream and downstream will not occur due to retention of water, however, examination of the flow lines should reflect this phenomenon. For steady-state simulations, demonstration of reaching steady flow conditions is demonstrated when the flow entering the model equals the flow leaving the model.	√
Free-Over fall & Weir Flow	Especially if **Supercritical** is set to OFF, the percentage of free-over fall and weir flow velocity points should be checked. The review should seek to check that excessive number of points are not free-overfalling, and if so: that this is in accordance with the expected flow (e.g. weir flow over a levee) – check that the weir option is on if significant weir flow exists; and/or the effect on the overall flow patterns is minimal. The review is best carried out by: • Monitoring the numbers after “CS” or “FO” on the screen or in the .tlf file • Specifying flow regime output to generate the _R.dat file. This file shows the flow regime. The presence of significant areas of supercritical and/or weirs can be acceptable in large areas of sheet flow. However, care should be taken in interpreting the flow behavior in these areas, particularly if the flow is supercritical as complex hydraulic processes (e.g. hydraulic jumps, surcharging against buildings) can occur. Typically, most supercritical and weir flow occurs: • around the edge of a model where it is wetting and drying and has little influence over the general flow behavior; or • down steep slopes or over significant drops (eg. over a levee).	√
Hydraulic Structures	Head losses through a structure need to be validated through: • Calibration to recorded information (if available). • Crosschecked using desktop calculations based on theory and/or standard publications (eg. Hydraulics of Bridge Waterways). • Crosschecked with results using other hydraulic software (e.g. HEC-RAS). Simple checks can be made by calculating the number of dynamic head losses that occur and checking that this in accordance with that expected It is important to note that contraction and expansion losses associated with structures are modeled very differently in 1D and 2D schemes. 1D scheme rely on applying form loss coefficients, as they cannot simulate the horizontal or vertical changes in velocity direction and speed. 2D schemes model these horizontal changes and, therefore, do not require the introduction of form losses to the same extent as that required for 1D schemes. However, 2D schemes do not model losses in the vertical or fine-scale horizontal effects (such as around a bridge pier) and, therefore, may require the introduction of additional form losses. See Syme 2001b for further details.	√
Eddy Viscosity	Check that the eddy viscosity formulation and coefficient is appropriate	√

Following on pages 38 and 39 Figure 18 and 19 contains the existing condition XPSWMM model. The XPSWMM model was built from the province's 2018 DEM which is Lidar derived. The raw data was imported into Global Mapper (Geospatial software) where it was reviewed and edited if necessary. A digital terrain model was created in Global Mapper. Through Global Mapper a grid file (XYZ) file was created and sent to XPSWMM. XPSWMM then creates the DTM. The DTM still satisfies the theory that a good 2D model should contain for each 2D cell at least 2 vertices on average.

TIME STEP:

As a rule, the time step is typically half the cell size. For steep models with high Froude numbers and supercritical flow, smaller time steps may be required. For this site it will not be an issue (this Site is relatively flat).

If the model is operating at high Courant numbers (>10), sensitivity testing with smaller time steps to demonstrate no measurable change in results should be carried out. The occurrence of high mass errors is also an indicator of using too high a time step. It is recommended that the time step of the 2D engine be equal to or an integer multiple of the time step of the 1D calculations.

We have adapted the following time steps to start:

1D model	1.0 sec
2D model	1.0 sec

XP2D is finite volume based, explicit formulation. As mentioned above the program will use the initial time step and divide it by 10 to start the simulation. From that point on the program will adjust. The above noted criteria are met and there are no stability issues with the model.

CELL SIZE:

The cell sizes of 2D domains need to be sufficiently small to reproduce the hydraulic behavior. Based on review of benchmark studies, experience and consultation with Innovyze the chosen **Grid size of 2.5 m** is adequate. It has enough resolution to capture local ditches, and space between buildings.

CELL ROUGHNESS:

The grid roughness was set to a Manning n of 0.050 for the existing natural areas and residential lawns as noted on Figure 18. There are no urban areas where the 2D grid is provided. Therefore, a single manning n is provided. The Developed site was modelled as a 1D system (hydrology nodes, to storage node, with control structure) out letting to a node linked to a 2D cell.

HYDROLOGY PARAMETERS:

The existing XPSWMM hydrology model is the same as the PCSWMM model. Refer to Table 6 for the post developed (site developed) hydrology parameters. The Post Developed model has urban runoff and the existing tributaries are subdivided.

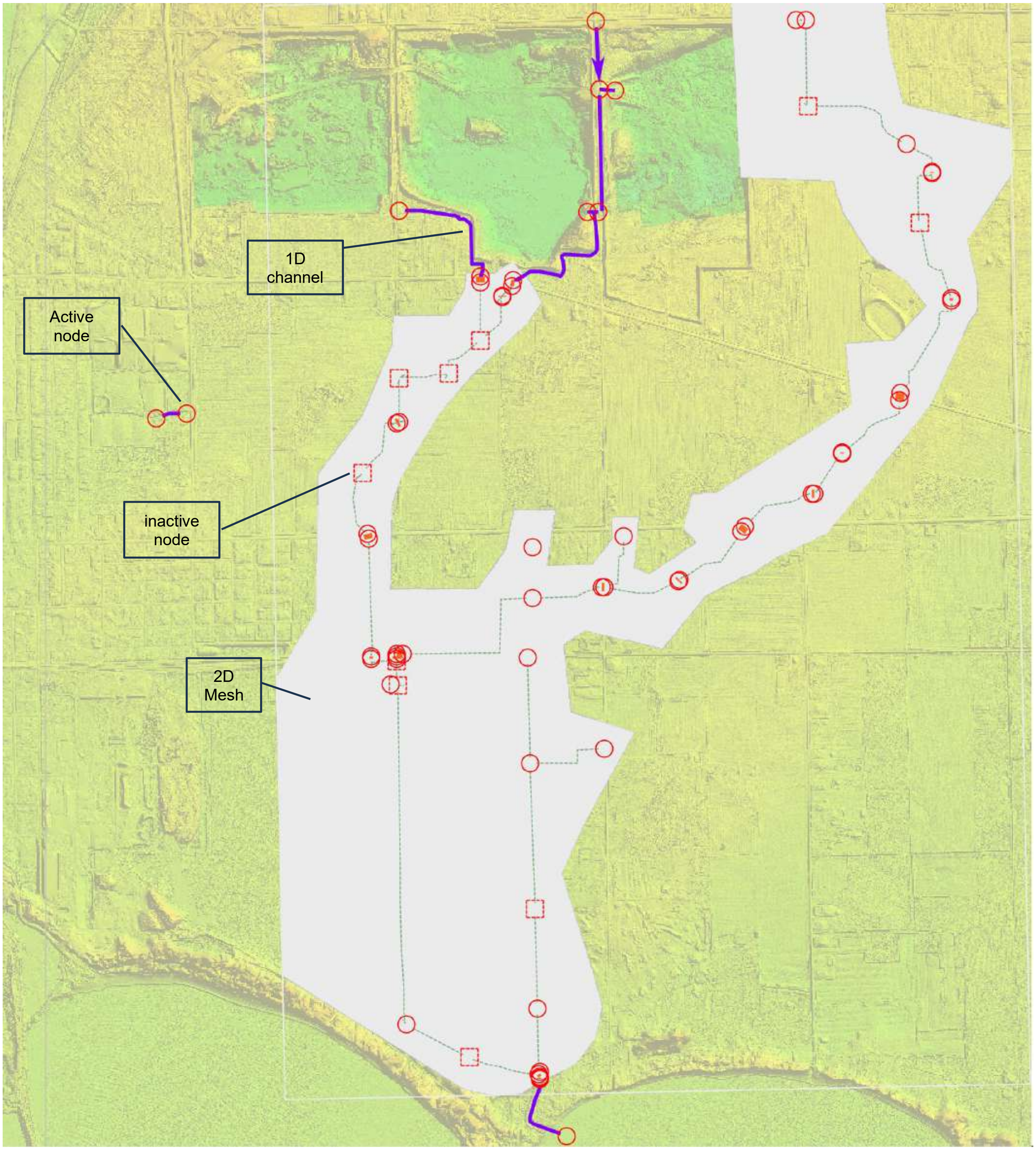
Table 6. Post Development hydrology parameters

Name	Subcatchment	Impervious Area depression storage (mm)	Pervious Area depression storage (mm)	Area ha	Width m	Slope m/m	Impervious Percentage %	Infiltration Reference	Hydrology Methods
J1	1	10	5	28.746	288.0	0.002	4.5	M1	SWMM
J10	1	10	5	5.441	60.0	0.007	85	PC9_4	SWMM
J12	1	10	5	100.6	680.0	0.01	4.5	W10	SWMM
J15	1	10	5	15.657	137.5	0.001	3	PC7	SWMM
J15	2	10	5	4.66	64.5	0.001	0	PC7	SWMM
J16	1	10	5	7.463	50.0	0.001	1	PC8	SWMM
J17	1	10	5	7.7	153.0	0.004	4.5	PC5	SWMM
J18	1	10	5	1.98	40.0	0.004	55	PC10	SWMM
J19	1	10	5	63.43	906.0	0.001	0	PC4-QE1	SWMM
J2	1	10	5	26.526	420.0	0.004	4.5	M2	SWMM
J20	1	10	5	66.06	660.0	0.001	0	PC3-QW1	SWMM
J21	1	10	5	19.343	198.0	0.005	4.5	PC1	SWMM
J21	2	10	5	36.597	374.0	0.002	4.7	PC2	SWMM
J22	1	10	5	58.295	511.0	0.008	4.5	W1	SWMM
J23	1	10	5	77.959	488.0	0.005	4.5	W2	SWMM
J24	1	10	5	18.36	275.0	0.002	4.5	W12	SWMM
J24	2	10	5	41.66	495.0	0.001	4.5	W7	SWMM
J24	3	10	5	10.022	250.0	0.002	4.5	WB2	SWMM
J25	1	10	5	82.306	986.0	0.001	4.5	W6	SWMM
J26	1	10	5	22.3	354.0	0.002	4.5	W5	SWMM
J27	1	10	5	34.15	491.0	0.003	4.5	W14	SWMM
J28	1	10	5	41.21	330.0	0.002	4.5	W3	SWMM
J29	1	10	5	6.607	185.6	0.003	4.5	W8	SWMM
J29	2	10	5	5.167	260.0	0.004	4.5	WB1	SWMM
J30	1	10	5	12.037	502.0	0.008	4.5	W9	SWMM
J32	1	10	5	8.87	239.0	0.007	4.5	PC9_3	SWMM
J4	1	10	5	18.79	469.8	0.01	4.5	M4	SWMM
J5	1	10	5	16.705	597.0	0.01	4.5	M5	SWMM
J6	1	10	5	8.32	201.0	0.003	5	B1	SWMM
J7	1	10	5	41.95	411.0	0.01	4.5	M3	SWMM
J8	1	10	5	26.23	1380.0	0.03	4.5	W11	SWMM
J88	1	10	5	3.65	36.5	0.004	45	PC11	SWMM
J15.1.1	1	10	5	20.839	279.5	0.002	4.5	PC6	SWMM
Node 56-1	1	2	5	2	86.6	0.02	60	urban	SWMM
J87	1	10	5	28.715	342.0	0.004	4.5	W13	SWMM
Node84	1	2	5	24.426	302.7	0.01	54	urban	SWMM
Node88	1	2	5	50.173	433.8	0.01	43	urban	SWMM
Node92	1	2	5	31.92	346.0	0.01	66	urban	SWMM
Node96	1	2	5	7.866	170.0	0.01	70	urban	SWMM
Node100	1	2	5	33.047	347.3	0.01	62	urban	SWMM
Node108	1	2	5	12.722	218.4	0.01	70	urban	SWMM
Node109	1	10	5	11.537	208.0	0.01	4	PC7	SWMM

Note the two quarry areas J19 and J20 are included in the above parameters, however the areas were turned off in the simulation. The flows from the quarry areas are included in the model as steady state pumped flows. The CN curve number of 83 was used in the urban infiltration model. The area = 1143.13 is very close to the Wignell Drain area of 1089.58 ha + 51.83 ha (flow area to south Welland, node88) = 1141.41 ha (existing conditions).

Figure 20 is the post development XPSWMM model showing additional nodes added to the model representing post development drainage areas shown in Figure 21. Figure 22 is a close up of the nodes at Pond A and Pond C that have outlets crossing Elizabeth St and Killaly Street respectively.

Figure 18 – XPSWMM Global Existing model



Legend: Δ - storage node, o –node/MH, Link (pipe) dummy link inactive node
Link culvert Link open channel Link orifice and or weir

Grid Manning n = 0.05

Figure 19 – XPSWMM existing model showing Tributary areas and node links

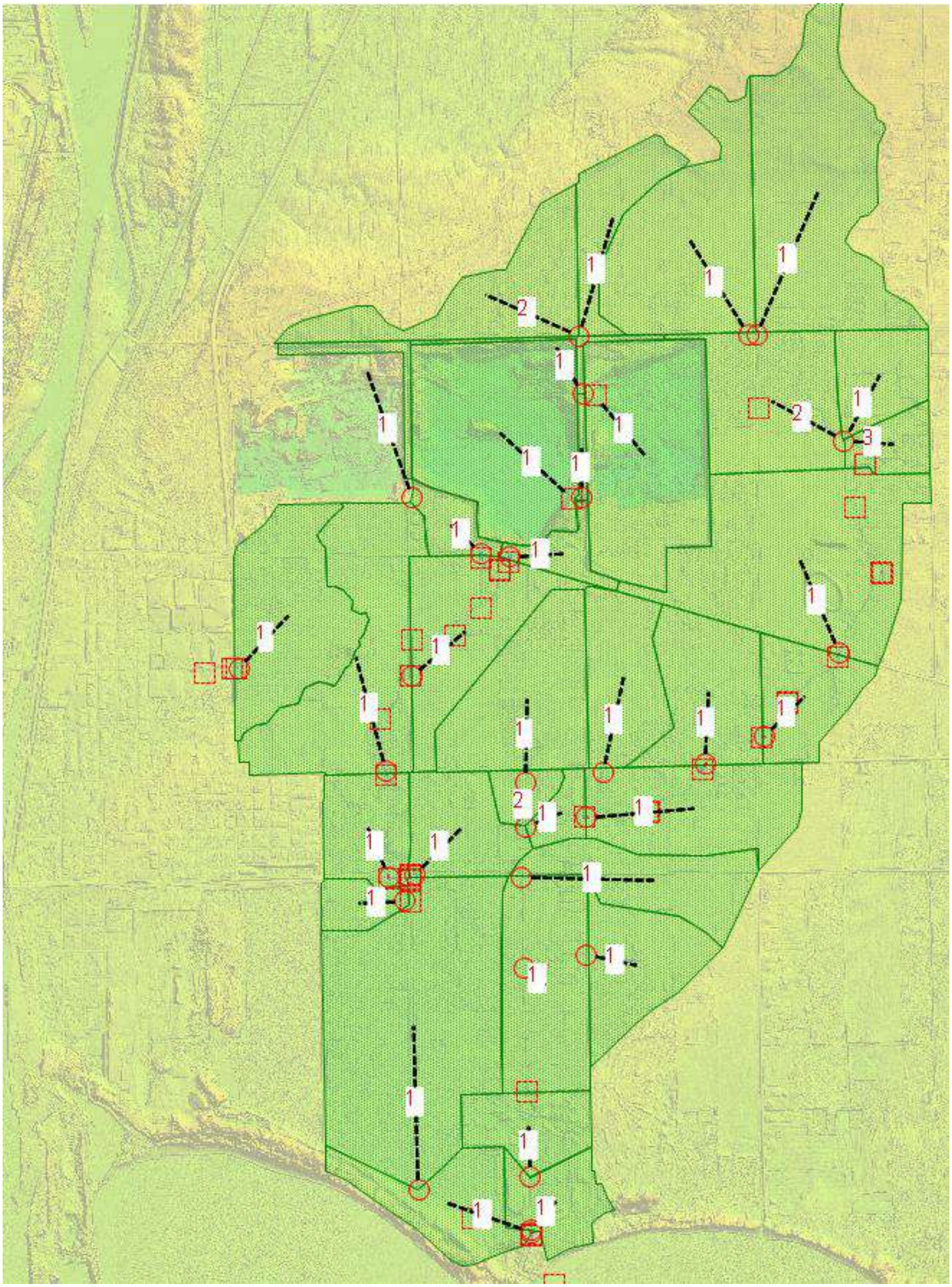
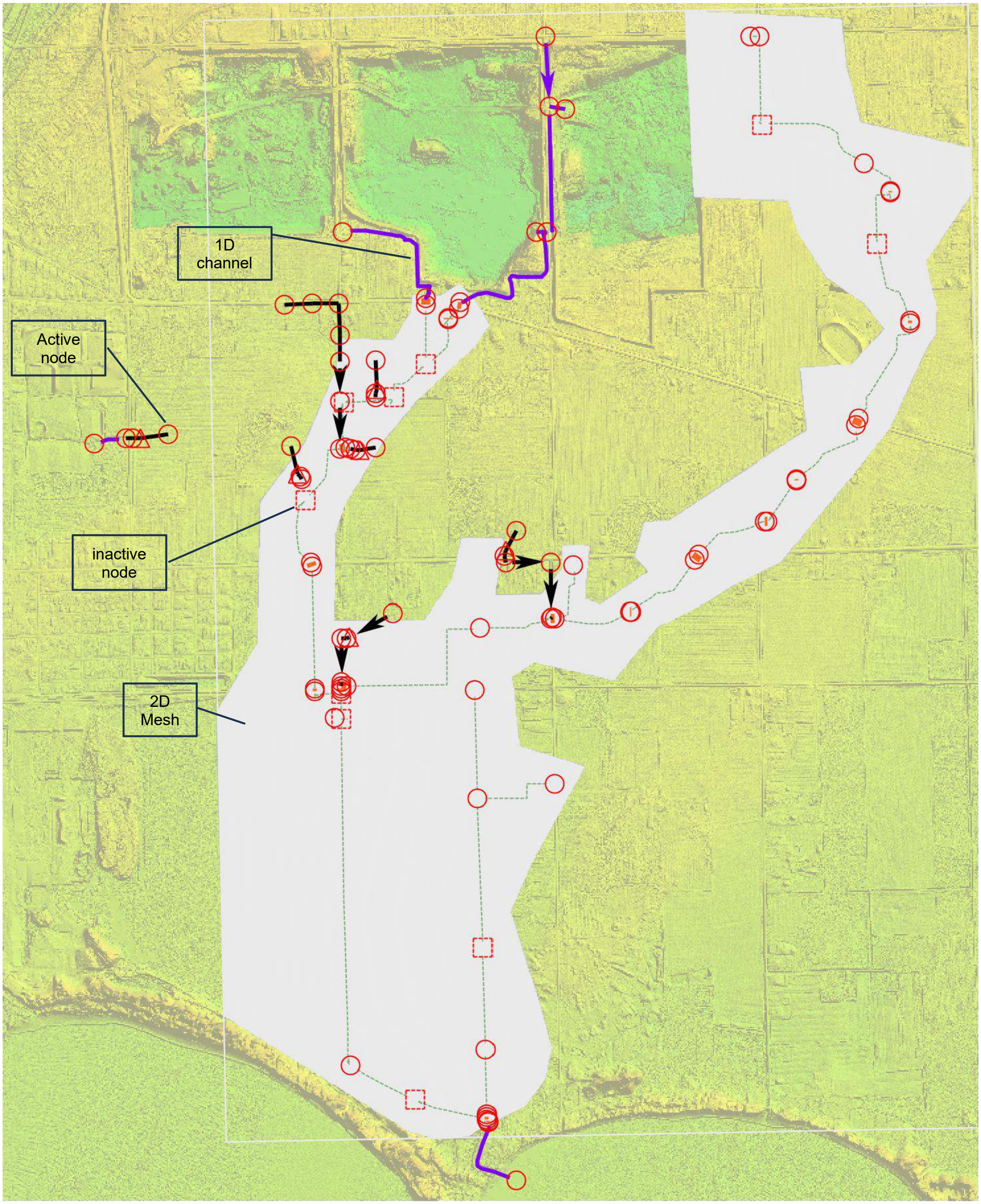


Figure 20 – XPSWMM Global Site Developed model



Legend: Δ - storage node, o –node/MH, Link (pipe), dummy link, inactive node, Link culvert, Link open channel, Link orifice and or weir

Figure 21 – XPSWMM Site Developed model showing Tributary areas and node links

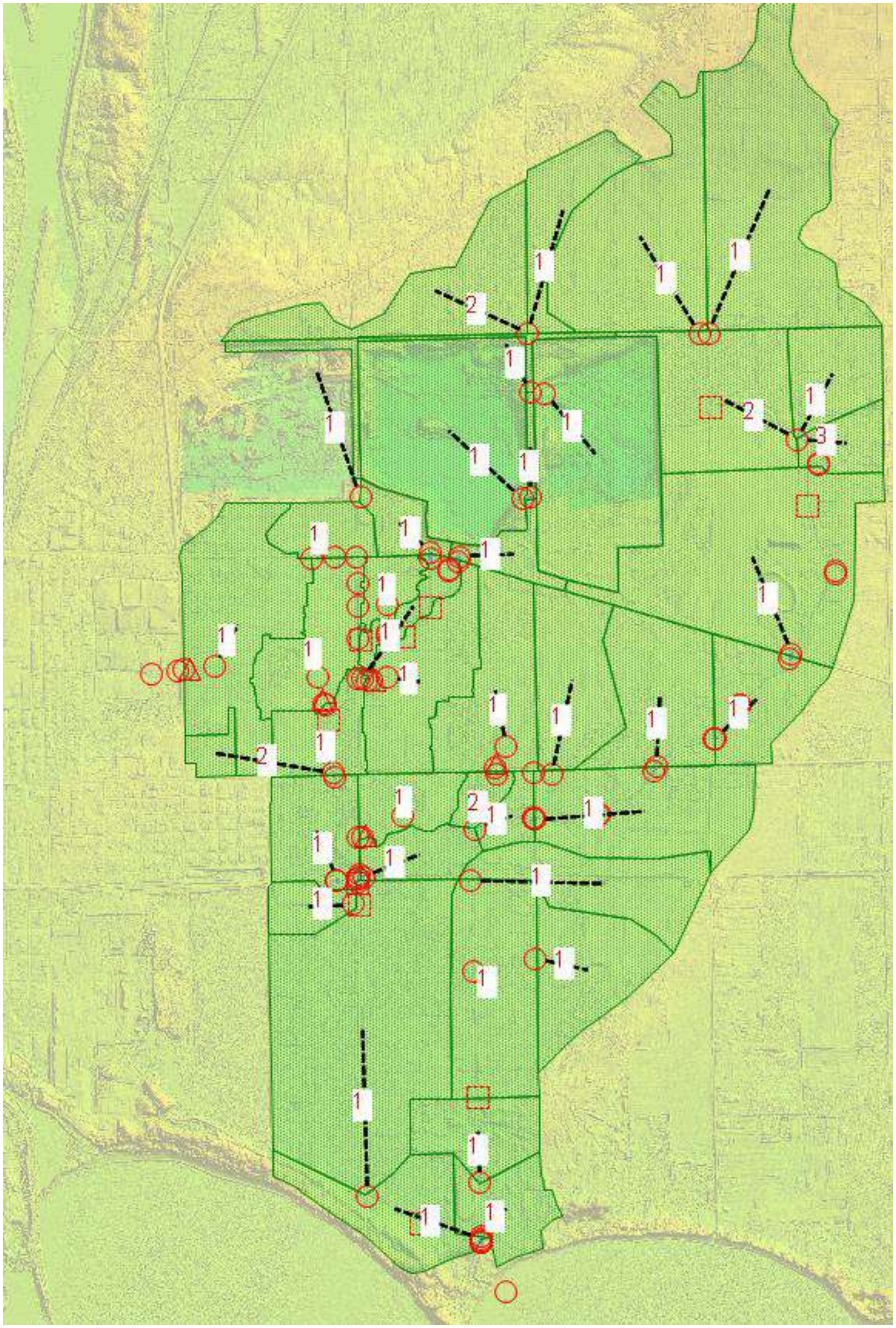
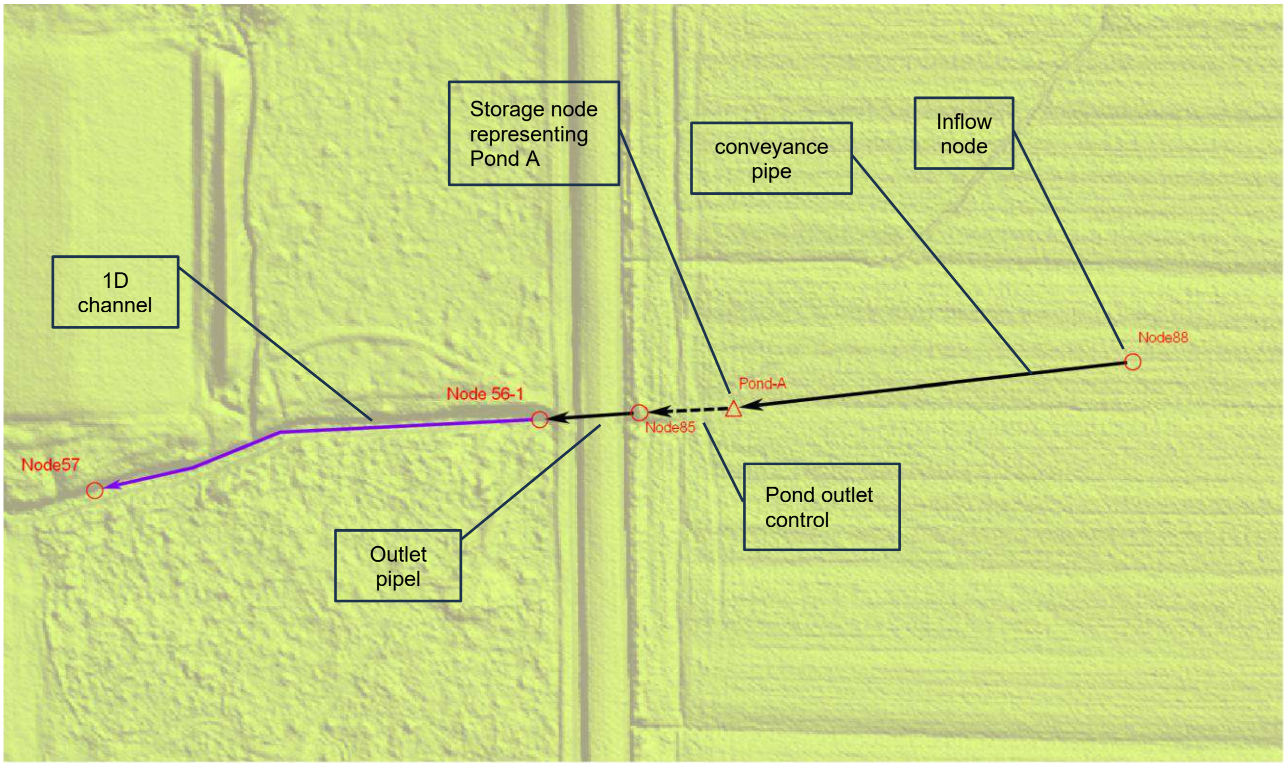
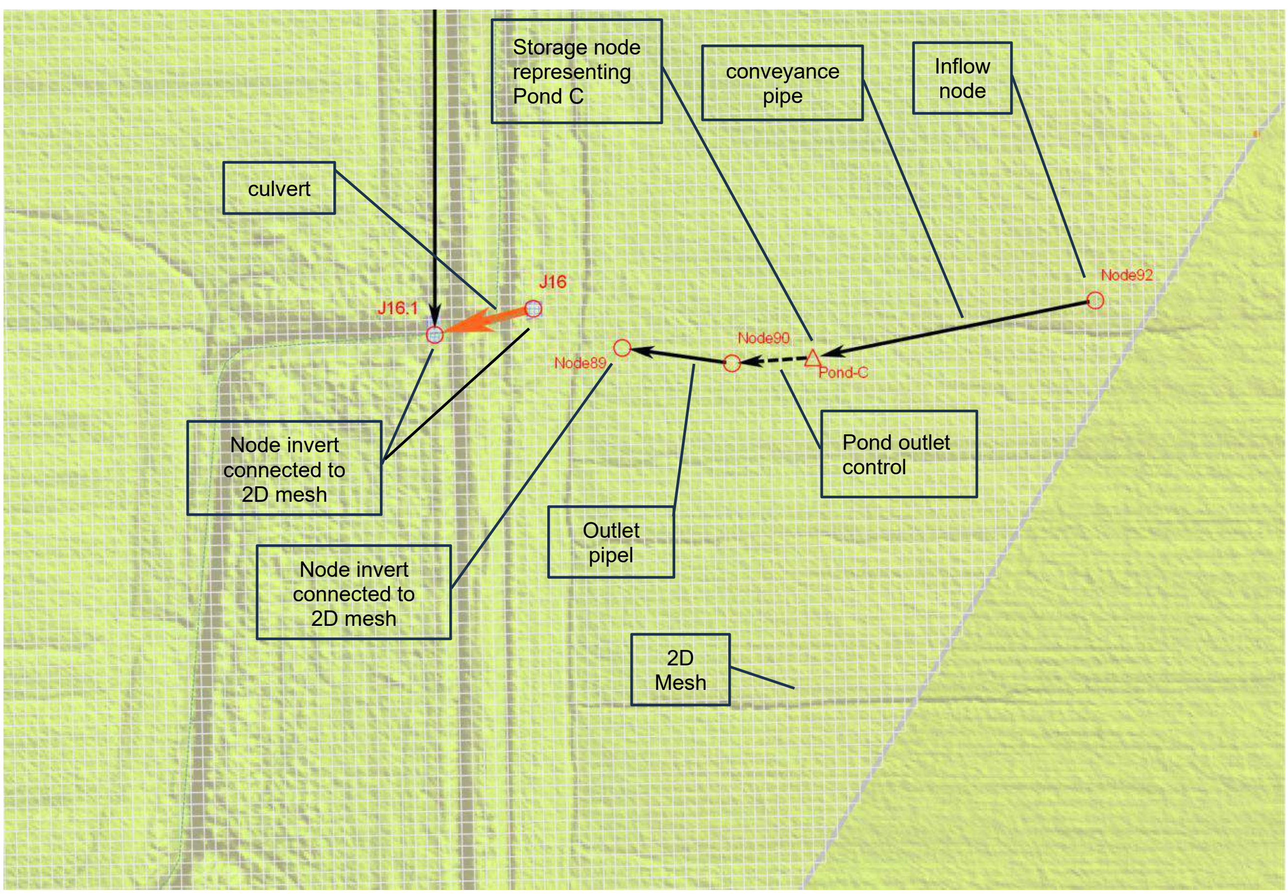


Figure 22 – Close up of urbanized XPSWMM Site Developed model





DISCUSSION OF XP2D MODEL:

The following items are in no particular order.

1) The PCSWMM model by EWA was utilized in the XPSWMM model. The red square nodes shown in Figure 18 above are nodes from PCSWMM representing the nodes at the end of the links. In XPSWMM we replaced the Wignell Drain links with a 2D mesh. Thus, the red squares are the XPSWMM nodes turned off. In addition, the links were turned off. Only select channels and culvert crossings were modelled.

2) The hydrology nodes (where run-off is directed) are linked to 2D mesh if the runoff is in the valley area (Wignell drains). If connected to pipes, they are not linked to 2D mesh.

3) The pond outlet nodes for ponds B to F are linked to the 2D mesh at the creek outfall. Pond A has no 2D mesh.

4) To accept flow from a 1D domain to a 2D domain or vise versa the nodes must be linked to the 2D mesh.

5) Culvert crossings utilize entrance loss of 0.5 and exist loss of 1.0.

6) Culvert crossing utilize manning n as per the NPCA HEC-RAS model (variable with depth)

7) HGL levels at select locations are retrieved via points 1 to 13. The points are in identical locations in the existing and post models.

8) There are no celerity issues with the model.

9) The continuity error is excellent.

10) There were no error messages from the analysis.

11) The HGL maximum at any given point in the system is the most useful result. The animation of the flow with time, best shows how the upper flows are attenuated through the system and the effect of downstream tail water if it exists.

12) Hydro-dynamic models provide the most accurate, reliable and defensible representation of flows in the collection system. They account for varying inflows, non-coincident peak flows, in system storage, hydrograph attenuation, and tail and backwater effects. In addition, the integration with a 2D model allows accurate spill over calculations of the Flood plain along with the attenuation effects of the spill.

13) **The following are the key items affecting HGL in dynamic models:**
   - pipe and or channel volume,
   - length of flow, spill and volume in the spill area,
   - runoff hydrograph (length in time and volume),
   - timing of peaks which are controlled by the above.

14) The analysis is based on clean channels. No percentage blocked.

MODELLING RESULTS:

FLOOD SCENERO (Peak Flows)

The following Table 7 is a summary of the storage volumes required in the proposed ponds to control the 100-year storm to pre-development levels. Conceptual pond designs for each pond showing conceptual elevations can be referenced on Figures S-9 to S-16 in Appendix D.

Table 7. Required Storage Volumes Site Developed

BLOCK ID	Upstream Contributing Area (ha)	100 Year Storage Required (m3)	100 Year HGL - in pond (m)	100 Year Storage Provided at 2.0 m depth (m3)	Top of Pond elevation (m)	Pond Freeboard (m)
Pond A	50.173	27,224	178.21	46,716	178.80	0.59
Pond B	24.43	14,424	178.99	19,039	179.30	0.31
Pond C	31.92	18,556	179.37	23,932	179.66	0.30
Pond D	33.05	18,974	178.25	32,063	178.85	0.60
Pond E	12.72	8,119	178.75	12,994	179.07	0.32
Pond F	7.87	6,292	179.45	9,732	179.80	0.35

Refer to Table 8 below for the comparison of peak outflows for the ponds. Note, existing is the target flow from the XPSWMM existing model (see summary of pond target flows) while the developed heading is the XPSWMM post model results from the ponds.

Table 8. Comparison of Pre-Development and Developed Site Pond flows.

Storm Event	Storm Type	**Peak Flow Rate (m^3/s)**											
		Pond A		Pond B		Pond C		Pond D		Pond E		Pond F	
		Existing Target flow	Developed	Existing Target flow	Developed	Existing Target flow	Developed	Existing Target flow	Developed	Existing Target flow	Developed	Existing Target flow	Developed
2 Year	24Hr SCS	0.085	0.083	0.118	0.075	0.154	0.136	0.159	0.102	0.061	0.038	0.038	0.005
5 Year	24Hr SCS	0.192	0.113	0.224	0.109	0.293	0.277	0.304	0.165	0.117	0.103	0.072	0.007
10 Year	24Hr SCS	0.299	0.196	0.314	0.155	0.411	0.353	0.425	0.268	0.164	0.130	0.101	0.020
25 Year	24Hr SCS	0.467	0.374	0.451	0.237	0.589	0.438	0.610	0.424	0.235	0.162	0.145	0.022
50 Year	24Hr SCS	0.603	0.526	0.567	0.281	0.741	0.489	0.768	0.525	0.296	0.184	0.183	0.058
100 Year	24Hr SCS	0.751	0.661	0.611	0.317	0.798	0.532	0.827	0.626	0.318	0.204	0.197	0.043
100 Year	12Hr AES	0.728	0.712	0.510	0.331	0.666	0.532	0.689	0.628	0.265	0.203	0.164	0.043

SCENARIO 1 - The following Figures 23 to 28 depict the modelling and mapping for the existing conditions (Base) for 2 to 100-year storm event.

Figure 23 - XPSWMM depth MAP (100year) - site undeveloped existing flows

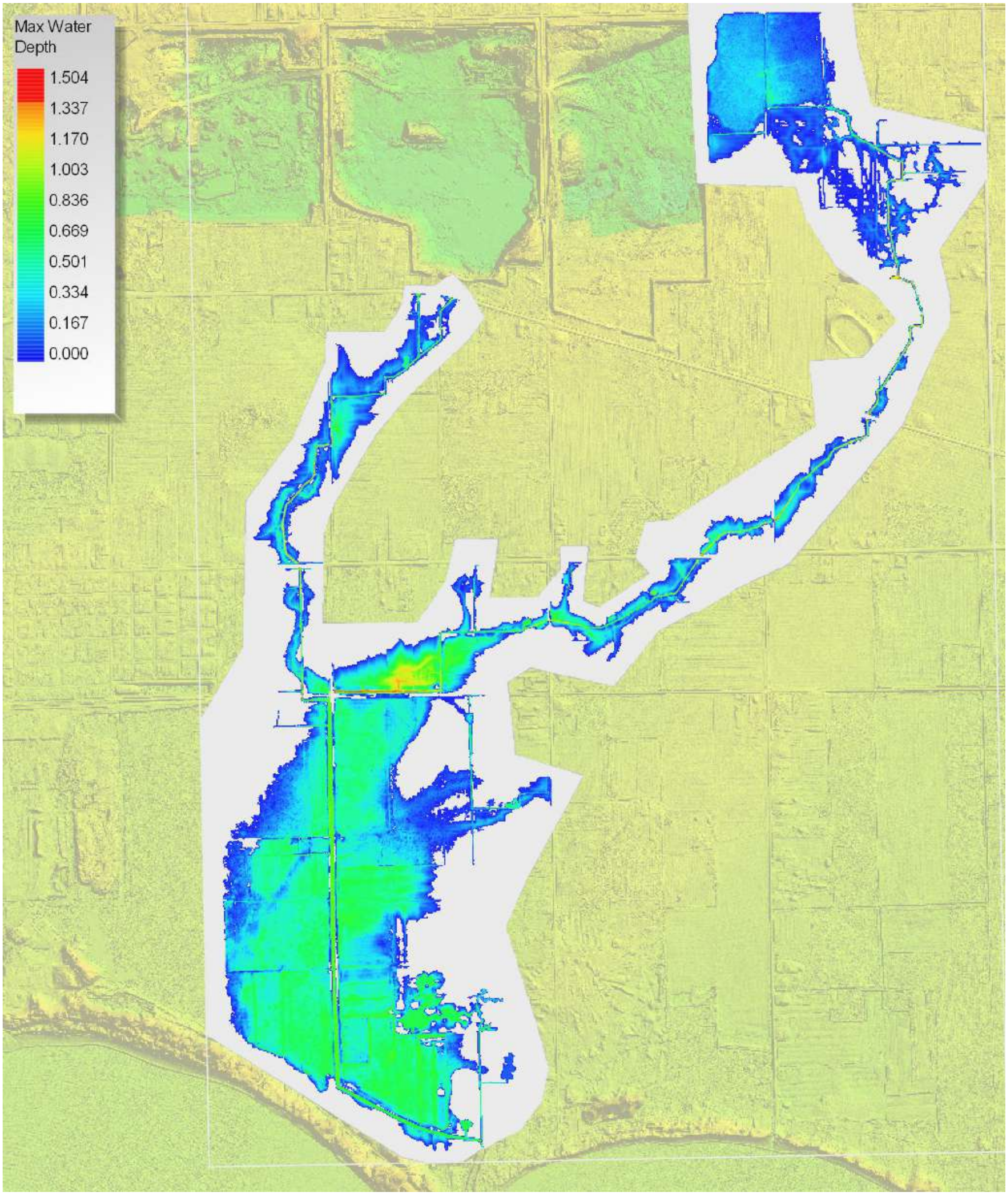


Figure 24 - XPSWMM depth MAP (50year) - site undeveloped existing flows

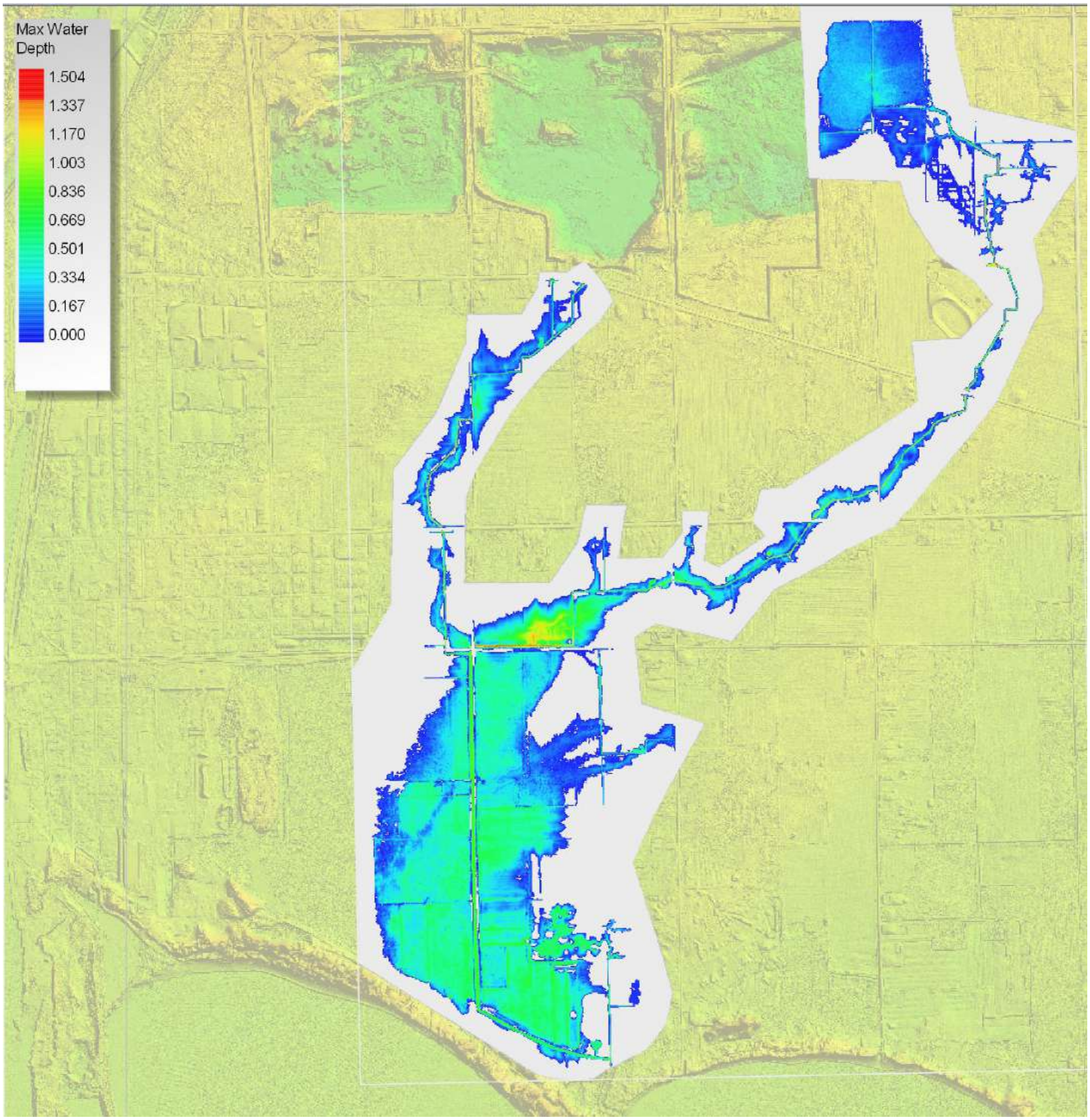


Figure 25 - XPSWMM depth MAP (25year) - site undeveloped existing flows

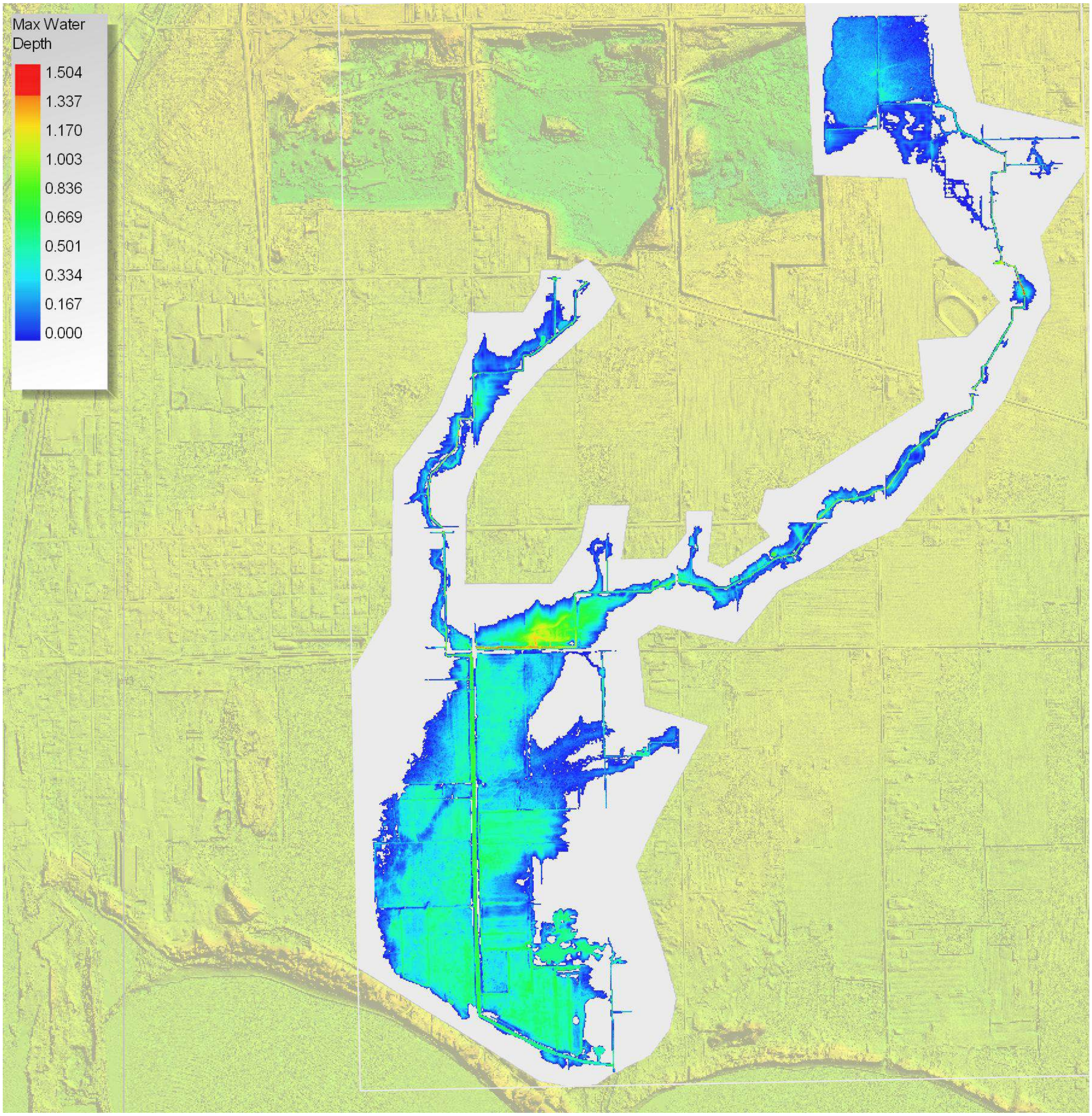


Figure 26 - XPSWMM depth MAP (10 year) - site undeveloped existing flows

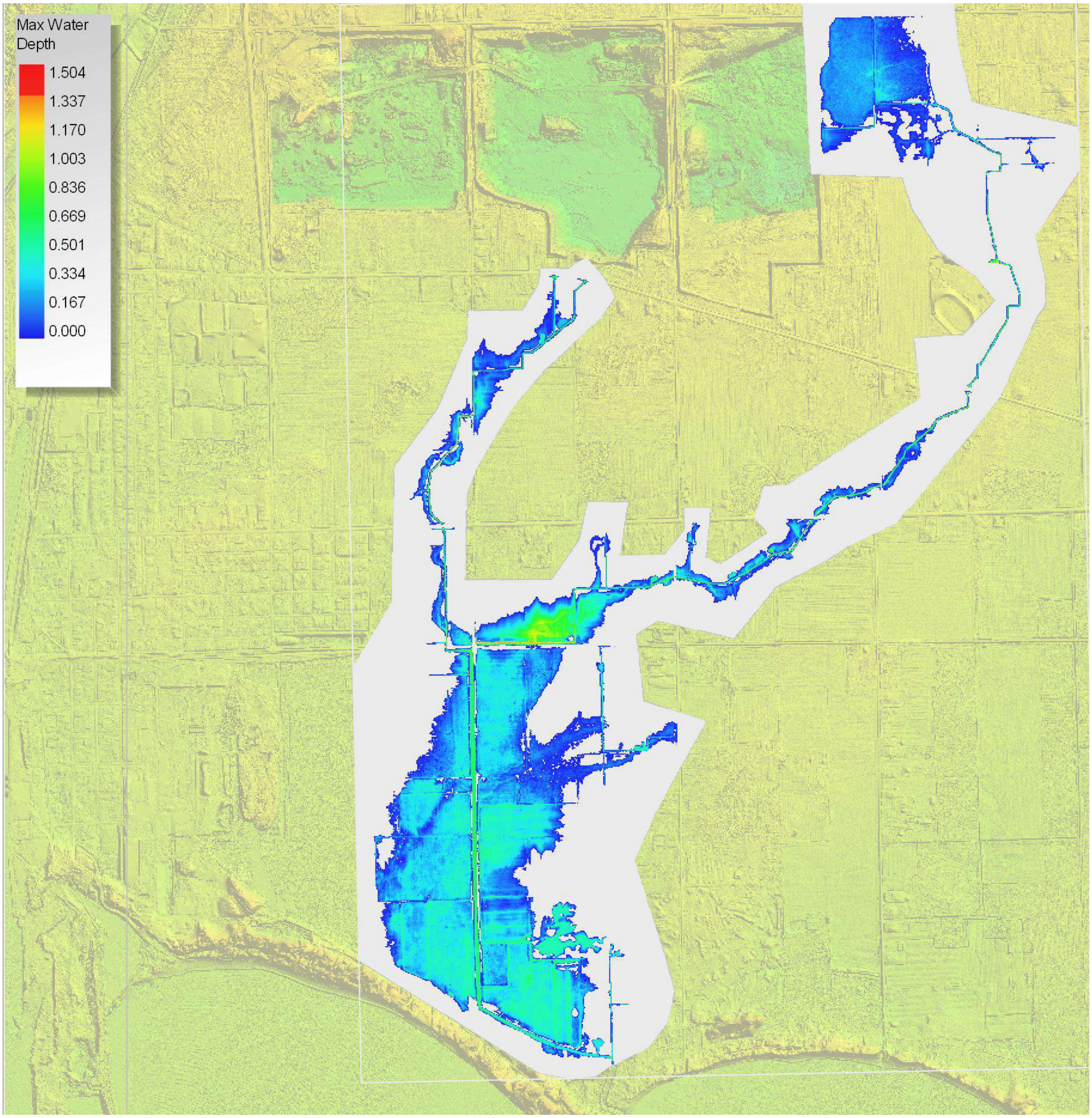


Figure 27 - XPSWMM depth MAP (5 year) - site undeveloped existing flows

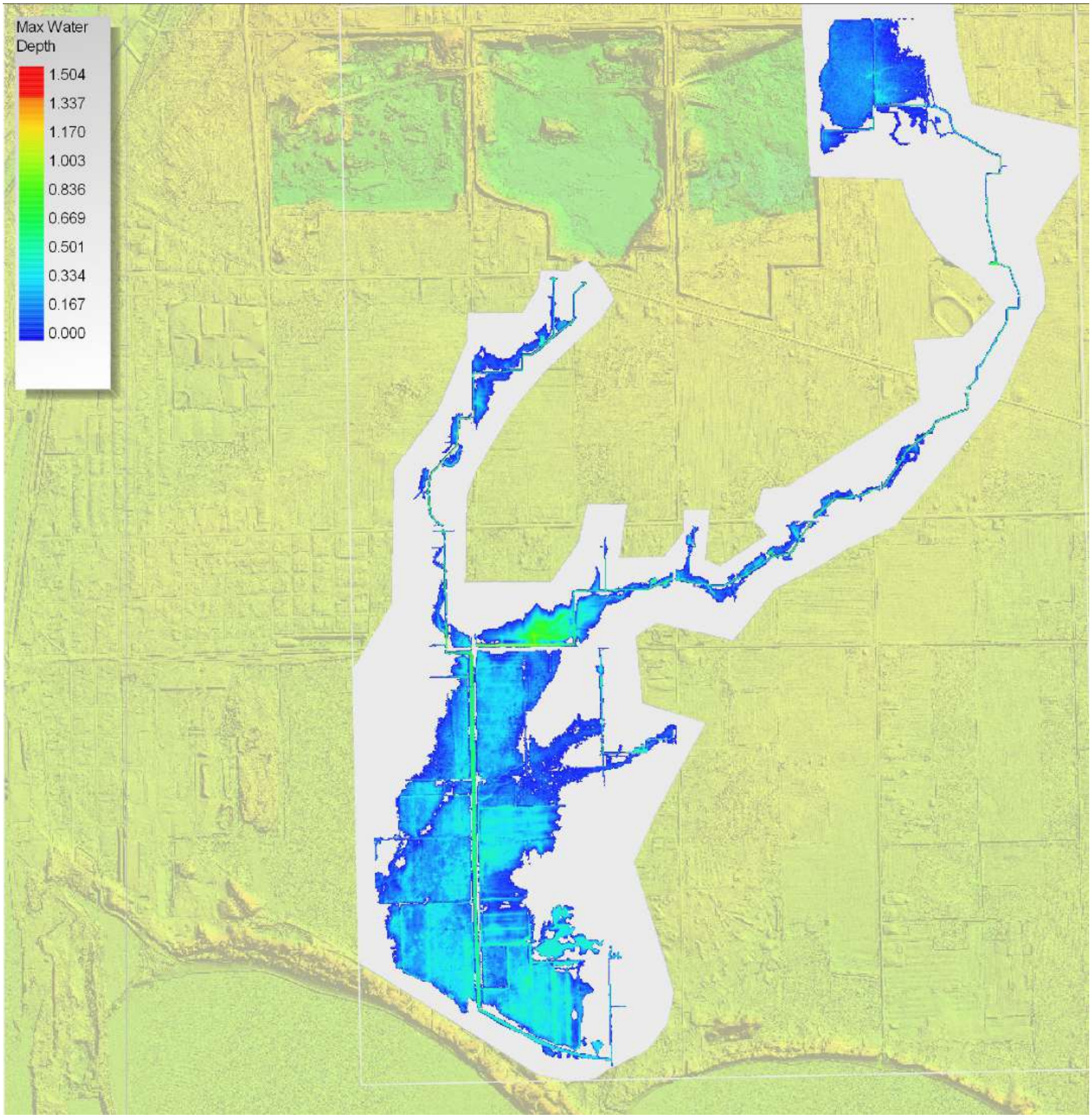
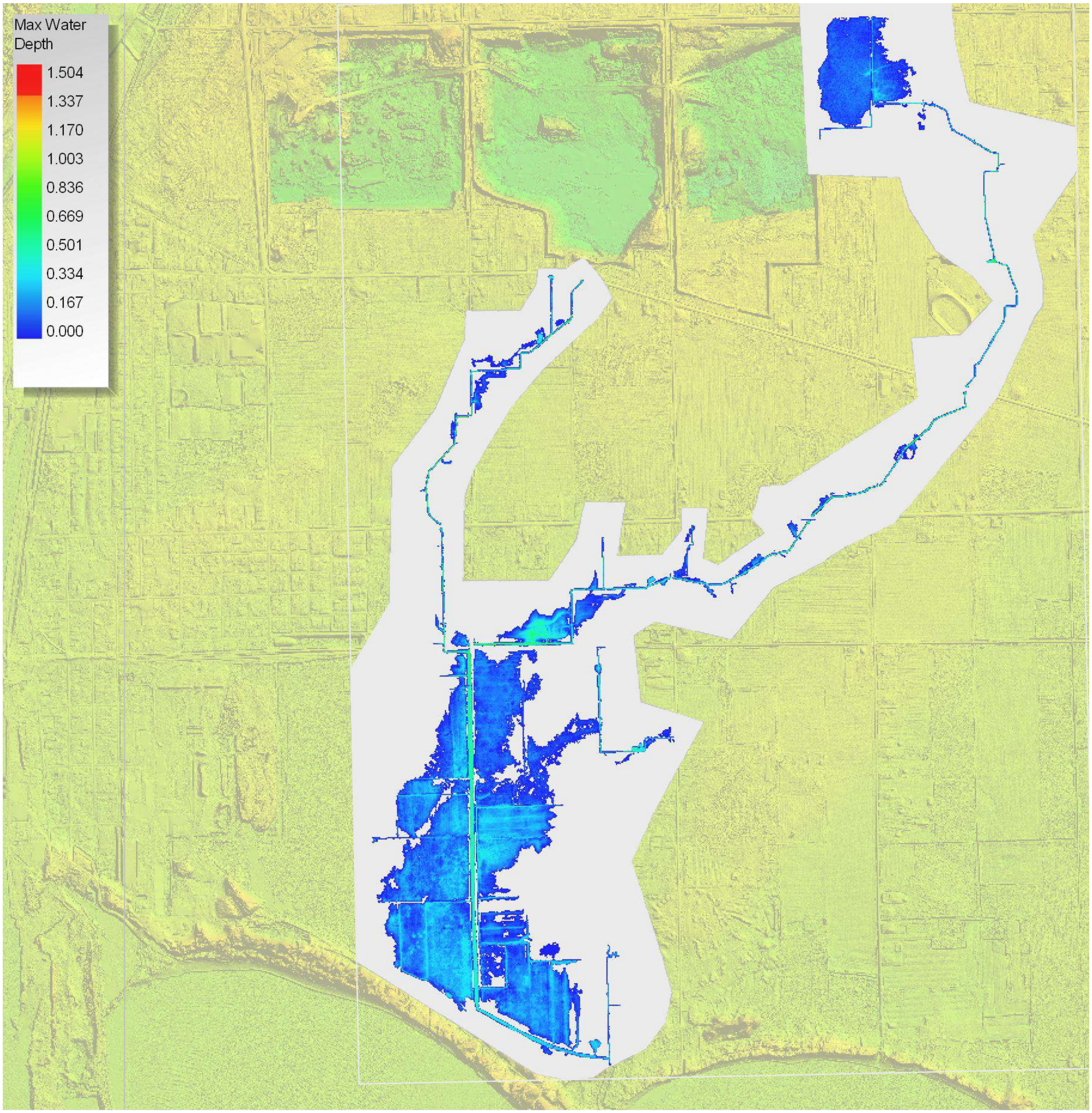
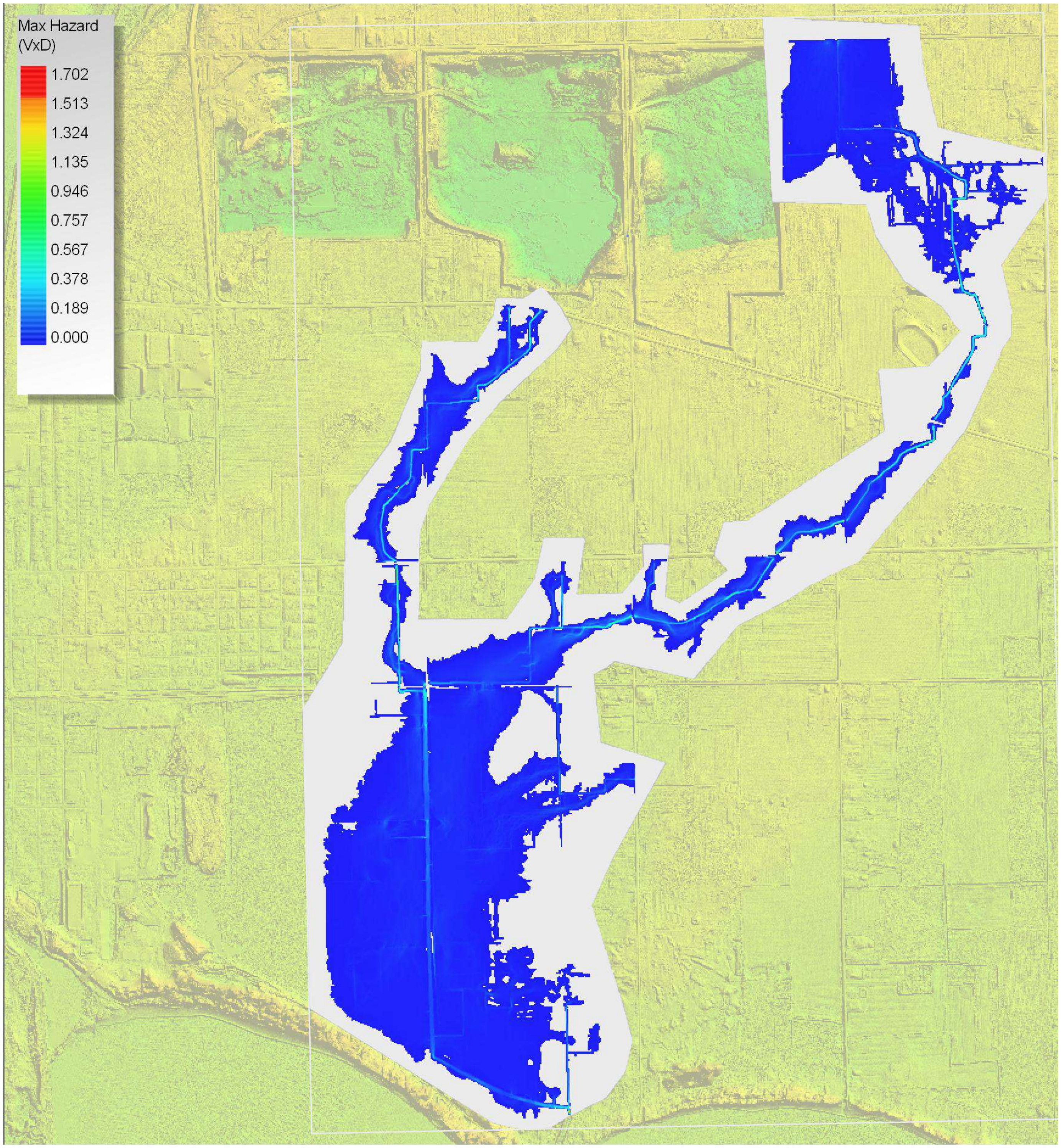


Figure 28 - XPSWMM depth MAP (2 year) - site undeveloped existing flows



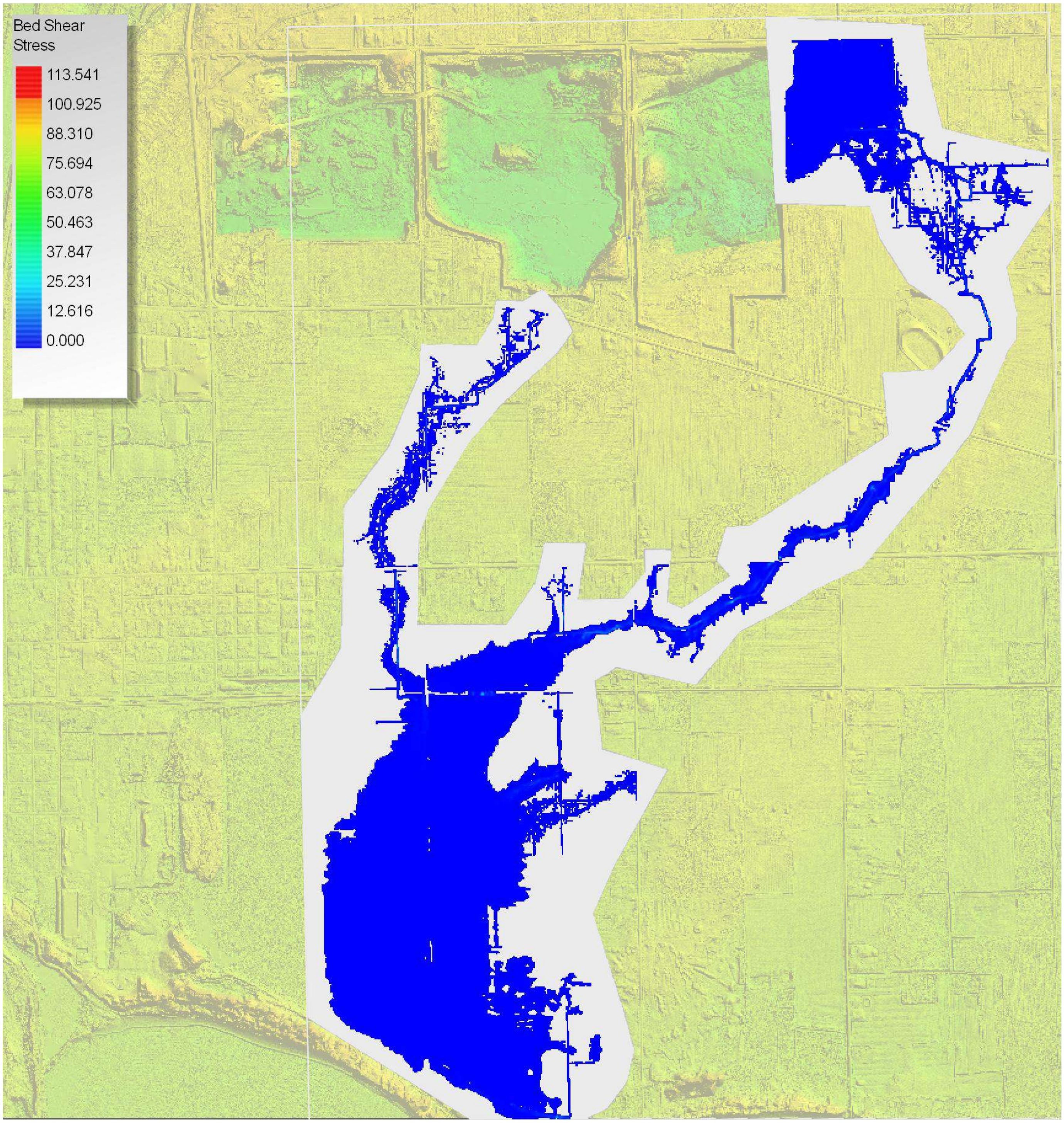
The hazard map shown on Figure 29 is a visualization of hazard which is a quotient of velocity and depth for each grid cell used to assess risk. Please note that all hazard areas are outside the proposed urban boundary of the subject site and are contained within the floodplain areas.

Figure 29- XPSWMM hazard MAP (100year) - site undeveloped existing flows



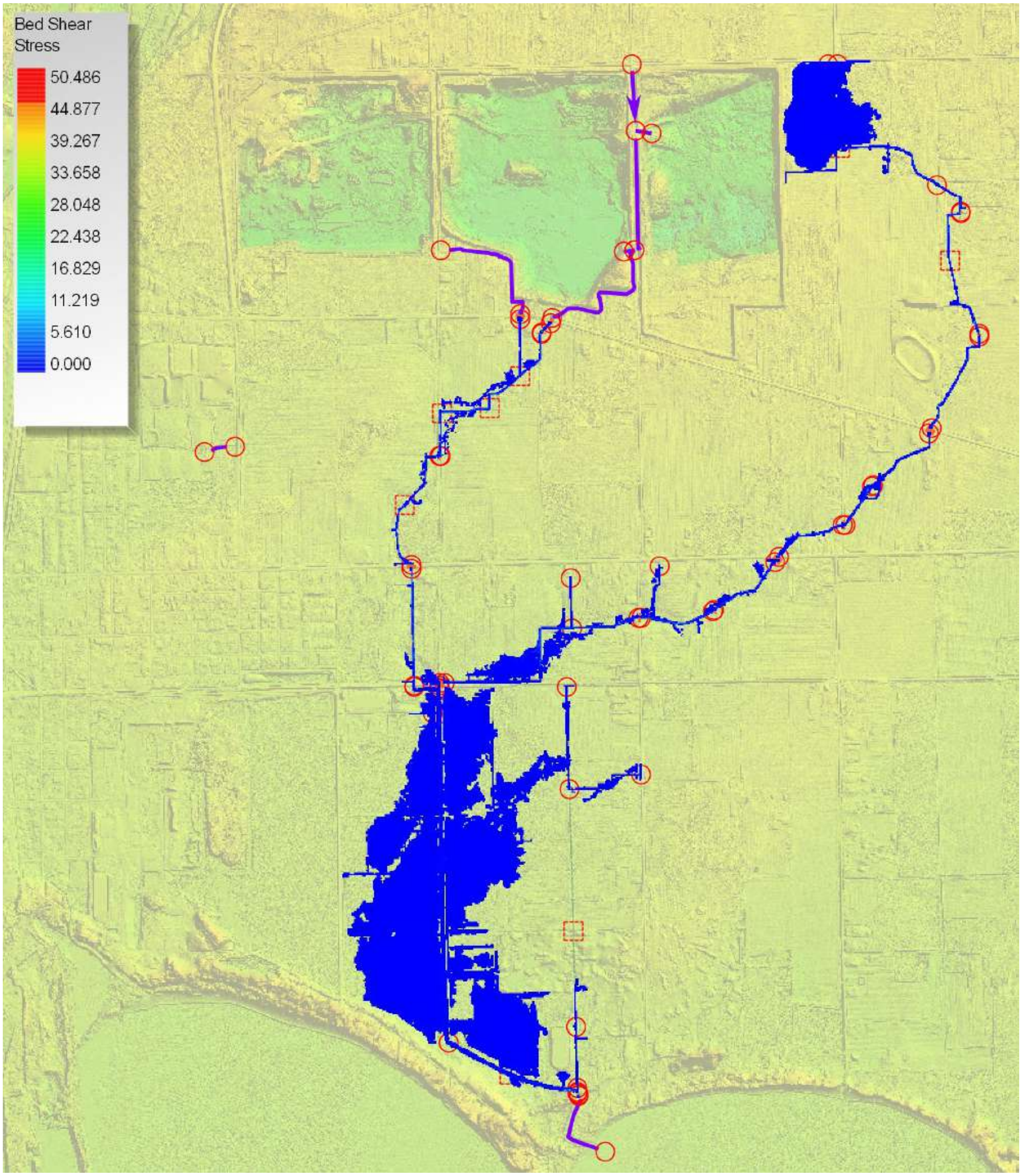
The bed shear maps shown on Figure 30 and 31 are used to assess erosion potential for frequently occurring storms such as the 2-year storm. Note that Figure 30 shows the existing 100-year shear bed map which is less than the allowable for the vegetation in situ. See the erosion review on pg 69 for further information on bed shear and erosion.

Figure 30 - XPSWMM bed shear MAP (100 year) - site undeveloped existing flows



The above map is the 100-year bed shear. The maximum is 114 Pa. anywhere in the system.

Figure 31 - XPSWMM bed shear MAP (2 year) - site undeveloped existing flows



The above map is the 2-year bed shear. The maximum is 50 Pa. anywhere in the system.

SCENARIO 2 – The following Figures 32 to 37 depict the modelling and mapping for the proposed conditions (site developed) for 2 to 100-year storm event.

Figure 32 - XPSWMM depth MAP (100year) - Site Developed

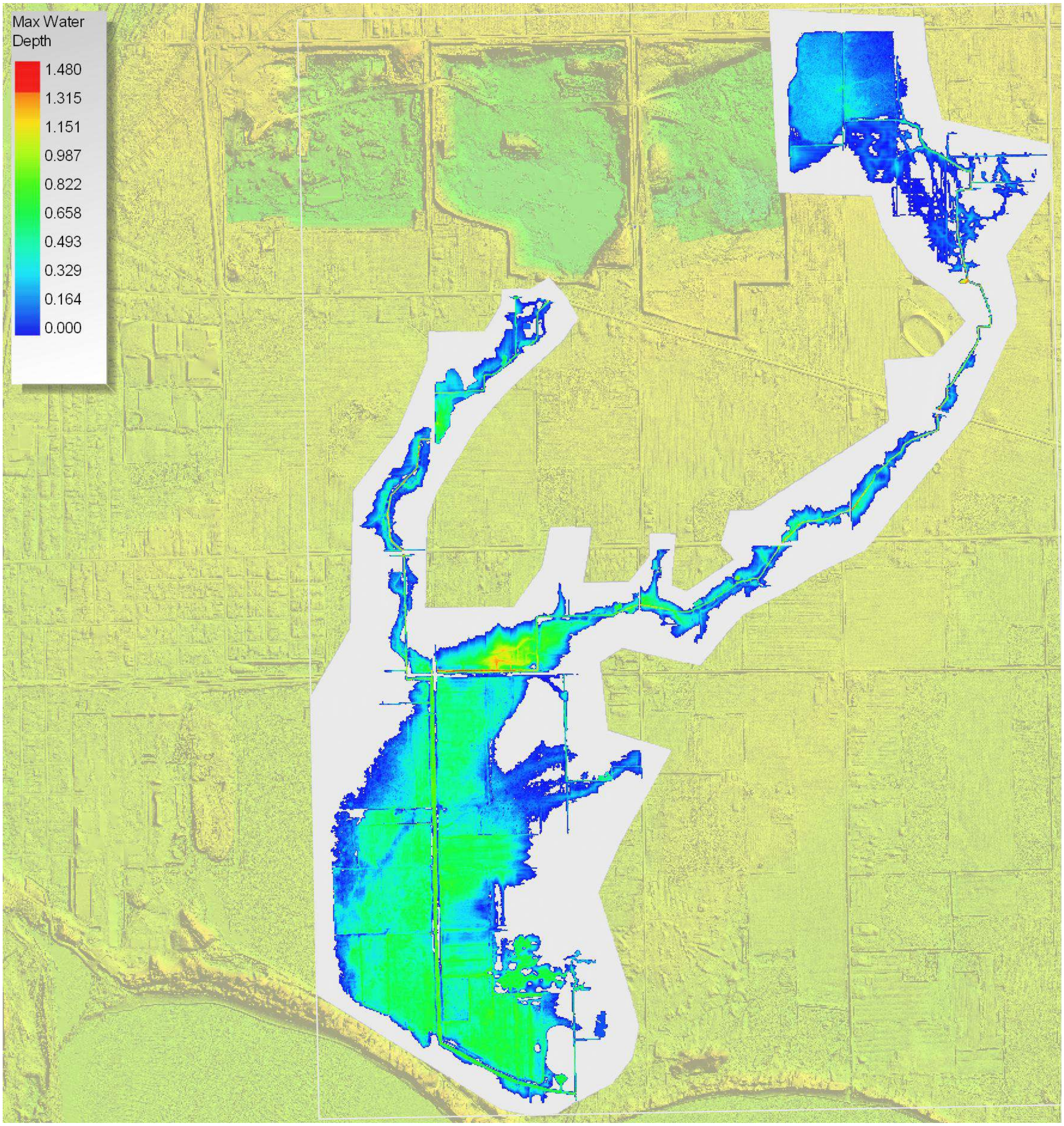


Figure 33 - XPSWMM depth MAP (50year) - Site Developed

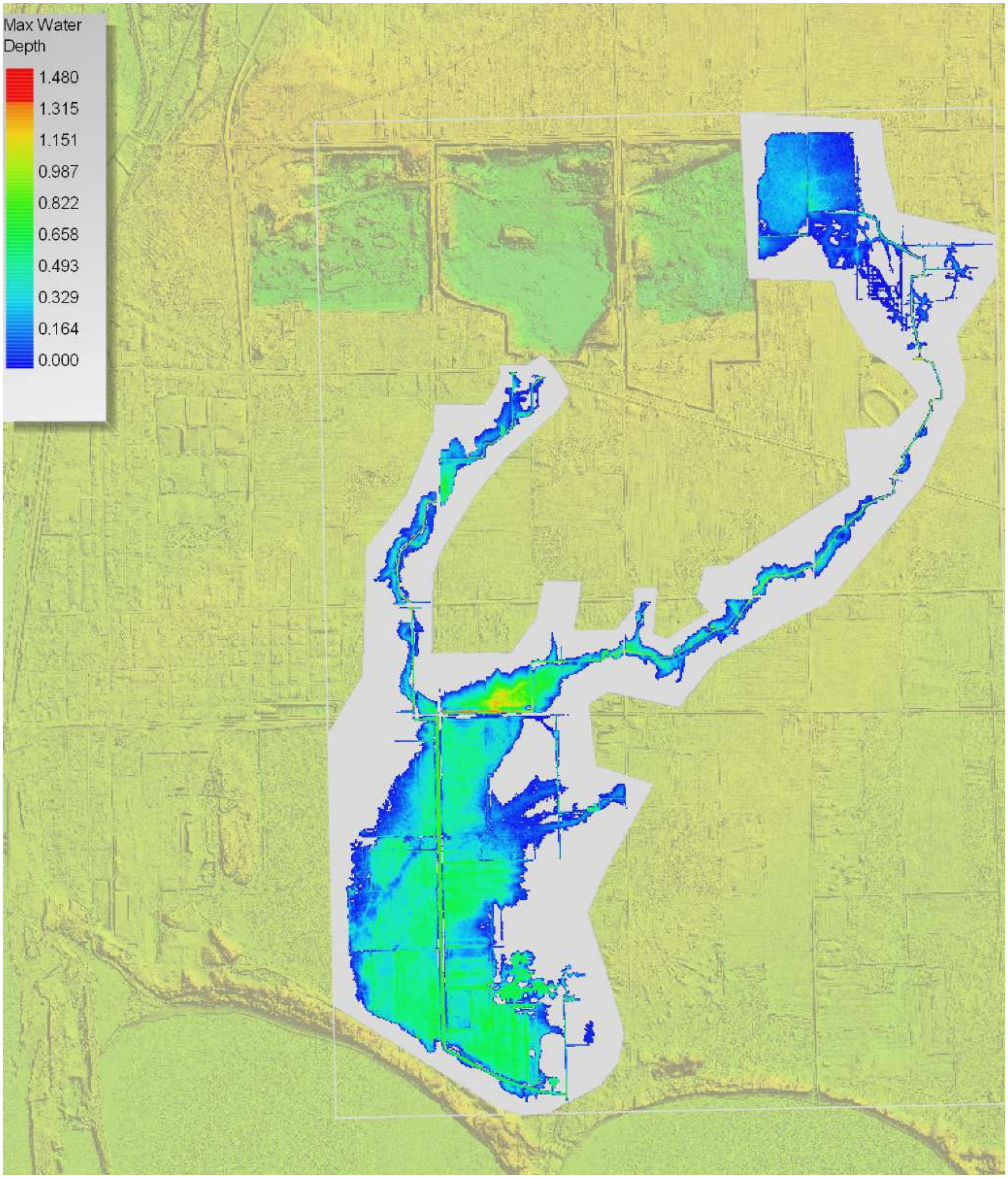


Figure 34 - XPSWMM depth MAP (25year) - Site Developed

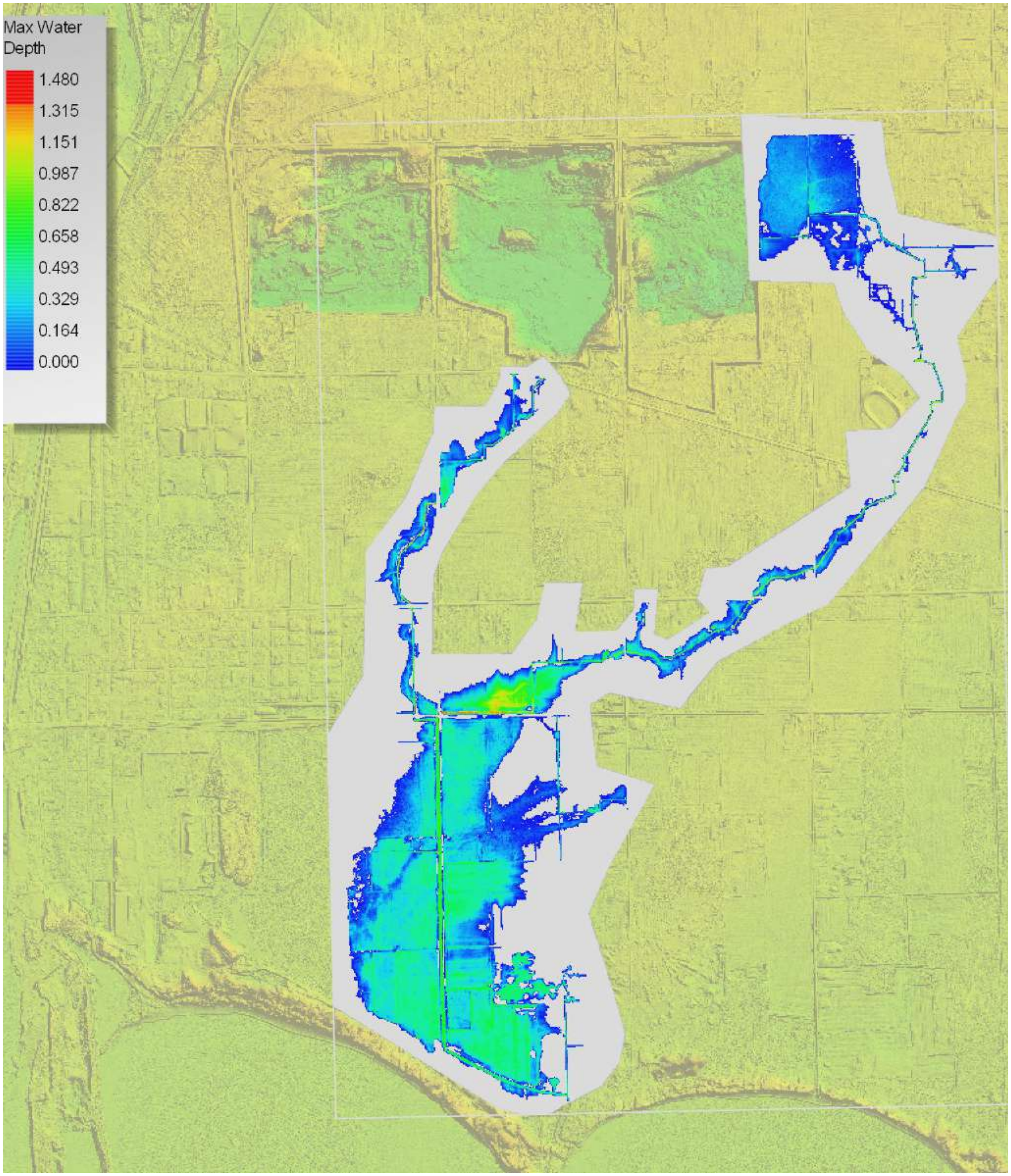


Figure 35 - XPSWMM depth MAP (10 year) - Site Developed

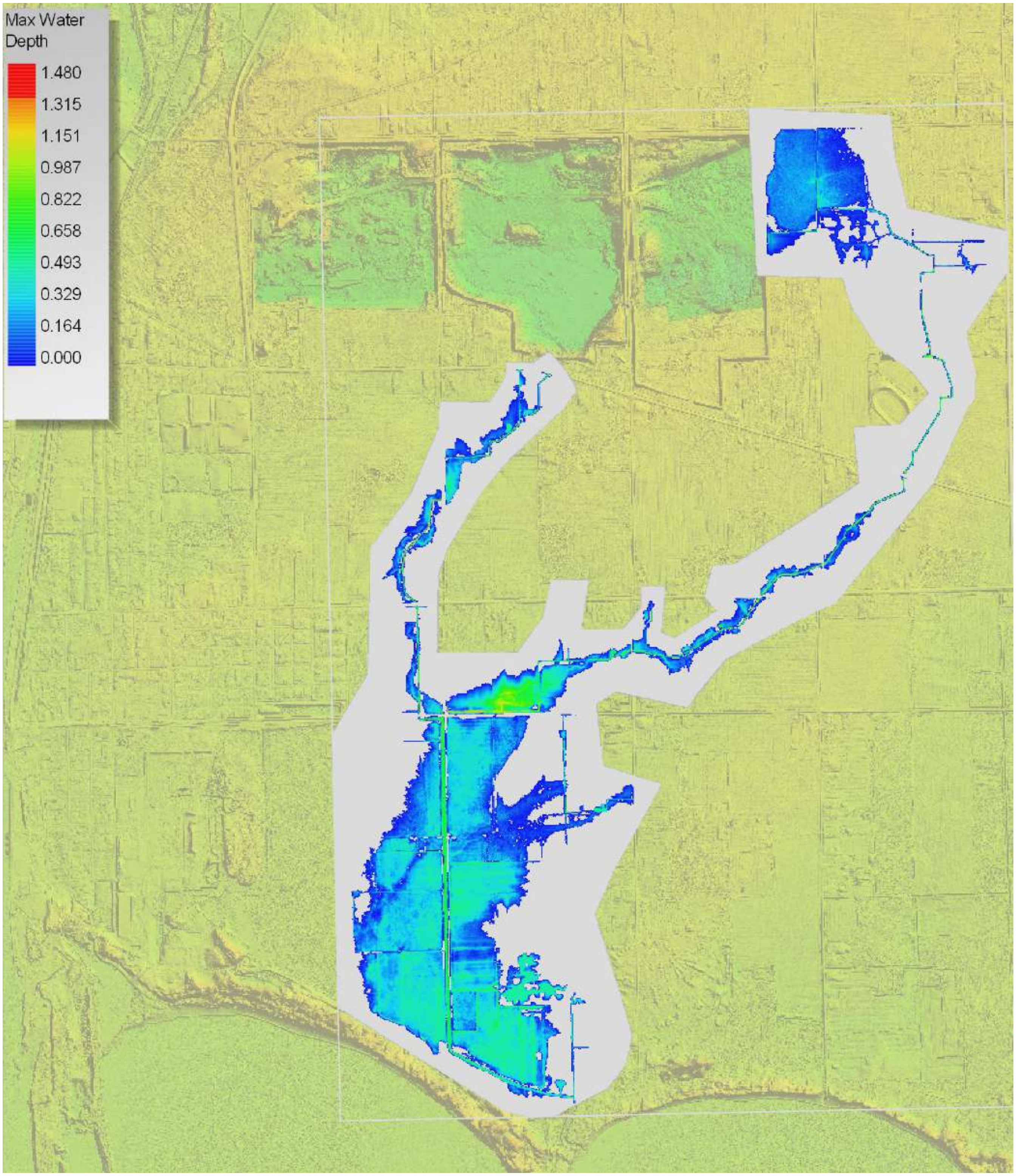


Figure 36 - XPSWMM depth MAP (5 year) - Site developed

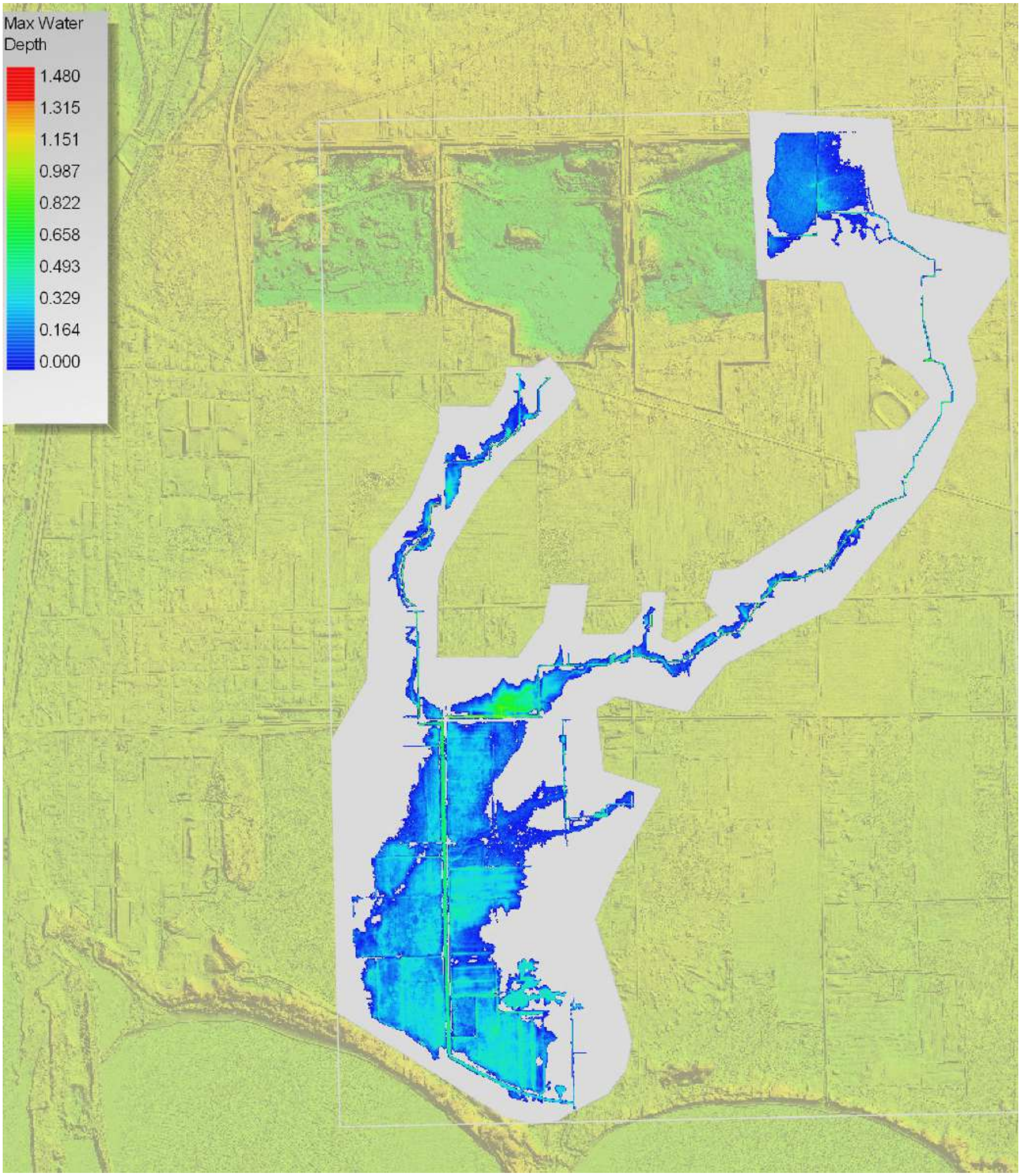
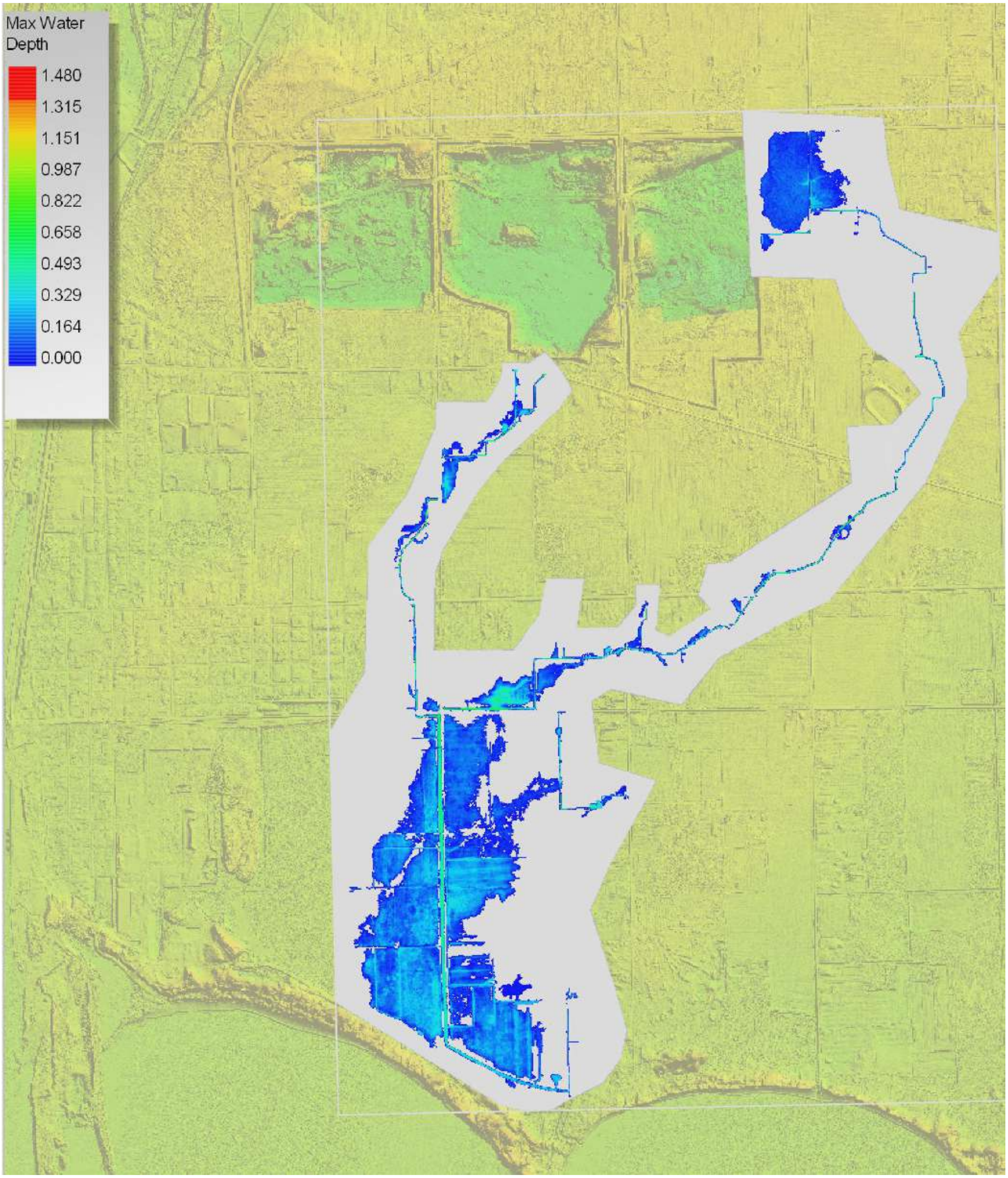
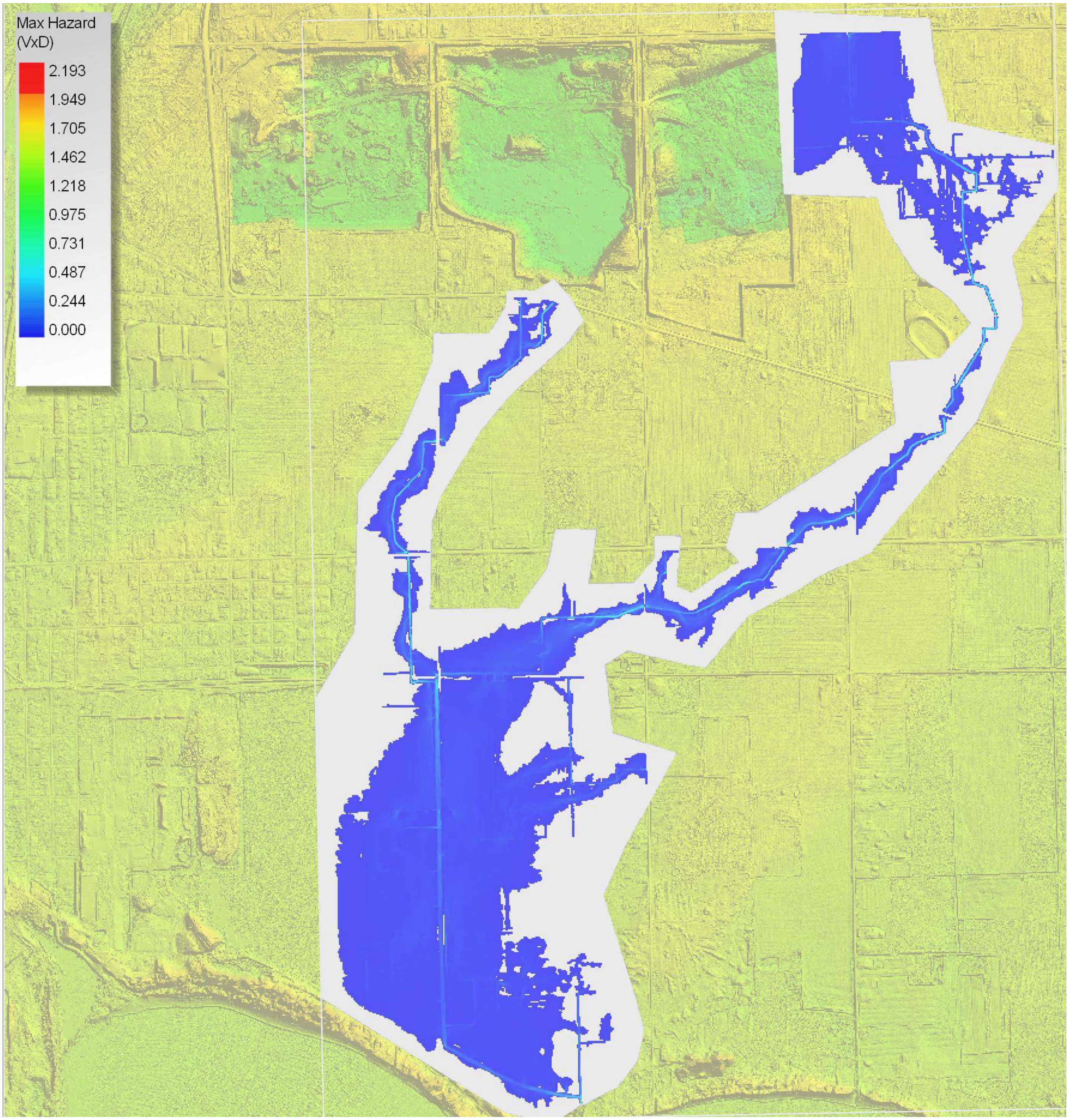


Figure 37 - XPSWMM depth MAP (2 year) - Site developed



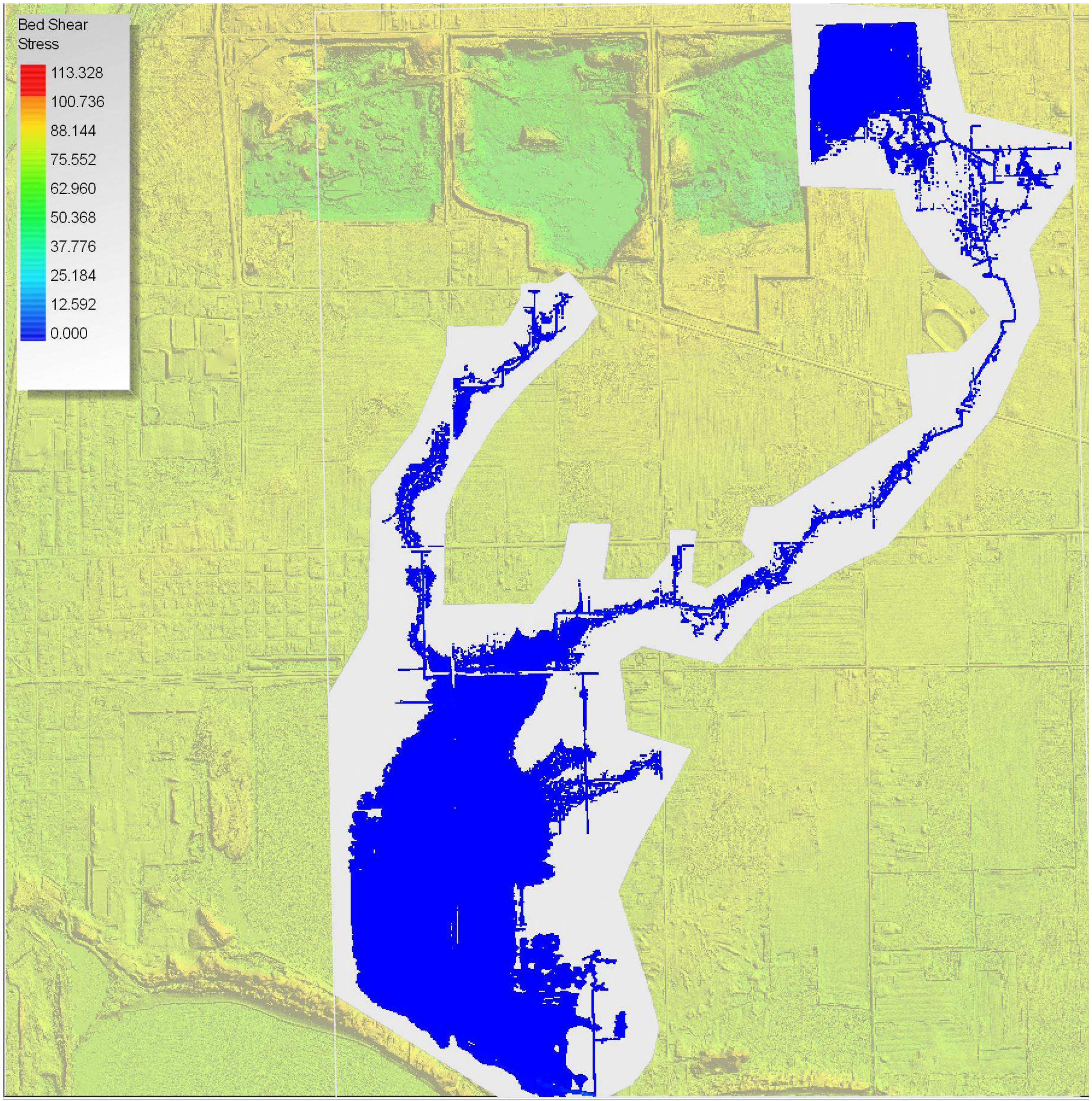
The hazard map shown on Figure 38 is a visualization of hazard which is a quotient of velocity and depth for each grid cell used to assess risk. Please note that all hazard areas are outside the proposed urban boundary of the subject site and are contained within the floodplain areas.

Figure 38 - XPSWMM hazard MAP (100 year) - Site developed



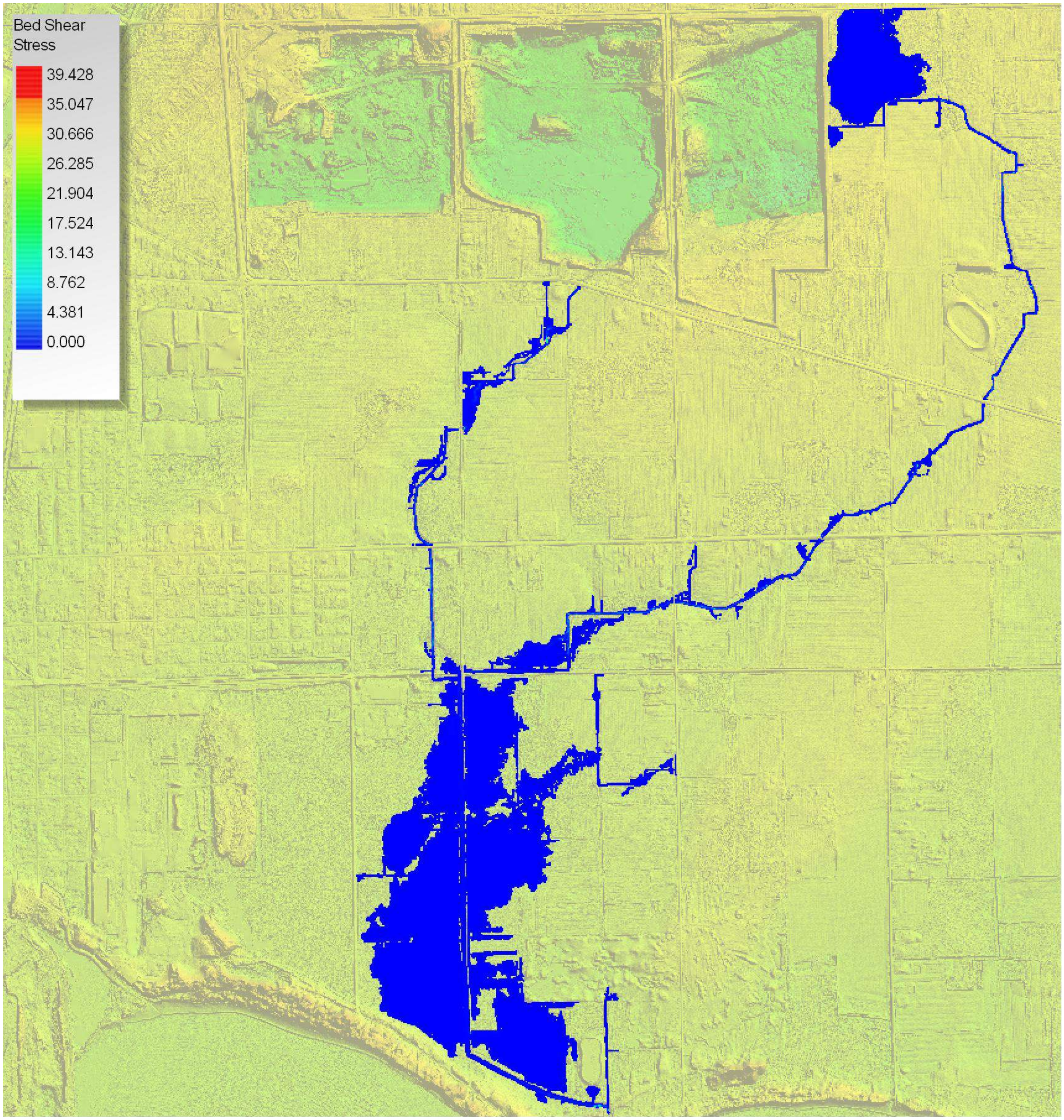
The bed shear maps shown on Figure 39 and 40 are used to assess erosion potential for frequently occurring storms such as the 2-year storm. Note that Figure 39 shows the existing 100-year shear bed map which is less than the allowable for the vegetation in situ. See the erosion review on pg 69 for further information on bed shear and erosion.

Figure 39 - XPSWMM bed shear MAP (100 year) - Site developed



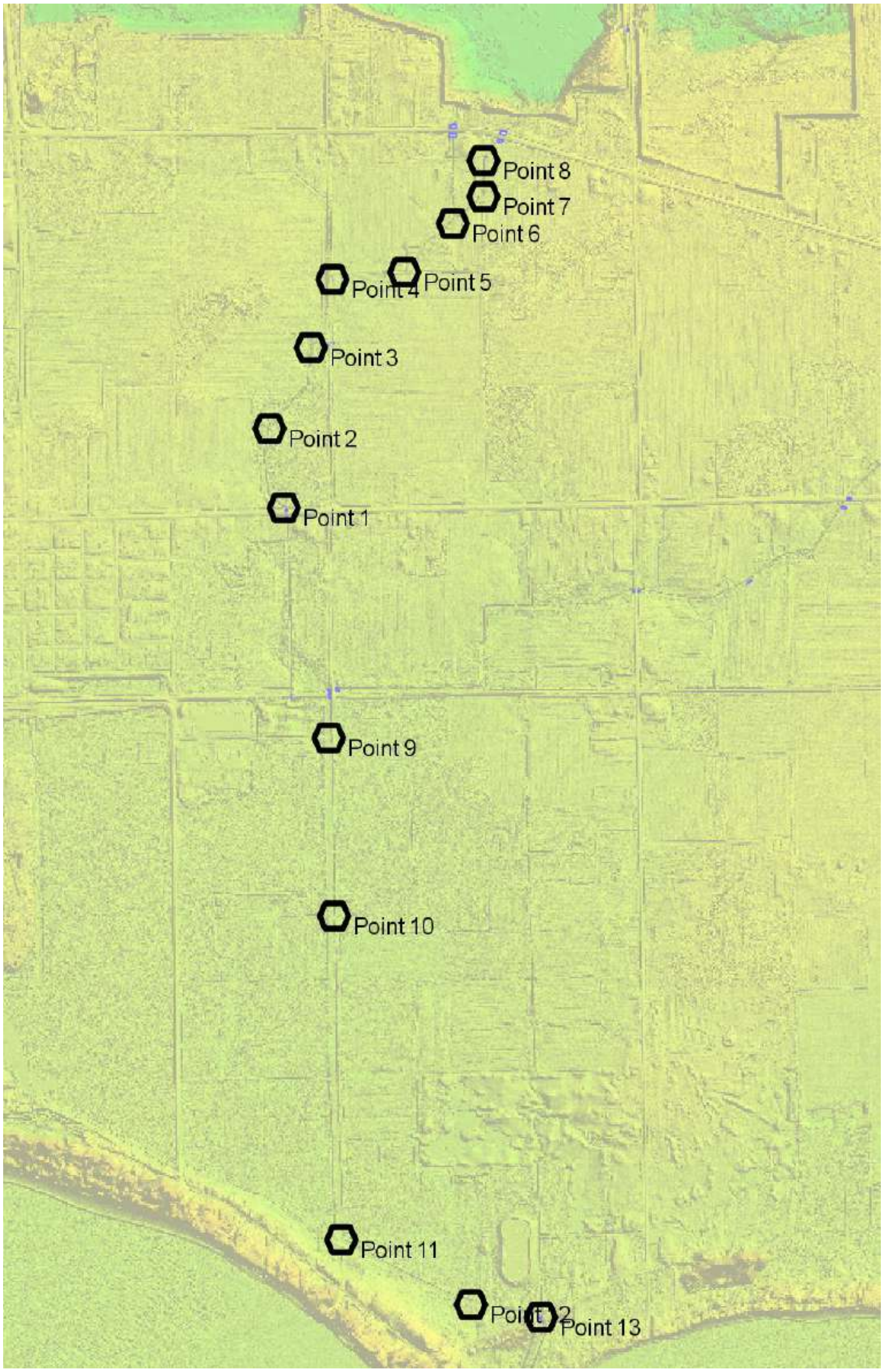
Note, the 100-year maximum in the developed site is less than the existing conditions.

Figure 40 - XPSWMM bed shear MAP (2 year) - Site developed



Note, the 2-year maximum in the developed site is less than the existing conditions.

POINT HEAD PLOT



The points shown above are locations where HGL are retrieved. See Table 9 for points summary. See below for the section plots. It should be noted that the geospatial locations of these points and sections are identical for existing conditions scenario and the site developed scenario.

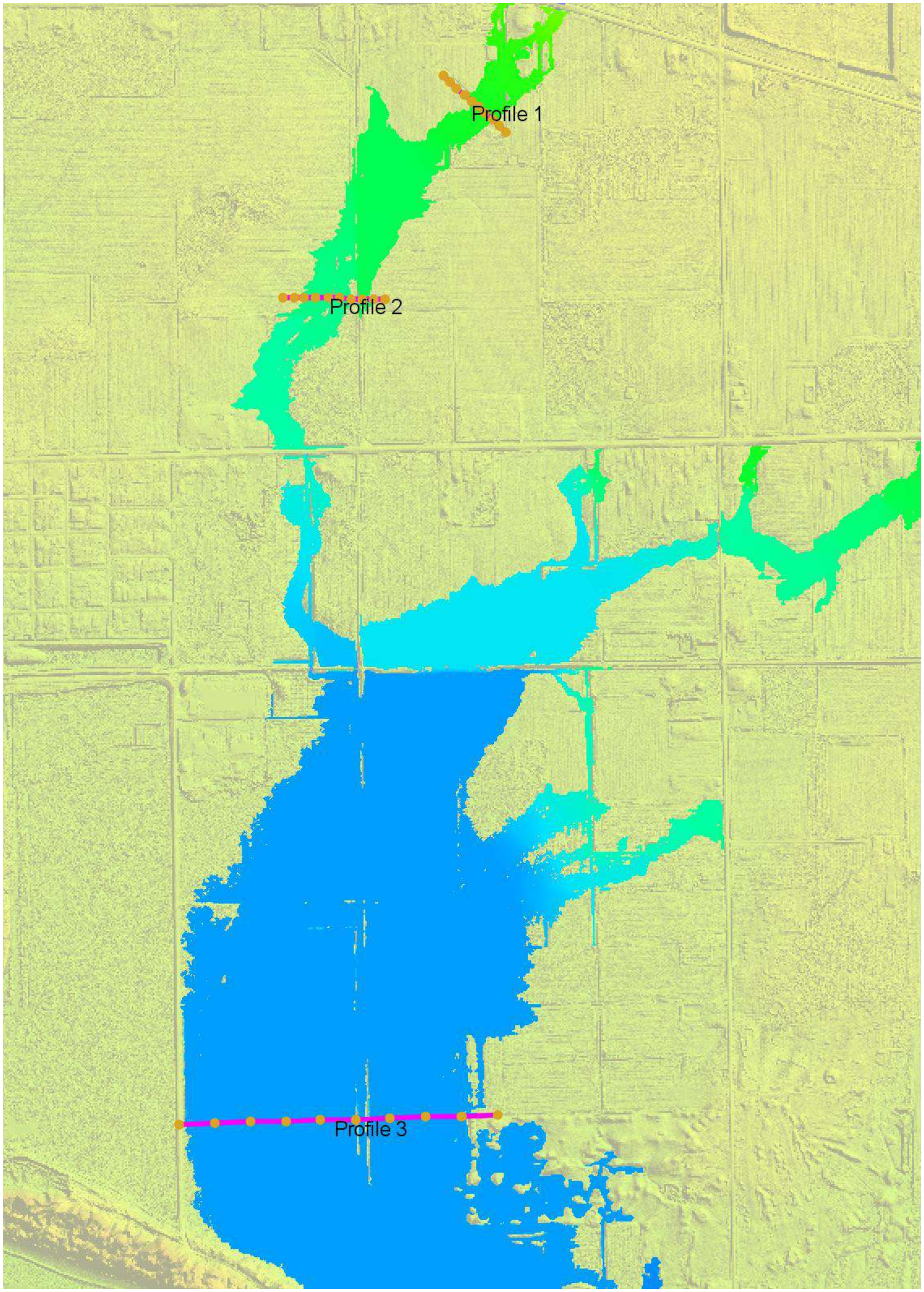
Table 9 Summary of hydraulic effects existing and Site Developed

	100-year 24 hr SCS		50-year 24 hr SCS		25-year 24 hr SCS		10-year 24 hr SCS		5-year 24 hr SCS		2-year 24 hr SCS	
	HGL (m)											
Location	existing	Developed	existing	Developed	existing	Developed	existing	Developed	existing	Developed	existing	Developed
Point 1	177.08	177.06	176.94	176.96	176.80	176.82	176.60	176.61	176.50	176.51	176.35	176.37
Point 2	177.20	177.19	177.10	177.13	177.03	177.07	176.96	177.00	176.88	176.92	176.74	176.76
Point 3	177.44	177.45	177.40	177.43	177.36	177.40	177.30	177.34	177.24	177.27	177.08	177.11
Point 4	177.77	177.74	177.74	177.70	177.68	177.65	177.58	177.58	177.50	177.54	177.40	177.49
Point 5	177.96	178.00	177.94	177.98	177.92	177.95	177.90	177.91	177.85	177.86	177.78	177.78
Point 6	177.44	178.43	178.41	178.42	178.40	178.40	178.38	178.38	178.32	178.33	178.22	178.22
Point 7	178.69	178.69	178.66	178.67	178.62	178.64	178.57	178.60	178.54	178.56	178.44	178.50
Point 8	179.05	179.04	178.98	178.98	178.90	178.91	178.78	178.78	178.68	178.69	178.54	178.57
Point 9	175.74	175.74	175.69	175.69	175.67	175.67	175.64	175.63	175.60	175.59	175.49	175.50
Point 10	175.73	175.73	175.68	175.68	175.63	175.63	175.55	175.55	175.47	175.48	175.40	175.40
Point 11	175.73	175.73	175.68	175.68	175.63	175.62	175.54	175.54	175.45	175.46	175.36	175.35
Point 12	175.69	175.69	175.62	175.61	175.54	175.52	175.43	175.42	175.35	175.35	175.27	175.27
Point 13	175.53	175.50	175.45	175.44	175.38	175.38	175.25	175.26	175.16	175.16	175.00	175.00

Note, essentially the existing and the developed Site HGL are the same.

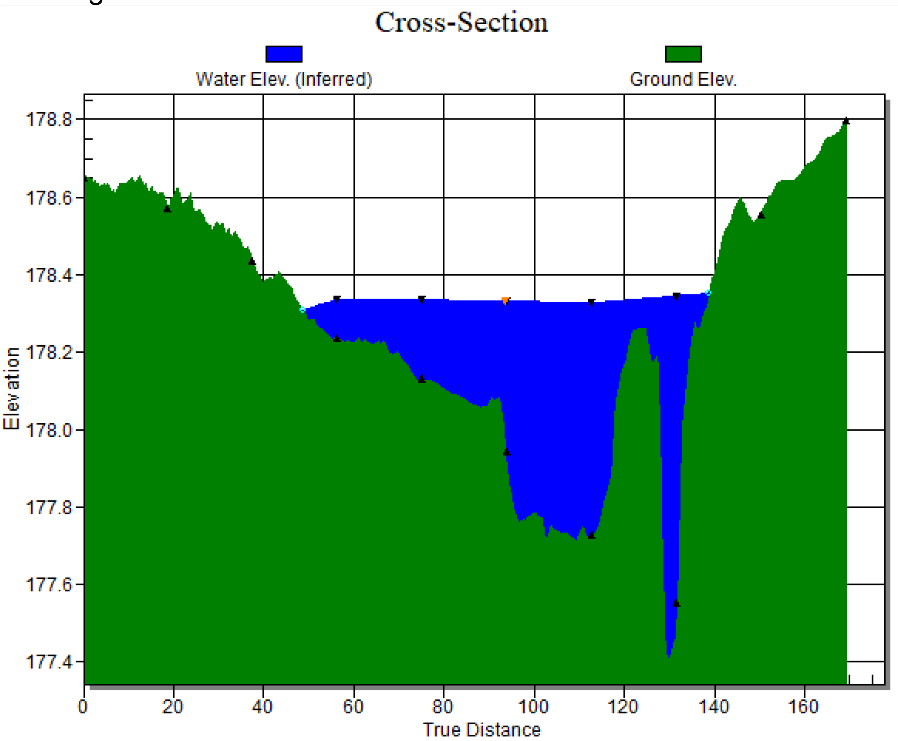
SECTION PLOTS

The following map shows the locations of profile plots for existing and proposed models.

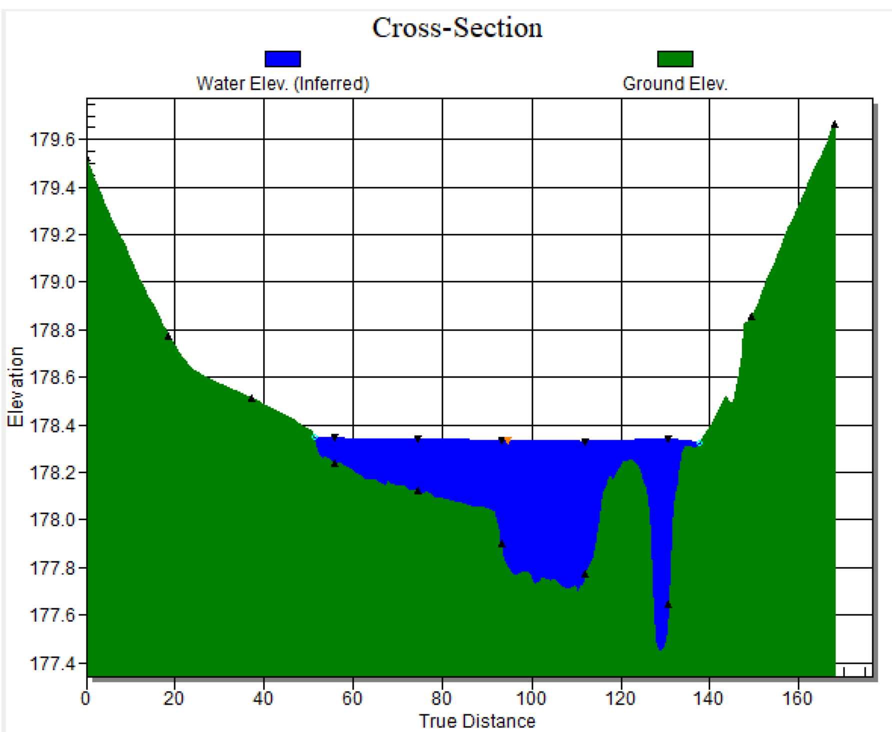


Profile 1 100-year 24 hr SCS

Existing

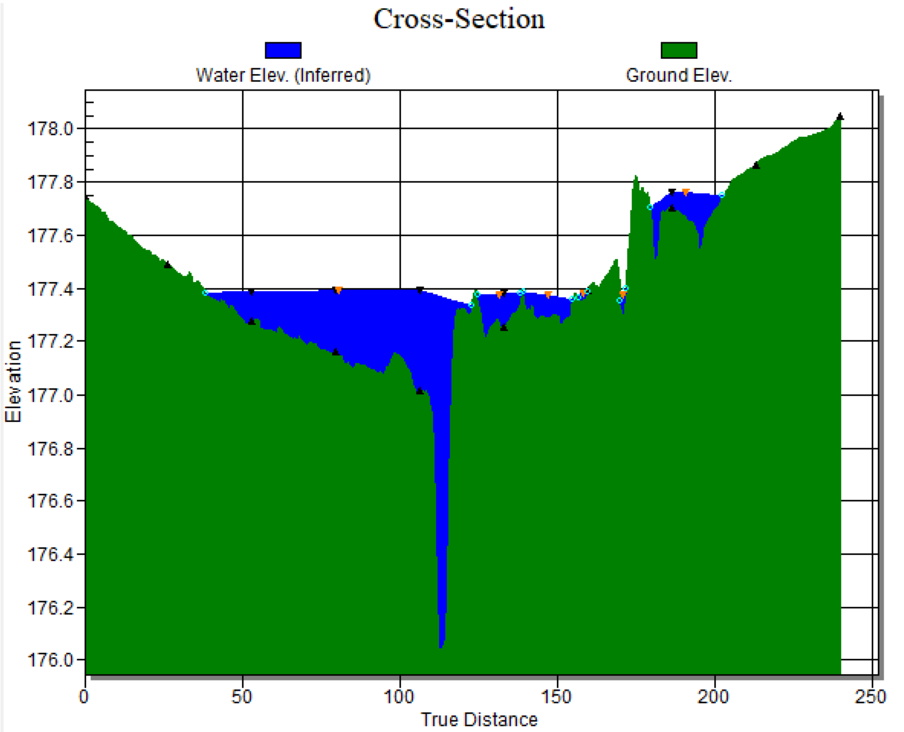


Site Developed

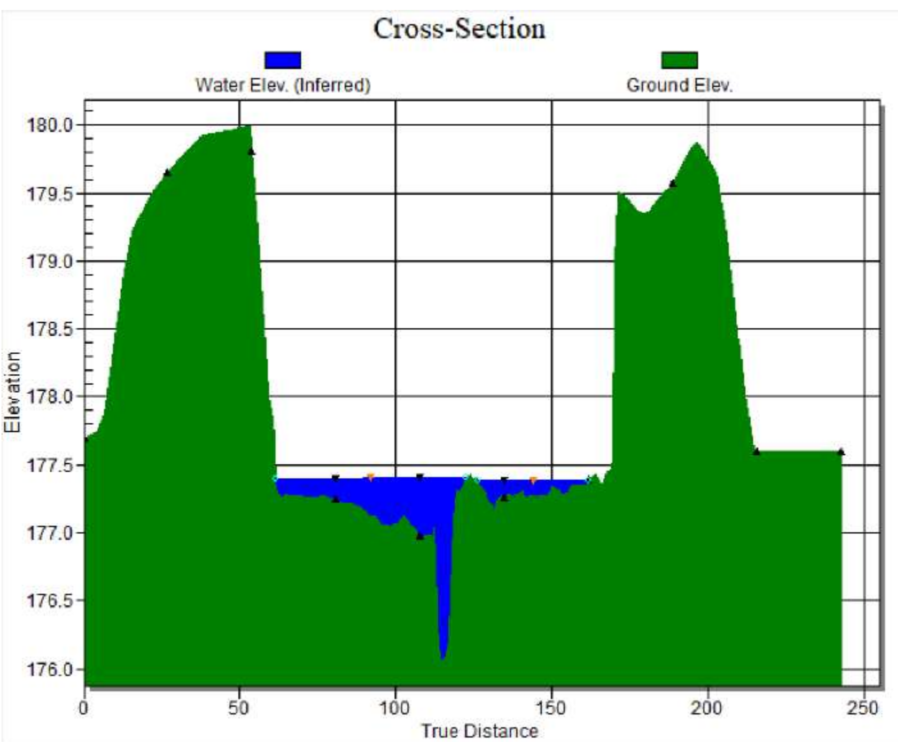


Profile 2 100-year 24 hr SCS

Existing

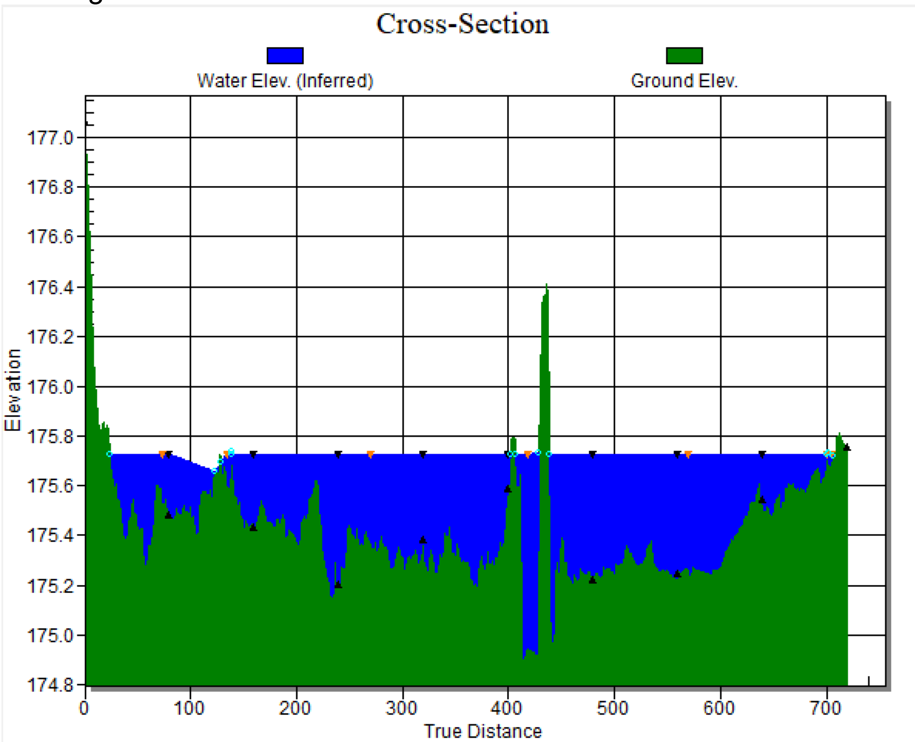


Site Developed

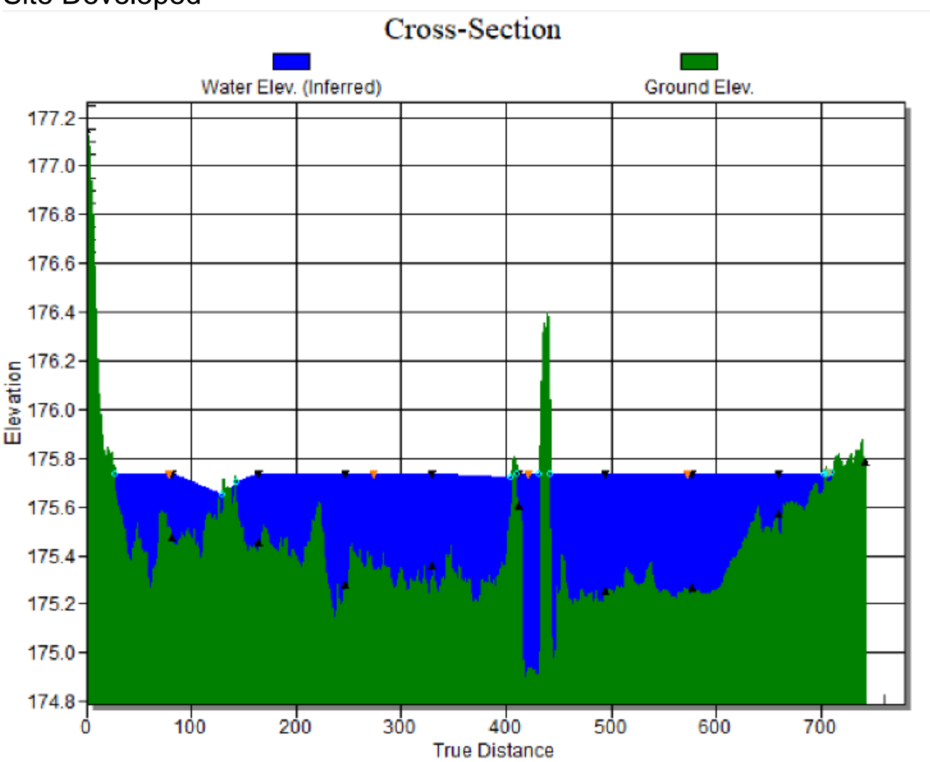


Profile 3 100-year 24 hr SCS

Existing



Site Developed



EROSION REVIEW:

The bed shear stress is calculated as follows in XP2D:

$$\tau_{bed(metric)} = \frac{\rho g v^2 n^2}{y^{1/3}}$$

where:

ρ = Density

g = Gravity

v = Velocity

n = Manning's n

y = Depth

Units are in N/m^2

Reproduced from HEC-15, FHWA-NHI-01-021 August 2001, Urban Drainage Design Manual, the following are the recommended shear stress as per vegetative cover.

Lining Category	Type	Permissible Unit Shear Stress (Pa)
	Class A	177.2
Vegetative	Class B	100.6
	Class C	47.9
	Class D	28.7
	Class E	16.8

Refer to the bed shear maps above for the calculated unit shear stress. Majority of the cells in the landscaped area and the drain area have bed shear stress less than 114 N/m2. Note this is for the 100-year Storm. The 2-year storm bed shear maps indicates a bed shear of 50 N/m2 in the existing condition and 39 N/m2 in the developed condition (maximum). This is well within the permissible range for a well vegetated area.

The objective is to not increase the erosion forces in the receiving natural stream. The MOE outlined an interim approach in 1994 and updated it in the Stormwater Management Planning and Design Manual (MOE, 2003). This updated approach consists of either a detailed design approach or a simplified design approach that is currently being improved to address inadequacies. Accordingly, it is recommended that the general approach be followed as outlined in the Stormwater Management Planning and Design Manual (MOE, 2003). This consists of designing SWM ponds to include active storage for the runoff from a 25 mm storm, followed by a check on erosion velocities in the downstream receiver. Quantity control to detain and release the 25mm, 4-hour Chicago design storm over a 24-hour period shall be provided for all receiving systems that are demonstrated to be stable watercourses or for proposed development that comprise less than 10% of the total area that drains to the receiving system.

Note, the shear bed method in 2D modelling is quickly becoming the method of choice due to testing that shows shear stress is a better indicator of erosion than velocity.

Refer to Table 10 for the comparison of existing to developed flows (target flows)

No.	Location description or crossing	S-4	Killaly culvert crossing east of Lorraine Road
S-1	Hwy # 3 culvert crossing west branch	S-5	Snider culvert crossing just north of Friendship Trail (former CNR)
S-2	Hwy # 3 culvert crossing east branch	S-6	Friendship trail culvert crossing adjacent to Snider Road west side
S-3	Killaly culvert crossing west of Snider Road	S-7	Outlet to Lake

Table 10 XPSWMM comparison of Outflow – existing (target) and redeveloped

Storm Event Storm Type	Target Peak Flow Rate (m^3/s)																				
	S-1			S-2			S-3			S-4			S-5			S-6			S7		
	Existing	Developed	PCSWMM	Existing	Developed	PCSWMM	Existing	Developed	PCSWMM	Existing	Developed	PCSWMM	Existing	Developed	PCSWMM	Existing	Developed	PCSWMM	Existing	Developed	PCSWMM
2 Year 24Hr SCS	0.439	0.439	0.406	0.730	0.728	0.764	1.092	1.098	1.169	0.685	0.690	1.184	1.025	0.989	1.803	1.026	1.024	2.923	0.480	0.469	2.908
5 Year 24Hr SCS	0.794	0.794	0.750	1.215	1.214	1.256	1.815	1.783	2.156	1.554	1.546	2.806	1.948	1.854	4.266	1.956	1.922	6.375	1.049	1.033	6.580
10 Year 24Hr SCS	1.054	1.054	1.009	1.592	1.595	1.671	2.413	2.461	2.975	2.325	2.320	4.276	2.515	2.404	5.189	2.541	2.502	7.933	1.475	1.413	8.521
25 Year 24Hr SCS	1.402	1.403	1.370	2.164	2.167	2.328	3.225	3.364	4.220	3.415	3.419	5.018	2.654	2.623	5.719	3.374	3.226	7.933	1.770	1.753	9.380
50 Year 24Hr SCS	1.651	1.651	1.655	2.656	2.656	2.906	3.937	4.032	5.273	4.230	4.243	5.294	2.684	2.643	6.081	3.886	3.820	7.933	1.955	1.919	9.791
100 Year 24Hr SCS	1.916	1.916	1.951	3.159	3.158	3.557	4.689	4.574	5.719	4.979	5.002	5.629	2.687	2.645	6.481	4.374	4.311	7.933	2.164	2.145	10.00
100 Year 12Hr AES	0.606	0.606	-	1.976	1.978	-	4.105	3.948	-	4.619	4.637	-	2.694	2.657	-	4.048	4.070	-	1.766	1.738	-

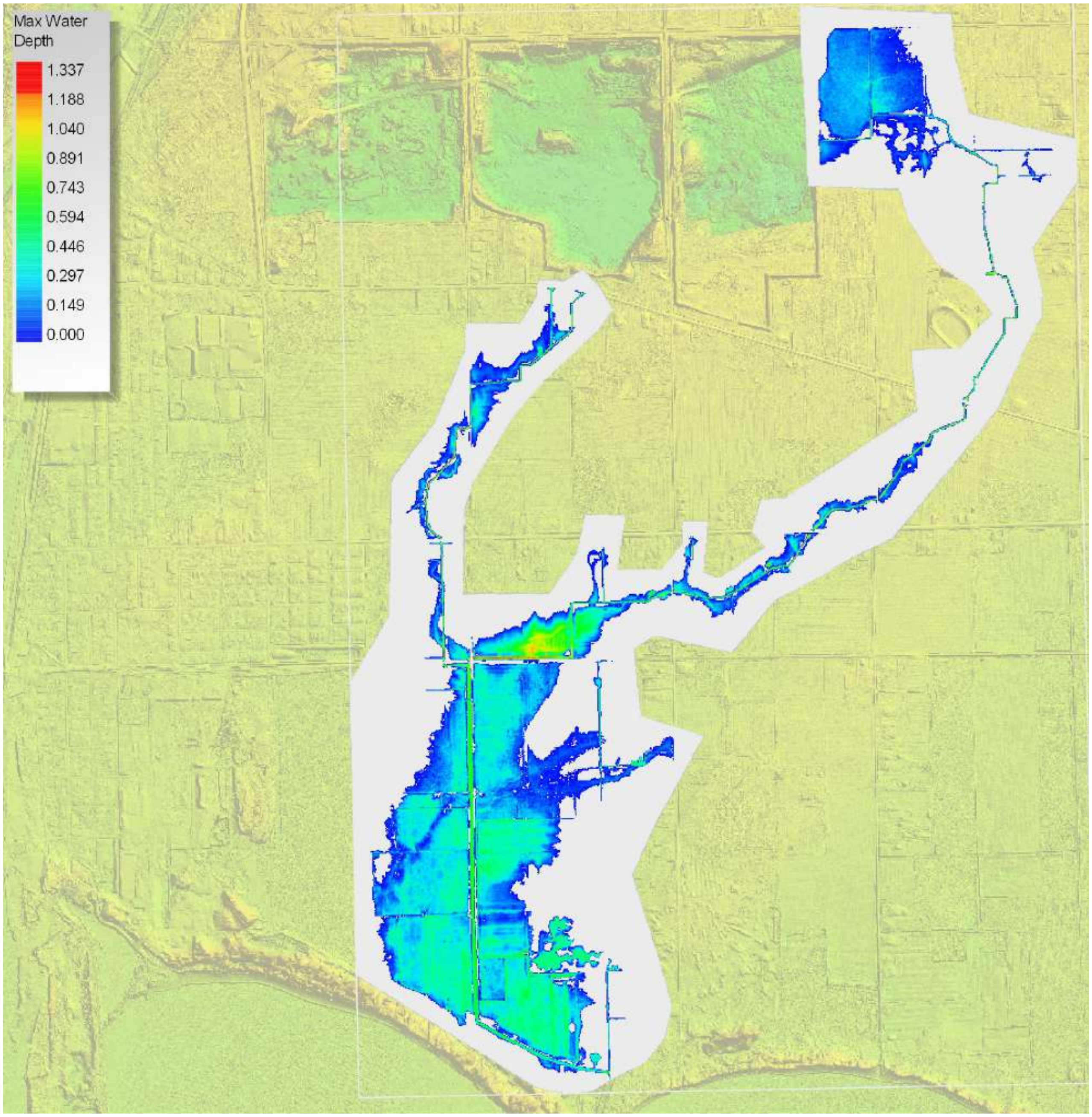
Note, PCSWMM had approximately 0.53 m3/sec from the quarry area which should not be included.

SCENERIO 3 Existing conditions - Lake boundary modified

In this scenario the Lake Erie boundary condition was 100-year Lake level + 10-year surface runoff event (Figure 41). The outfall was raised to **75.10 m** to represent the Lake 100-year level + wave setup. At the control structure a flap gate was added since the control structure has sluice gates. Only the 10-year storm was run.

In order to keep probabilities of lake levels and surface runoff real the scenarios will be as follows: 10-year surface and 100-year Lake Level and vice versa. Many Conservation Authorities and MNR use this criterion.

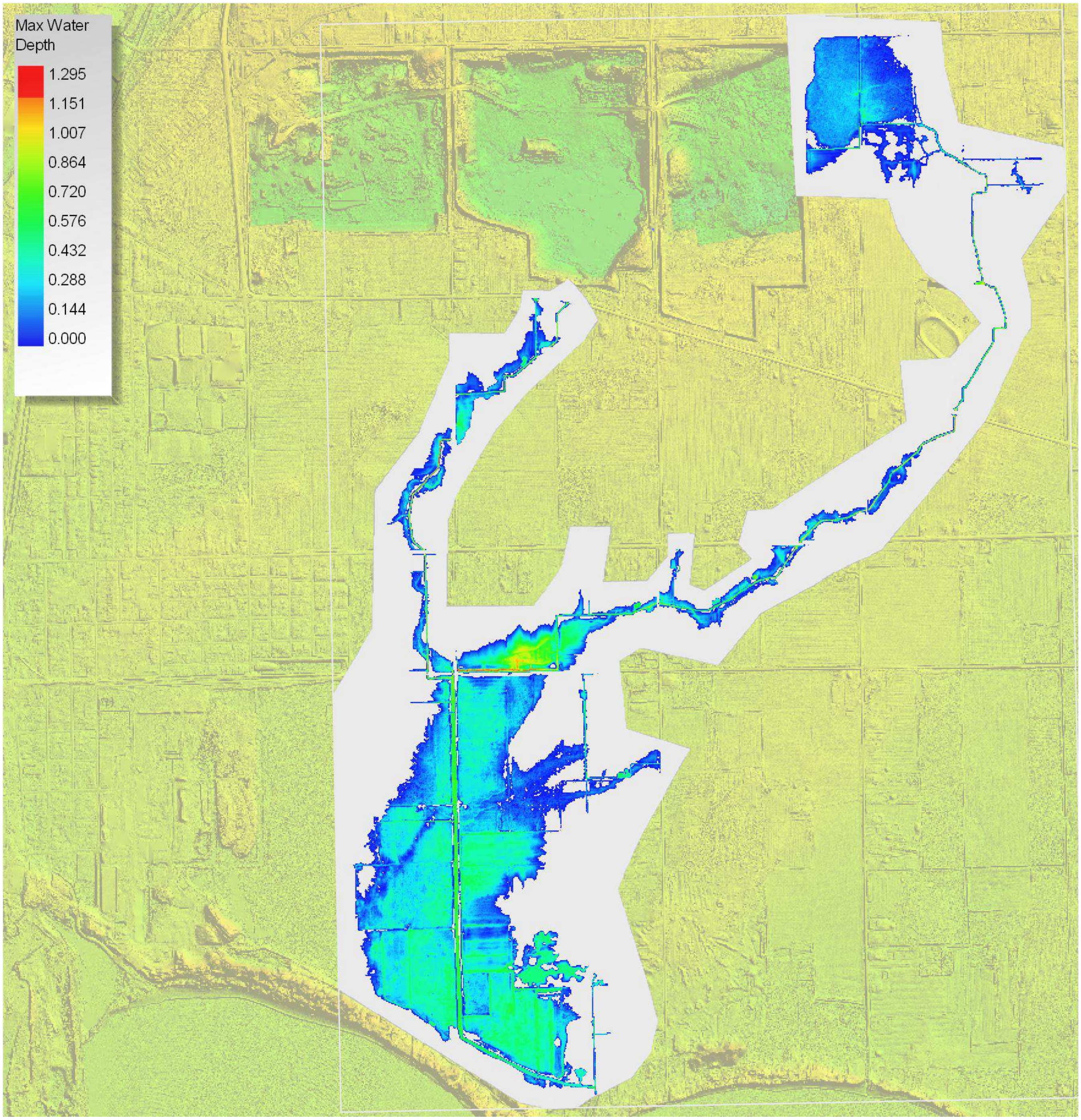
Figure 41 - XPSWMM depth MAP- Existing – 100 yr Lake – 10 yr storm



SCENERIO 4 Developed - Lake boundary modified

In this scenario the Lake Erie boundary condition was 100-year Lake level + 10-year surface runoff event (Figure 42). The outfall was raised to **75.10 m** to represent the Lake 100-year level + wave setup. At the control structure a flap gate was added, since the control structure has sluice gates. Only the 10-year storm was run.

Figure 42 - XPSWMM depth MAP- developed – 100-yr Lake – 10-yr storm



The following Table 11 summarizes the effect of Lake Erie levels on the drain for the 10-year storm under free flow conditions and when the lake is at 100-year levels.

Table 11 Summary of Lake effects - hydraulic effects existing and Site Developed

Location	10-year 24 hr SCS 100 yr Lake		10-year 24 hr SCS Free flow	
	HGL (m)			
	Existing	Developed	Existing	Developed
Point 1	176.61	176.61	176.60	176.61
Point 2	176.96	177.00	176.96	177.00
Point 3	177.30	177.34	177.30	177.34
Point 4	177.57	177.58	177.58	177.58
Point 5	177.89	177.91	177.90	177.91
Point 6	178.37	178.38	178.38	178.38
Point 7	178.58	178.60	178.57	178.60
Point 8	178.78	178.78	178.78	178.78
Point 9	175.64	175.63	175.64	175.63
Point 10	175.55	175.55	175.55	175.55
Point 11	175.54	175.54	175.54	175.54
Point 12	175.42	175.42	175.43	175.42
Point 13	175.31	175.31	175.25	175.26

Notes:

1. Based on Table 11 the only area effected by the high Lake water is point 13 near the Lake.
2. The development does not appreciably affect the area when the Lake is high.
3. The probability of lake levels and surface runoff rare events concurring are unlikely, thus the scenario of 10-year surface and 100-year Lake Level is more realistic.

REVIEW OF INGRESS/EGRESS AND RISK ASSESSMENT

INGRESS/EGRESS

Source: (Ontario MNR Technical Guide – River and Streams Systems: Flooding Hazard Limits, 2002).

MNR Technical Guide Appendix 5 page 28 states the question of safety for the passage of vehicles can be subdivided into:

- Flood depth and velocity considerations affecting egress of private vehicles from flood proofed areas;
- Flood depth and velocity affecting access of private and emergency vehicles to flood proofed areas.

The highest priorities for access to emergency vehicles should be given to police, ambulance and fire services, especially where evacuation is a distinct possibility in areas surrounded by flooding.

Access to the proposed development will be provided via streets that are not flooded. Note, Snider Road is the only street presently flooded. We are suggesting that Snider Road will be raised and a new culvert placed at the drain location to pass a 100-year storm. This will make Snider Road a non-flooded road.

Private Vehicle: allowable depth 0.3 m and velocity of 3.0 m/sec, hazard 1.0 m2/sec

Therefore, Private vehicle ingress/egress is not an issue.

Emergency Vehicle: allowable depth of 0.9 m and velocity of 4.5 m/sec for firefighting and depth 0.3 m and velocity of 3.0 m/sec for police and ambulance and hazard of 1.0 m2/sec (MNR Technical Guide).

Therefore, Private vehicle ingress/egress is not an issue.

Pedestrian: allowable depth 0.8 m and velocity of 1.7 m/sec and max hazard of 0.40 m2/sec (MNR Technical Guide).

Therefore, Pedestrian ingress/egress is not an issue.

RISK ASSESSMENT:

Application of the 2 x 2 rule (see Ontario MNR Technical Guide – River and Streams Systems: Flooding Hazard Limits, 2002) used in the assessment of potential loss of life. The 2 x 2 rule defines that people would be at risk if the product of the velocity and the depth (HAZARD) exceeded 0.40 square metres per second or if velocity exceeds 1.7 metres per second or if depth of water exceeds 0.8 metres. (Source MNR flood plain management).

Based on the above the risk is **low**.

WATER QUALITY

The Subwatershed Study & SWMP Implementation Document establishes the required guidelines for implementing stormwater quality for the proposed development. The requirements for Water quality are as follows.

"Control pollutant loadings in accordance with current MOE guidelines. Enhanced Level 1 protection as defined in the 2003 Stormwater Management Planning & Design Manual – reduce average long term annual load of suspended sediment by 80% or better. Accomplish through the use of LID source and conveyance controls."

Stormwater Source Control Policy for Industrial, Commercial and Institutional (ICI) Land Uses by NPCA is guide.

In order to achieve water quality for the proposed development each site will be required to implement the above measures to achieve an Enhanced Level 1 Protection of 80% removal of total suspended solids prior to discharge into downstream outlets.

In order to achieve the required water quality for each development the following methods can be considered at the detailed design stage for the subdivision. The following provides values established and generally accepted throughout the province for use of various TSS removal techniques.

Total Suspended Solid Removal Method & Removal Efficiency	
Removal Method	**Removal Efficiency**
Rooftop	*80%*
Grassed Swale (with Perforated Pipe)	*80%*
Grass Swale (no perforated Pipe)	*50%*
Soakaway & Infiltration Systems	*70-90%*
Chambers (with Infiltration)	*70-90%*
Bio retention	*80%*
Dry Swale	*80%*
Permeable Pavers (with Storage Bed)	*80%*
OGS (Oil/Grit Separator)	*50%-80%*
CB Shield	** 50%*
Wet Pond	*** up to 90% TSS removal if extended detention is used*

* - Based on published information provided by Manufacturer.
** - New Jersey Department of Environmental Protection

This may be due to constraints such as limited landscape space available throughout the site for implementing LID's, underlying soils conditions and conductivity to LID's, groundwater conditions and other factors that can limit the use of LID's. All reasonable attempts should be made during the detailed design stage to provide for the use of LIDs to enhance water quality measures.

The efficiencies of Low Impact Development Strategies are variable dependent on the maintenance and loading from the site usage. The above Table values are based on the generally accepted removal efficiencies based on the technique implemented.

In order to ensure the removal of oils each outlet will require an oil grit separator or method of removing oil spills prior to discharging to the downstream outlet and receiving water course.

It should be noted that Catchment G (see Figure S-6) will require independent, site level quality control since this catchment does not drain one of the SWM ponds provided within the development.

VOLUME CONTROL AND WATER BALANCE:

Improve quality of drainage discharging to each outlet from that of existing conditions;
Enhanced (Level 1), as per City, Region and NPCA Criteria. Reduction of the Total Suspended Solids (TSS) released to an enhanced level equal to 80% TSS removal, based on the MOECC 2003 criterion. Approach will be a train treatment.

LIDS:

The following LID methods are possible.

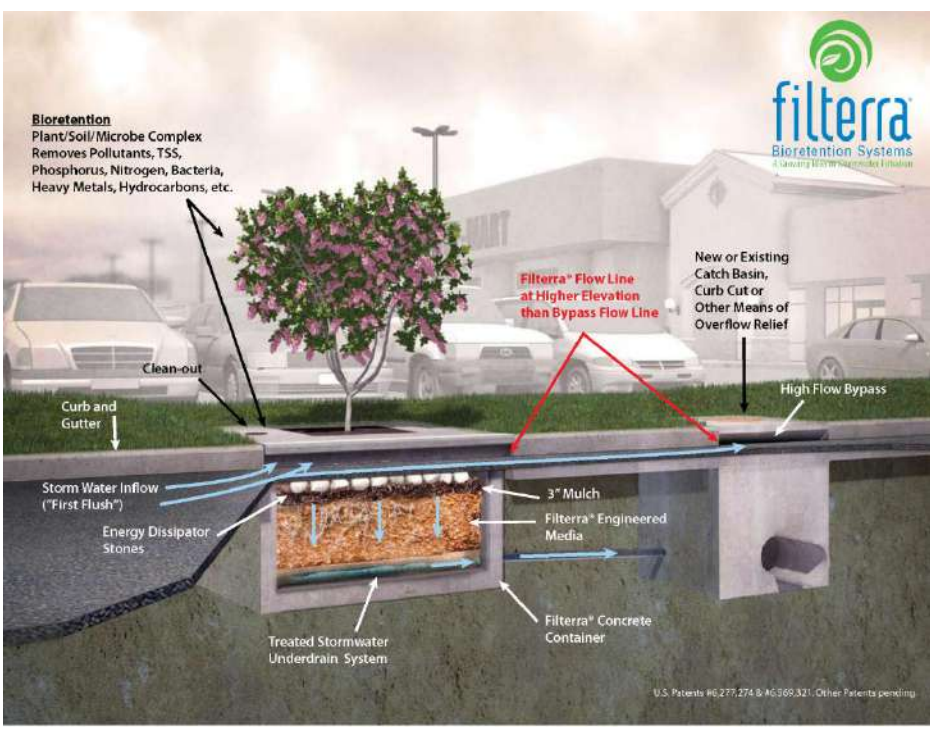
- Imbrium Filterra Bioretention System
- Silva Cells
- Soak away pits
- Bio swales
- others

We believe the following can be adapted for the SWM quality/water balance component:

1. Wet ponds as detailed in this report
2. Silva Cells or Imbrium Filterra Bioretention System on the roads if City will accept.
3. Soak away pits in the park area.
4. Bioswales if landscaped areas can accept.
5. Irrigation reuse.
6. Roof flow capture via barrels for reuse.

Imbrium Filterra Bio retention System

This is an appropriate method for water quality treatment in a train treatment environment. Storm water runoff enters the Filterra system through a curb-inlet opening and flows through a specially designed filter media mixture contained in a landscaped modular container. The following photos show the installed Filterra unit and a section through the unit.



Silva Cells

The Silva Cell is a modular suspended pavement system that uses soil volumes to support large tree growth and provide powerful on-site storm water management through absorption, evapotranspiration, and interception. The system is typically installed under pavement applications and can be configured in several different ways:

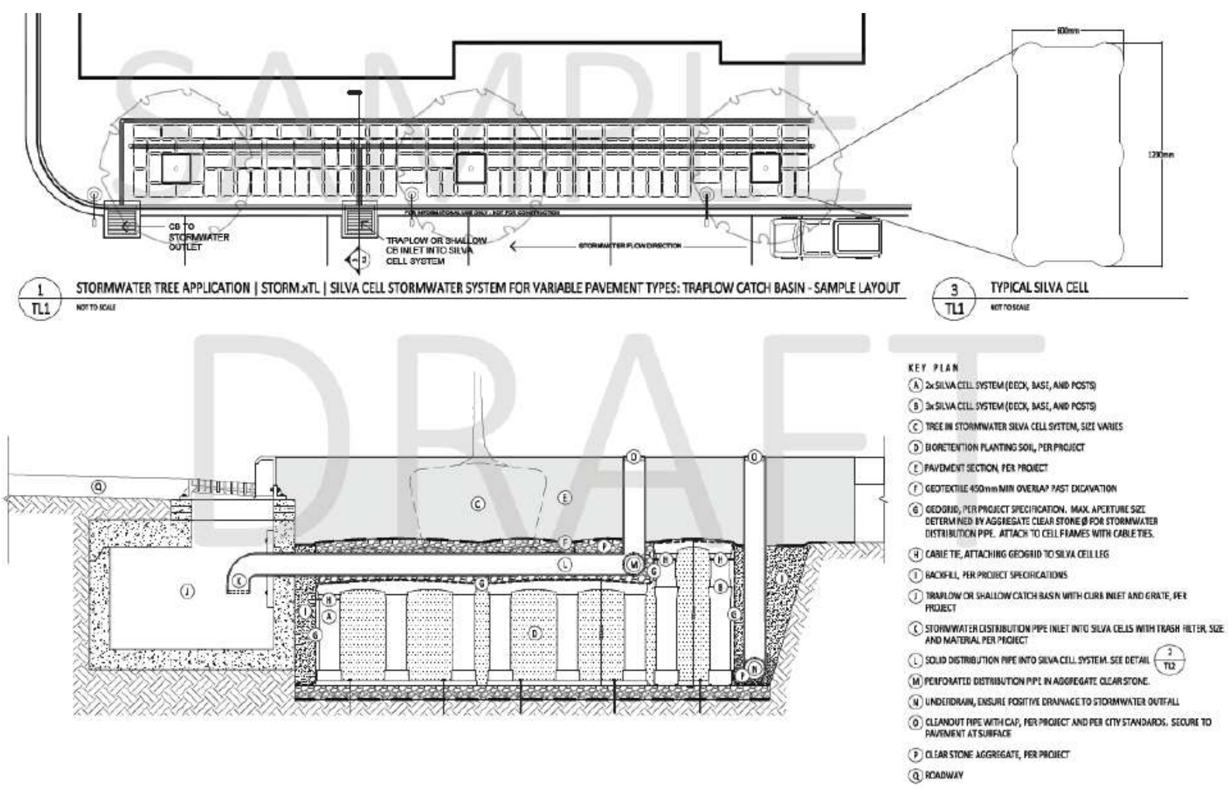
Streetscapes
Adjacent to or under sidewalks
Between buildings and streets.

Parking Areas
Under parking stalls adjacent
to medians or islands.

Public Spaces
Under plazas, promenades, courtyards, or other public spaces at office buildings, museums, schools, and transit centers

The Region of York is using Silva Cells on the widening and reconstruction of Yonge Street.

The following detail is a typical Silva Cell application.



WATER BALANCE/GROUNDWATER

Refer to report by EXP Hydrogeologist " Preliminary Hydrogeological and Water Balance Investigation", Killaly street East, Port Colborne Ontario.

The following is the summary from that report. Note, there is 33,161 m3 deficient in the infiltration rate from pre to post conditions. The site will require to infiltrate 33, 161 m3 of rainfall on an annual basis.

Killaly Street East, Port Colborne, ON
BRM-21000726-A0

Appendix E-4
Summary of Pre and Post-Development Water Balance (Unmitigated)

6. Comparison of Pre-Development and Post-Development Un-Mitigated

	Precipitation (cu.m.)	Actual Evapotranspiration (cu.m.)	Run-off (cu.m.)	Corrected Infiltration Rate for Areas with Shallow Groundwater Table (cu.m.)
Pre-Development	1,490,625	868,758	423,299	198,568
Post Development	1,490,625	579,163	746,057	165,406
			Pre-development Infiltration Rate	**133.3**
			Post-development Infiltration Rate Un-Mitigated	**111.1**
			Deficit Post Development Un-Mitigated	**33,161**

Criteria

Criteria for storm water balance, retention and low-impact-development (LID) is provided for the City of Port Colborne and by the Niagara Peninsula Conservation Authority (NPCA). The NCPA provides criteria in their manual *Niagara Peninsula Conservation Authority Stormwater Management Guidelines* (March 17, 2010).

Stormwater Volume Control Requirements in the NPCA manual provides criteria. The criteria applying to this development is generally described as follows:

- Any major development or disturbance that reconstructs 0.5 Ha of impervious surfaces are subject to storm water volume control criteria.
- Storm water volume reduction (storm water retention) may include such techniques as infiltration, reuse, rainwater harvesting, canopy interception, evapotranspiration and/or additional techniques.
- Redevelopment volume control – nonlinear redevelopment projects meeting the foregoing criteria shall capture and retain/treat on-site the runoff from a pre to post water balance analysis event from the new and/or fully reconstructed impervious surfaces.
- The retained runoff is to be dispersed on-site by the acceptable measures (above) in 48 hours.

The proposed development which comprises approximately 148.93 Ha of which 53.94 ha of impervious surface, is subject to the storm water balance/retention requirements. It is demonstrated as follows that the criteria can be addressed in the proposed development principally by infiltration, with additional retention provided by irrigation (evapotranspiration) and rainwater harvesting.

Based on the EXP report the deficit is 22.3 mm/a rainfall event, falling on the proposed new and reconstructed impervious surfaces, will generate the following storm water retention volume requirement.

Area of impervious surfaces = 53.94 x 10000 = 539,400 m^2

$$Required\ Stormwater\ Retention\ Volume = 539{,}400m^2\ x\ 22.3mm = 12{,}029m^3$$

Retention Strategy

It is proposed to principally retain the foregoing 12,029 m^3 by infiltration galleries whereby the foregoing volume of water will percolate into the underlying soil.

The locations and footprint available for infiltration galleries have been functionally considered in potential locations for infiltration galleries with a total footprint of 4.20 Ha. The infiltration footprint would be located within lands planned to be allocated for parks such that all infiltration galleries will be controlled. The footprints will need to be sized such that there is a minimum 5m setback (OBC latest edition) from the potential location of any buildings (above- or below-ground) on the adjacent development blocks.

The design criteria for infiltration galleries comprises the following factors. The province of Ontario's *Stormwater Management Planning & Design Manual* (2003) provides design criteria for infiltration galleries. The criteria are identified and addressed as follows.

- Underlying groundwater table elevation

- Criteria: the MECP states that the groundwater table or bedrock elevation should be 1.0m below the bottom of infiltration galleries.

- Design: A Hydrogeological Investigation was prepared by EXP, Dated September 15, 2021. Table 3.1: *Summary of measured groundwater elevations in Monitoring Wells* from 14 wells. The observed groundwater is typically 0.4 to 3m below existing grade. This is sufficient depth below-grade in which to install an infiltration gallery with 1m clear above the groundwater/bedrock. If necessary, the landscaped space in which the infiltration galleries will be installed can be graded such that there is sufficient cover above the stable groundwater table in which to install an infiltration gallery.

- Percolation rate of underlying soils. The MECP states that infiltration galleries should only be proposed where the percolation rate of receiving soils is greater than 15mm. Infiltration gallery footprints are to be designed considering percolation rates.

- EXP has not provided infiltration rate. We are assuming the MOE 2003 minimum of **15mm/hr** in the following analysis – this has been applied in the following infiltration gallery design calculations.

- Drain down time of infiltration galleries.

  Criteria: The MECP and NCPA manuals state that infiltration galleries should drain-down in 48 hours following the design storm event.

  Design: A drain-down time of 48 hours has been applied in the following infiltration gallery calculations.

Shown below is a sample infiltration gallery sizing calculation showing that the infiltration gallery footprint required to drain-down a retention volume of 12,029m^3 (above) within 48 hours is 41,766m^2, which aligns with the potential infiltration gallery areas within the site mentioned above (42,000m^2). This is less than the required footprint, therefore this is preliminarily a feasible means of addressing the storm water retention requirement in-full.

It is possible that in the future design, refinement of the infiltration galleries placement yields smaller available footprints than has been preliminarily identified above. In such a case, the water balance volume can be achieved by other forms such as irrigation and other forms of greywater reuse such as roof capture barrels.

INFIILTRATION GALLERY CALCULATION

PROJECT: Elite - Killaly Street East - Port Colborne

PROJECT No. : 21247

Location of infiltration gallery: TBD

						DESCRIPTION
						A = 1,000V / (PnΔt)
					UNIT	
	d = PΔt/1000		P=	15	mm/hr	Where:
			Δt =	48	hr	
d =	0.72	m				A = Filter bed surface area (m2)
			n=	0.40	-	V = Water volume (m3)
						Δt = time to drain (hr)
						n = void space ratio for aggregate used
V=	12029	m3	Atrib	539400	m2	(note: void space ratio of 0.4 to be used)
			runoff	22.3	mm	P = soil percolation rate mm/hr
						d = PΔt/1000
						Atrib = pervious area contributing runoff
						Where:
						d = maximum soak-away depth (m)
A =	1,000V / (PnΔt)					P= infiltration rate for native soils (mm/hr)
						Δt = time to drain (hr)
Af=	41766	m2				Af = Filter bed surface area provided (m2)
Af provided =	42000	m2				Vpit req'd = V/n
						Vpit provided = L x W x d
Vpit req'd =	30072	m3				
			Af =	42000	m	L = length of pit (m)
Vpit provided =	**42000**	m3				W = width of pit (m)
			d =	1	m	d = depth of pit (m)

NOTE: 10 SOAK-AWAY PITS WILL BE PROVIDED THAT TOTAL THE ABOVE VOLUME

SPECIAL SERVICING REQUIREMENTS DUE TO ROCK AND GROUNDWATER

Groundwater levels in the monitoring wells on site ranged from 0.4 to 3.0 m below grade (Elev. 175.2 to 180.3 m).

In the event the base of the pond is to be constructed below the recorded, static groundwater table, dewatering will be required to allow the excavation of the pond and the construction of the clay liner. Due to this site being in a well head protected zone a liner would be required.

WETLAND ASESSMENT

Refer to the EIS by Palmer for the evaluation of the wet land on site and the Wignelll bog.

The lower reach (south of freedom trail, CNR tracks) of the Drain system has zero (or nearly zero) positive graded channel to the Lake (outlet). The lower lands are historically the Wignelll Bog and are bog wetland that was formerly operated as a market garden and is now vacant land. The primary soil constituent of these lands is underlying peat, which acts as a sponge for runoff flows.

Recently it has been reported that there have been algea blooms within the Wignell bog. Algae blooms are primarily, a result of agricultural activity. Algea blooms lower dissolved oxygen concentrations in water, making it difficult for other plant and aquatic species to survive.

DISCUSSION/COMMENTARY AND OBSERVATIONS

The following are in no particular order:

1) The 1D/2D model approach is more realistic. The EWA model does not capture the culverts interacting with the channels in real time. The NPCA HEC-RAS has no respect to the attenuation effect behind the culverts.

2) The proposed infill development will not have a negative effect on the properties adjacent to the Site when developed.

3) Erosion is not a concern for the subject site. Refer to section on erosion review. Erosion control can be implemented as per MOE 2003 in the proposed wet ponds. Quantity control to detain and release the 25mm, 4-hour Chicago design storm over a 24-hour period

4) Snider Road will be raised in the post developed site. It must be taken out of the flood plain.

5) Raising Snider Road creates a stacking effect on the flood waters on the east side. This can be seen in the HGL point 4. In the pre scenario the flood waters flow over Snider Road and in the post scenario they do not (compare Figure 23 to 31). This can be rectified in the final design by provided a ditch at point 4 to the new outlet culvert crossing on Snider Road.

6) There is a difference in the topography of the drain area through the developed area. This will create small differences in the sections (pre to post). Refer to 3D view of post-developed graded Site.

7) The aquatic, wetland and terrestrial resources as identified by Palmer and shown on the draft plan by Armstrong are to be protected.

8) All wet ponds will have bottom draw outlets to control temperature.

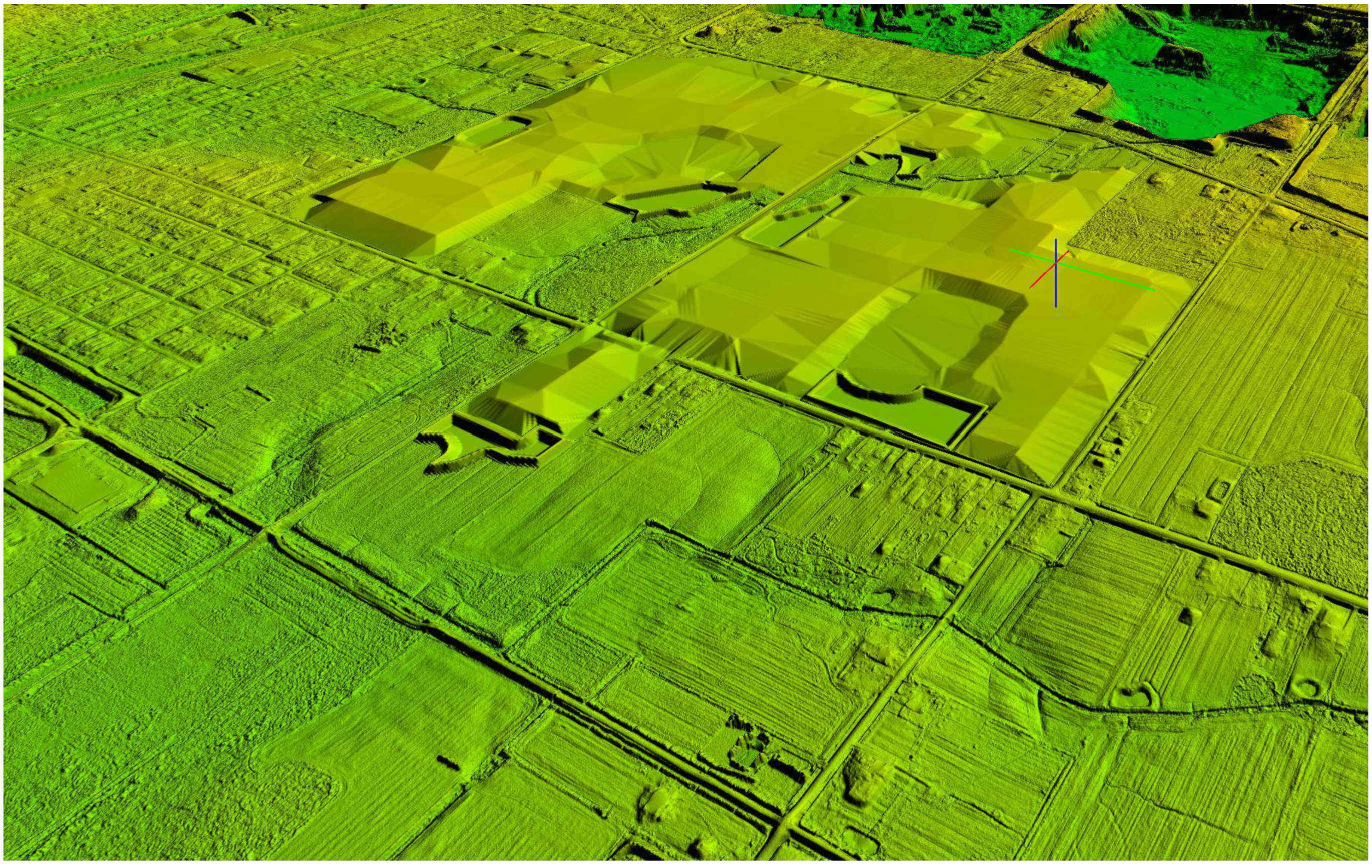
9) All pond outfall structures will be above the 100-year flood plain.

10) All ponds are outside the 100-year flood plain.

6.0 GRADING CONSIDERATION

Grading for the proposed site will be such that major overland flow will be directed to the proposed wet ponds. A conceptual grading design has been proposed on Figure S-8 in Appendix D showing how the site will tie into existing surrounding sites and direct overland runoff to the various wet ponds. Refer to Figure 43 for 3D view of the post developed graded site. The rendering is crude but shows that fill will be required.

Figure 43 – 3D view of post-developed graded Site



Note, the natural heritage sites are preserved
Note, the vertical scale is greatly exaggerated

7.0 RECOMMENDATIONS

We recommend the following:

1. A new central wastewater pump station be located within the development as shown in this report. A force main be routed from the pump station north to the future tunnel crossing the Welland canal. This crossing be co-ordinated with the Region of Niagara.

2. The pump station be sized to handle the entire development plus neighbouring properties that can economically drain to the new pump station.

3. The existing watermains be extended depending on the phasing plans and be looped (no long dead-end mains) as detailed in this report.

4. The NPCA flood shape file (regulatory) be implemented in the design of the subdivision.

5. That 6 SWMM ponds be incorporated into the design as detailed in this report.

6. That Snider Road be re-graded to take it out of the flood plain.

7. That a new culvert be established under neath Snider Road at the drain to allow a 100-year storm to pass.

8. That the NPCA regulatory flood model be updated once the geometry for Snider Road and the grading for the subdivision are established.

9. Infiltration galleries be implemented where the water table and rock elevation will be 1.0 m below the bottom of the gallery, in order to match water balance.

10. The aquatic, wetland and terrestrial resources as identified by Palmer and shown on the draft plan by Armstrong are to be protected.

8.0 CONCLUSIONS

The proposed development can be serviced with sanitary via extension of the existing sanitary sewer and upgrades as identified within. Water service can be provided via connecting to existing and extension through the subject property through looping as shown in this report. Stormwater management can be accomplished for quantity, quality, water balance, and infiltration as described in this report.

Based on the findings the site is serviceable and can be provided with adequate storm, sanitary and water services as described within the report.

9.0 REFERENCES

1. STORM WATER MANAGEMENT PLANNING AND DESIGN MANUAL, Ontario Ministry of the Environment, March 2003.

2. Engineering Guidelines for Servicing Land Under Development Applications December, 2012.

3. NIAGARA REGION WATER-WASTEWATER PROJECT DESIGN MANUAL, Revision 3, July 2023.

4. THE CITY OF WELLAND MUNICIPAL STANDARDS DESIGN CRITERIA, February 2013.

5. Niagara Peninsula Conservation Authority Stormwater Management Guidelines, March 10, 2017.

6. "Wignelll Watershed Hydrology and Hydraulics Report", EWA Engineers Inc., August 31, 2021.

7. **"**NIAGARA PENINSULA CONSERVATION AUTHORITY FLOOD PLAIN MAPPING WIGNELLL DRAIN CITY OF PORT COLBORNE**"**, NIAGARA PENINSULA CONSERVATION AUTHORITY, August 2011.

8. "ENVIRONMENTAL IMPACT STUDY", Elite Properties East of Port Colborne, Palmer, October 2023.

9. DESIGN GUIDELINES FOR DRINKING WATER SYSTEMS, by the Ministry of the Environment of Ontario (now The Ministry of the Environment, Conservation, and Parks; MECP), dated 2008.

10. Ministry of Environment "DESIGN GUIDELINES FOR SEWAGE WORKS 2008"

11. INFO-WORKS ICM 7.5 - Reference Manual, May 2016.

12. INFO-WATER PRO - Reference Manual, Latest edition.

13. NEW JERSEY STORM WATER BEST MANAGEMENT PRACTICES MANUAL, April 2004.

14. MNR Technical Guide – River and Streams Systems: Flooding Hazard Limits, 2002.

15. FEMA Chapter 4 - Flood Risk Assessment.

16. ROAD AND BRIDGE DECK DRAINAGE SYSTEMS by MTO, November 1982.

17. XPSWMM users Guide by INNOVYZE 2022.

18. EPA SWMM 5, Build 5.1.012, Manual.

19. LOW IMPACT DEVELOPMENT STORMWATER MANAGEMENT MANUAL, 2008, by Credit Valley Conservation Authority and Toronto Town Conservation Authority.

20. THE EROSION AND SEDIMENTATION CONTROL GUIDELINES FOR URBAN CONSTRUCTION prepared by the Greater Golden Horseshoe Area Conservation Authorities.

Respectfully Submitted;
The Odan Detech Group Inc.

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Paul Hecimovic, P.Eng

Mitchell Hillmer, P.Eng