CANARD VALLEY ESTATES SUBDIVISION
PHASE 2
STORMWATER MANAGEMENT PLAN

IN THE
TOWN OF AMHERSTBURG
COUNTY OF ESSEX
ONTARIO

LICENSED PROFESSIONAL ENGINEER
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PROVINCE OF ONTARIO

File No. 18-743

November 2019
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Consulting Engineers

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INTRODUCTION

The development, known as Canard Valley Estates Subdivision, is a 19.0 ha parcel of land located in the Town of Amherstburg, County of Essex, Ontario. The development is divided in two phases: 10.2 ha for Phase 1 and 8.8 ha for Phase 2.

This report provides a stormwater management plan for the Canard Valley Estates Subdivision, Phase 2 Development.

The servicing of Phase 1 was completed in 2005. The stormwater measures were based on the '*Conceptual Stormwater Management Report for the Breschuk/Middle Sideroad Residential Community Phase 1 and 2"* prepared by HGS Limited in 2004.
The existing stormwater management facility includes an exfiltration system consisting of large diameter storage pipes in combination with Big 'O' subdrains and stone drainage layer.

The recommended stormwater management (SWM) plan was selected to meet the current *'Windsor/Essex Region Stormwater Management Standards Manual'* (ERCA SWM Manual), prepared by Stantec Consulting Ltd. for the Windsor/Essex Conservation Authority, 1st Publication dated December 6, 2018.

The purpose of the Plan is to provide the required quality and quantity controls for stormwater flows generated by the development under analysis and to minimize the hydrologic impacts of the proposed works on the existing drainage system.

The proposed strategy is to convert a portion of the existing exfiltration type stormwater management facility located in the park area to a new wet pond. The pond will service both, the existing Phase 1 and the proposed Phase 2 Canard Valley Estates Subdivision.

The pond is designed to accommodate the required quality and quantity controls and will retain runoff generated by the development during storm events up to and Including 1:100 year storm frequency. It is proposed to install a 150mm dia. orifice and backflow preventer in the existing 450mm dia. outlet pipe to ensure the release rate from the SWM system into the existing municipal drain will not exceed the allowable rate and to protect the subdivision SWM system from backwater inflow.

1. GENERAL

1.1 Site Location

The proposed works are located on Part of Lot 6, Concession 8, Geographic Township of Anderdon now in the Town of Amherstburg, County of Essex, Ontario.
The parcel is bounded by Essex County Road 10 (Middle Sideroad) to the north, Essex County Road 11 (Walker Road) to the east, existing wood lot to the south and agricultural lands to the west. The Site location Plan is shown on Figure 1.

1.2 Land Use

The area is currently under agricultural use and designated in the Official Plan as residential (h-R1A) which allow the proposed single unit residential development, as shown on Figure 2.

1.3 Reginal Drainage Network

The area lies within Canard River watershed, King Creek sub-watershed.

The existing SWM facility currently servicing Phase 1 discharges flow into a 900mm diameter pipe along Walker Road, known as McGregor Drain, which flows southerly and outlets to King Creek Drain.
The majority of land designated as Phase 2 currently flows westerly to the Parent Drain via overland flow, shallow swales and the Parent Trembley Drain (W). The Parent Drain flows southerly and outlets to King Creek Drain.
The overall map of the regional drainage network is demonstrated on Figure 3.

1.4 Soil Classification

The general area is characterized as brookston clay , as identified by the Ontario Ministry of Natural Resources and Essex Region Conservation Authorities, as shown on Figure 4.
This soil type drainage is almost level with poor natural drainage and assigned in Windsor/Essex Stormwater Management Standards Manual to hydrologic soil group ‘D’.

1.5 Existing Conditions

The total area of Phase 1 and Phase 2 development is 19.22 ha.
Phase 1 area is 10.4 ha and consists of:

- 0.99 ha of park/SWM facility
- 0.51 ha of drainage corridor
- 1.97 ha of R.O.W
- 6.75 ha of fully serviced residential lots
- 0.18 ha of grading buffer along north property line

The Phase 1 storm sewers and overland flows currently discharge runoff into the exfiltration SWM facility located in park area (Block 80). The stormwater from the park exfiltration system is collected by 1600mm dia. pipes and discharged into the existing municipal sewer through a 450mm dia. pipe.

Existing lots from Lot 21 to Lot 34 do not have rear yard catchbasins and discharge rear yard flows into a swale with Big 'O' subdrain in sewer stone bedding located south of south property line and connected to the SWM system outlet.

The total area of Phase 2 is 8.82ha. Currently the parcel is under agricultural use. A strip of woodlot exists along the south limits of the property.

The pre-developed drainage conditions are demonstrated on Figure 5.

1.6 Proposed Conditions

It is proposed to develop Phase 2 of the subdivision as single unit homes. The proposed development will consist of:

- 0.30 ha for SWM pond expansion
- 0.87 ha of lands reserved for tree line protection
- 1.93 ha of R.O.W.
- 5.72 ha of single unit homes (69 residential lots).

A portion of the existing exfiltration type SWM facility located in the park area will be expanded to the total area of 1.29ha and converted to a wet pond. The temporary pond will be abandoned and lot 34 designated to the temporary pond will be available for the dwelling construction.

The new pond is designed as a three-cell wet pond and will have two separate sediment forebays for each phase. The sewer and overland stormwater flows of both the existing Phase 1 and the proposed Phase 2 developments will be released into the pond sediment forebays, stored in the pond and the existing 1600mm diameter pipes and released into the existing municipal drainage system at a restricted rate. The new 150mm diameter orifice will be installed in the 450mm diameter outlet in manhole MH1 to provide the required flow restriction.

The new 450mm diameter backflow preventer will be installed in the existing 450mm diameter outlet pipe for the protection of the subdivision SWM system from tail water effect.

The layout of the proposed post-developed drainage plan is demonstrated on Figure 6.

The preliminary road and lot grading plan is demonstrated on Figure 7.

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STORMWATER MANAGEMENT PLAN

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SITE LOCATION MAP

FIGURE NO.
1

N.T.S.

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STORMWATER MANAGEMENT PLAN

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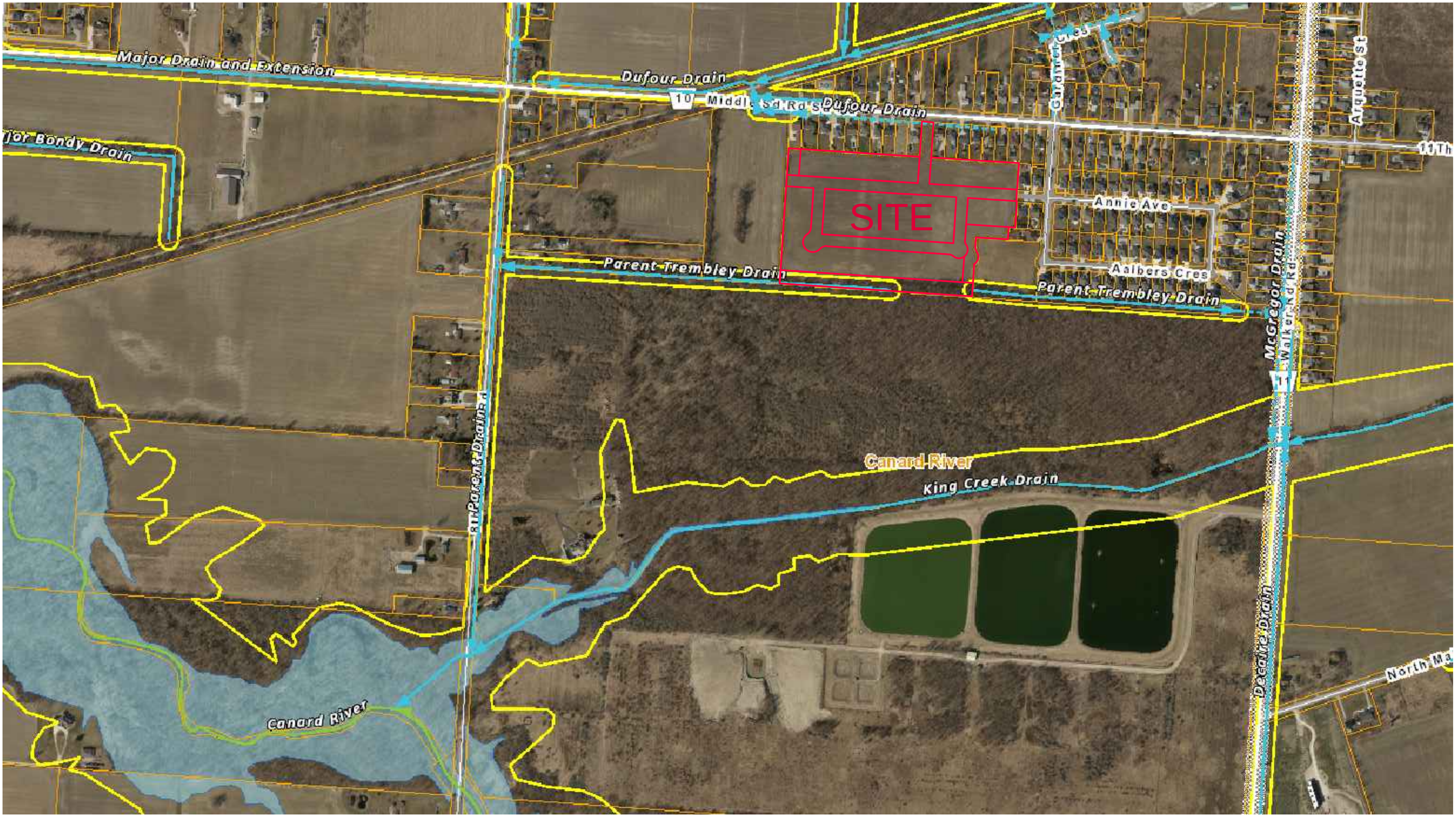
ZONING MAP

FIGURE NO.
2

N.T.S.



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CANARD VALLEY ESTATES SUBDIVISION PH. 2
STORMWATER MANAGEMENT PLAN

REGIONAL DRAINAGE NETWORK

PROJECT NO.
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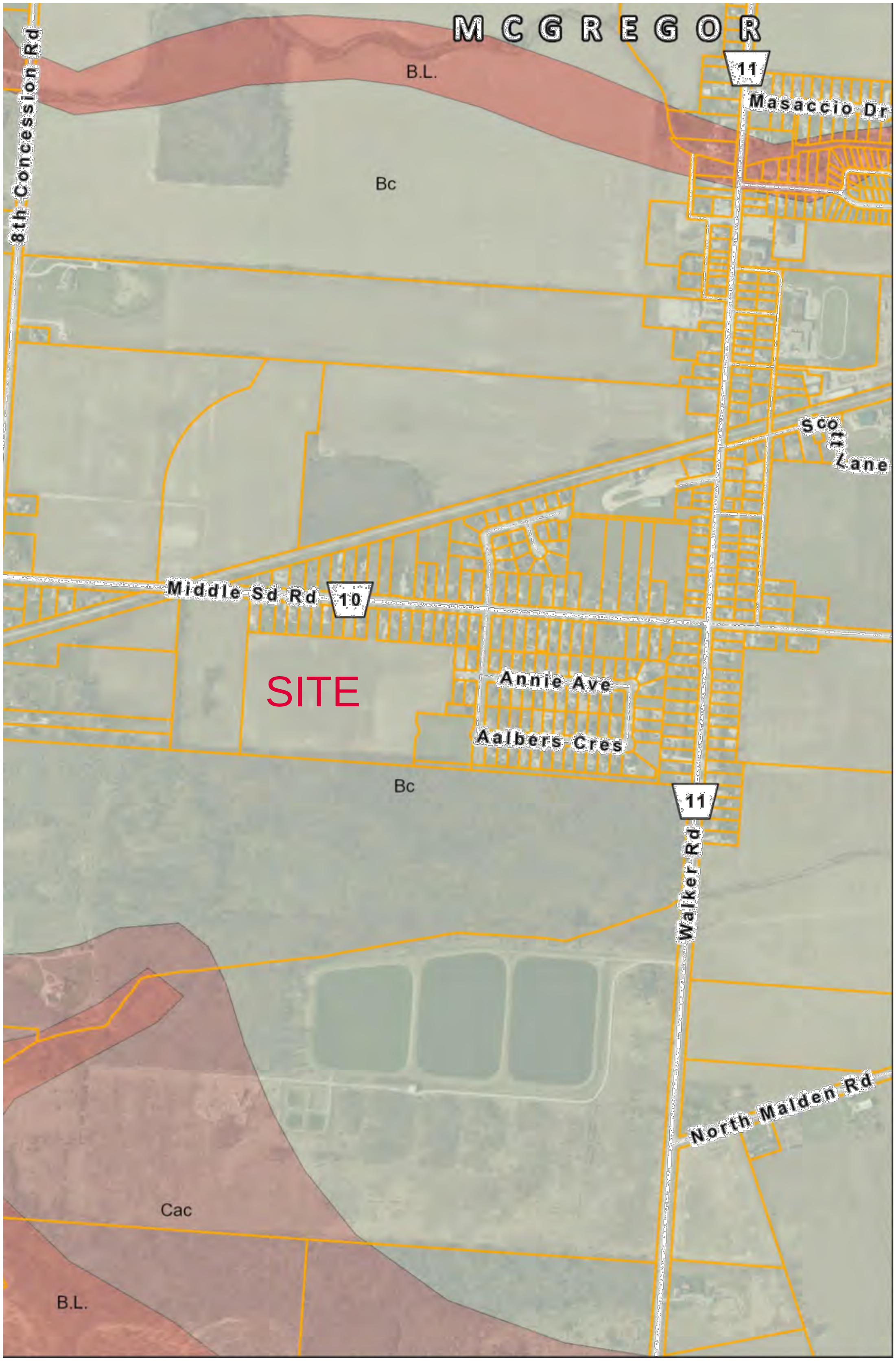
FIGURE NO.
3

N.T.S.



Essex Region Conservation Authority

Public Interactive Mapping

Legend

⬚ Parcel Fabric

Essex Soils

- 🟨 B-s
- 🟥 B.L.
- 🟩 Bc
- ⬜ Bcl
- 🟨 Bel
- 🟨 Bes
- 🟧 Bg
- 🟧 Bg-s
- 🟨 C-s
- 🟫 Cac
- 🟫 Cacl
- 🟫 Cc
- 🟨 Cdl
- 🟨 Es
- 🟥 Fl
- 🟧 Fsl
- ⬜ Gs
- 🟫 Hl
- 🟧 Hs
- 🟫 Jc
- 🟫 M
- 🟫 Ma
- 🟨 P-r
- 🟫 Pc

Location

0 266.70 533.4

Meters

1: 12,000

7/26/2019

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STORMWATER MANAGEMENT PLAN

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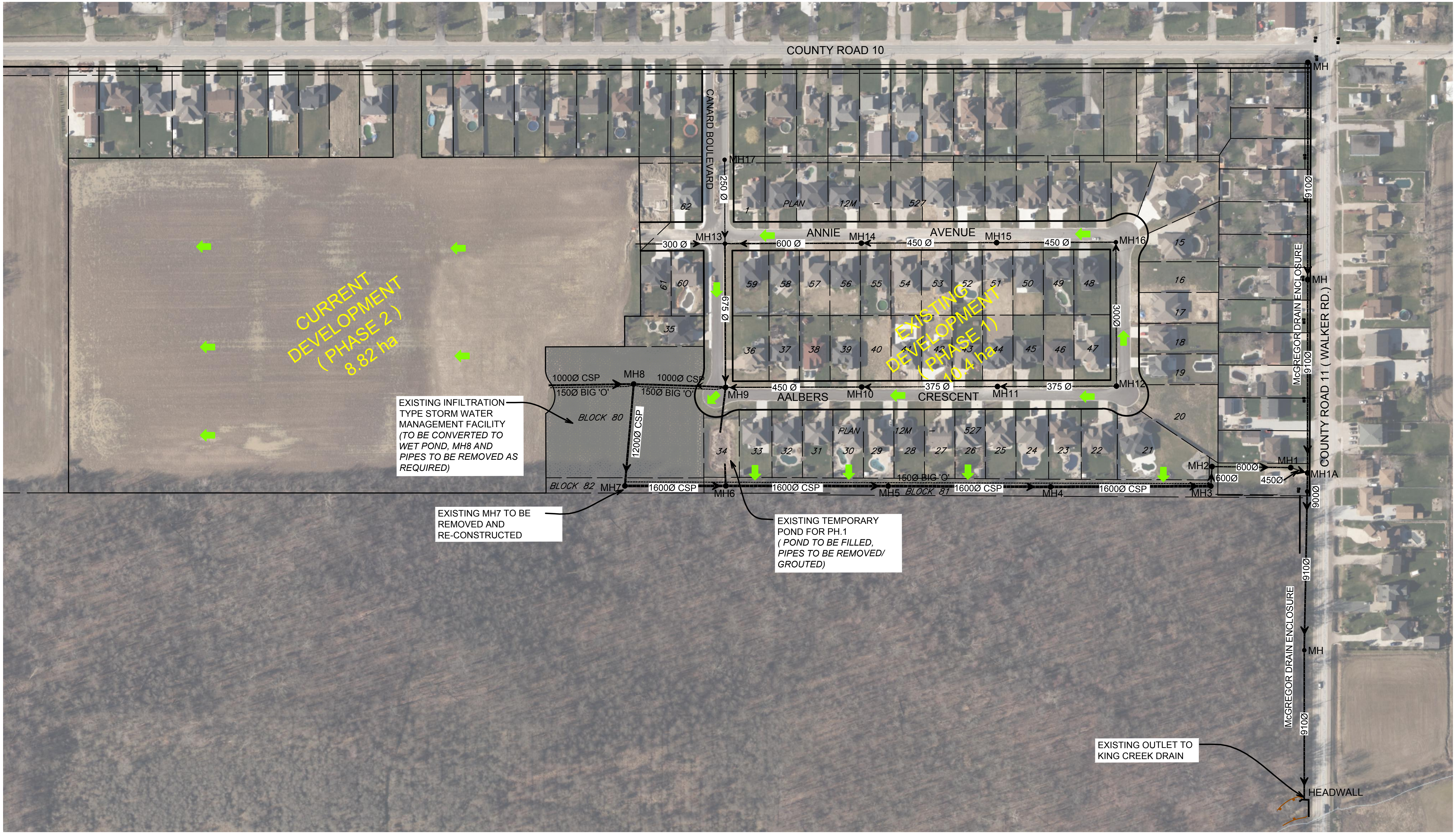
SOIL MAP

FIGURE NO.
4

1: 2500



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LEGEND:

➡ DENOTES OVERLAND FLOW ROUTE

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STORMWATER MANAGEMENT PLAN

PRE-DEVELOPED DRAINAGE PLAN

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FIGURE NO.
5

1:2500

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LEGEND:

➡ DENOTES OVERLAND FLOW ROUTE

CANARD VALLEY ESTATES SUBDIVISION PH. 2
STORMWATER MANAGEMENT PLAN

PROPOSED POST-DEVELOPED DRAINAGE PLAN

PROJECT NO.
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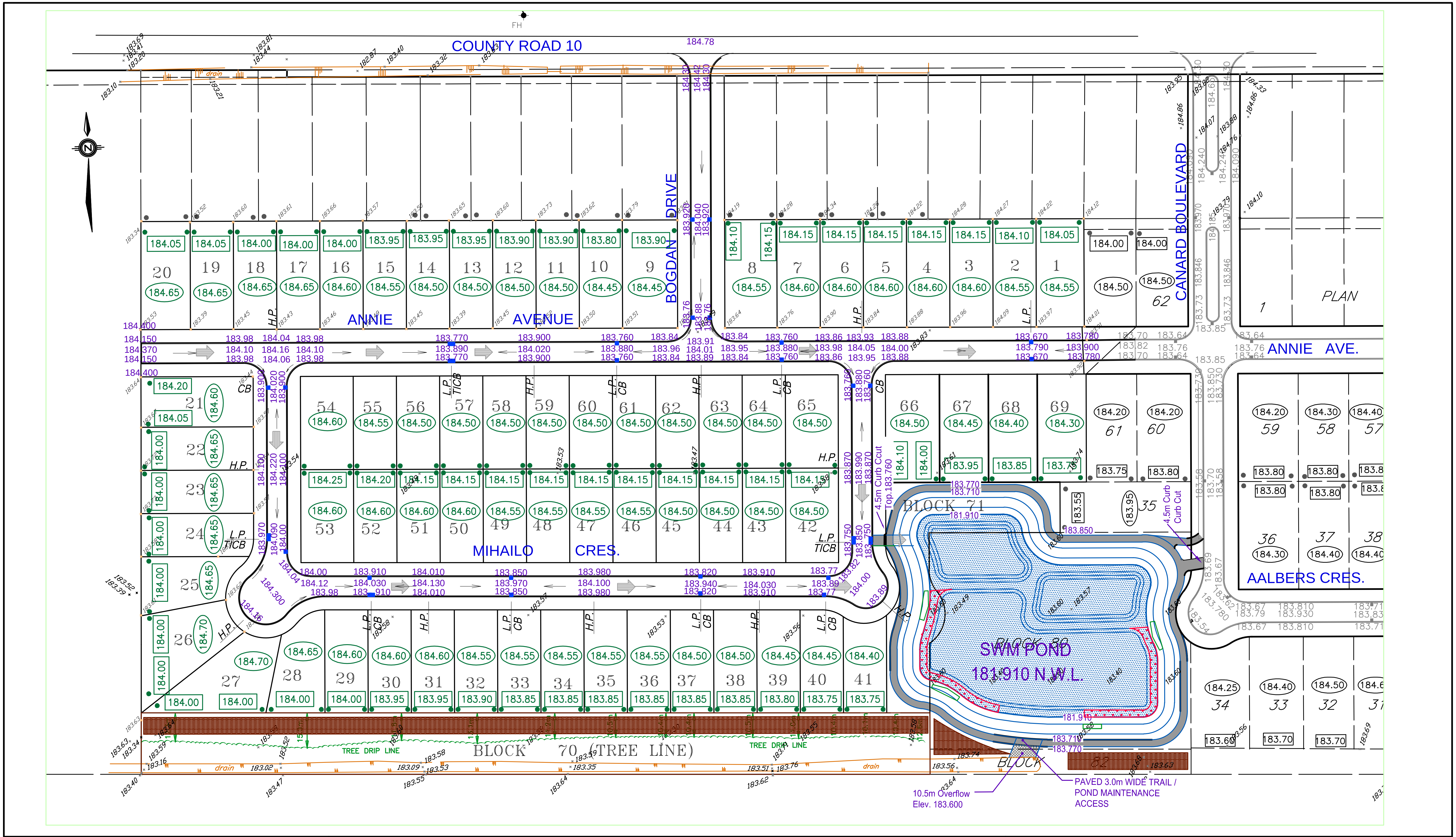
FIGURE NO.
6

1:2500



LEGEND:

➡	DENOTES OVERLAND FLOW ROUTE
183.55	PRELIMINARY REAR YARD CB ELEVATION
(184.45)	PRELIMINARY HOUSE ELEVATION
183.870	PRELIMINARY ROAD GRADES (C/L OR EDGE OF PAVEMENT)
▨	DENOTES NATURALIZATION AREA (PER ERCA AND MECP)

NOTE:

ALL GRADES ARE PRELIMINARY AND TO BE REVIEWED AT FINAL DESIGN STAGE

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CANARD VALLEY ESTATES SUBDIVISION PH. 2
STORMWATER MANAGEMENT PLAN

PRELIMINARY GRADING PLAN

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FIGURE NO.
7

1:1500

2. STORM SEWER (MINOR) SYSTEM DESIGN

2.1 Design Methodology and Return Period

Storm sewer network for the subdivision was designed using Rational Method**.**
The 1:5 year return period was used for new storm sewer design as standard for urbanized areas in Windsor - Essex Region.
The area under analysis is a residential district with single family homes. Thus run-off coefficient chosen for storm sewer design is **0.6**, per Table 3.2.2.7 of the ERCA SWM Manual.
The rate of runoff, defining the flow rate for a particular conduit, is associated with the time from the start of rainfall. This period, described as the time of concentration, equals the time of flow in the sewer from the most remote point in the system to the point under consideration. An additional time increment of 15 minutes is accounted for as the time required for surface flow into catch basins.

2.2 Rainfall Intensity

The design storm Intensity was calculated using the following Equation:

$$I = a/(T+b)^{c}$$

Where:

I is rainfall intensity in mm/hr

T is time in minutes measured relative to the beginning of the storm event

Intensity-Duration-Frequency (IDF) curve parameters (a,b,c,) based on 61 years (1946-2007) of historical rainfall data from Windsor Airport are summarized in Table 3.2.1.1 of ERCA SWM Manual.
Thus the Equation for intensity calculation for 1:5 Year return period is:

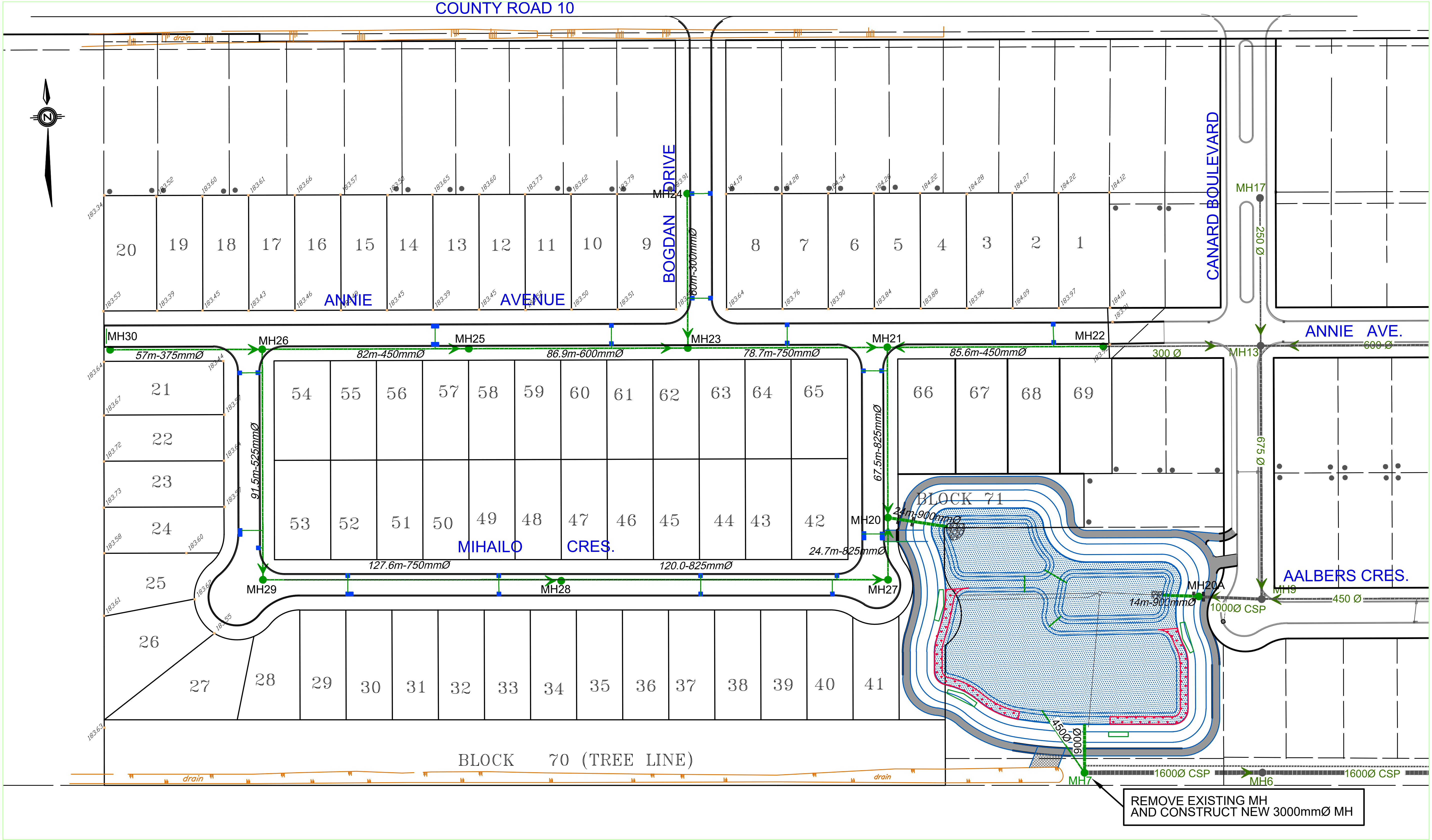
$$I = 1259/(T+8.8)^{0.838}$$

2.3 Hydraulics

Storm drainage systems are designed based on the assumption of free surface flow. The pressure created for full flow or by surcharge conditions are not considered in the sewer design. This approach is commonly employed in the design of shallow municipal storm sewer systems.
Sewer flow velocities based on flow rates, sewer size and slope are limited by the need to provide flushing capacity, at the low limit of 0.8m/s and by the need to prevent excessive mechanical corrosion (scour) by soil particles at 3.0 m/s.
The invert of inlet pipes except the last submerged section is set above the pond normal water level (N.W.L), thus the minor storm water level will not exceed the inflow sewer obvert and no backwater conditions will be created in minor storm water system.

2.4 Sizing

The sewers were designed by applying the Rational Method and an IDF relationship curve (as discussed in Section 2.2) to provide rainfall intensity for individual sewers. The sewer design chart is demonstrated in Table 3 (Appendix A). Design consideration, i.e., velocity, size and sewer depth constraints were satisfied.
The Preliminary Storm Sewer Drainage Plan is shown on Figure 8A (Appendix A),
The Preliminary Storm Sewer Layout is shown on Figure 8, the Preliminary Storm Sewer Profile is shown on Figure 9.



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CANARD VALLEY ESTATES SUBDIVISION PH. 2
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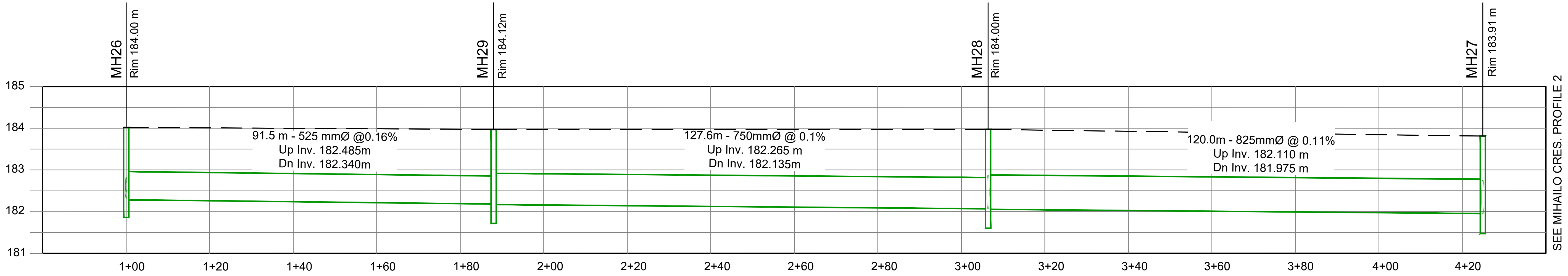
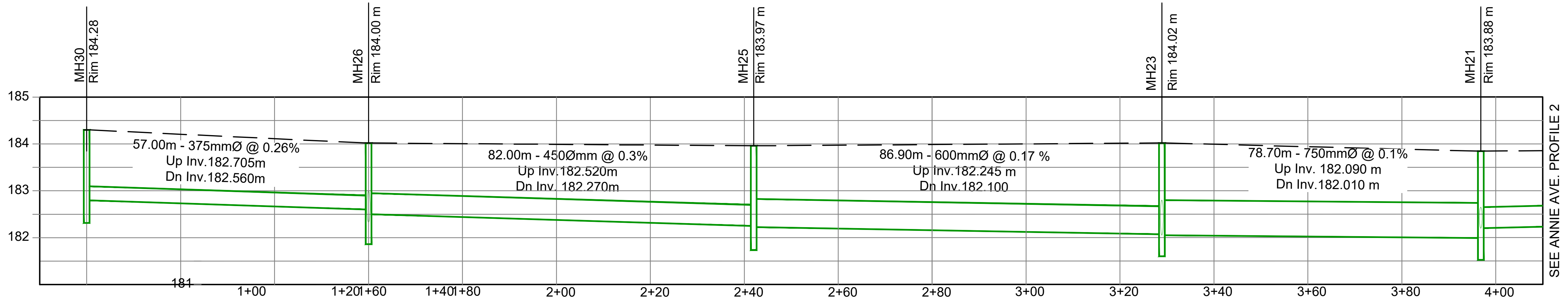
PROPOSED STORM SEWER LAYOUT

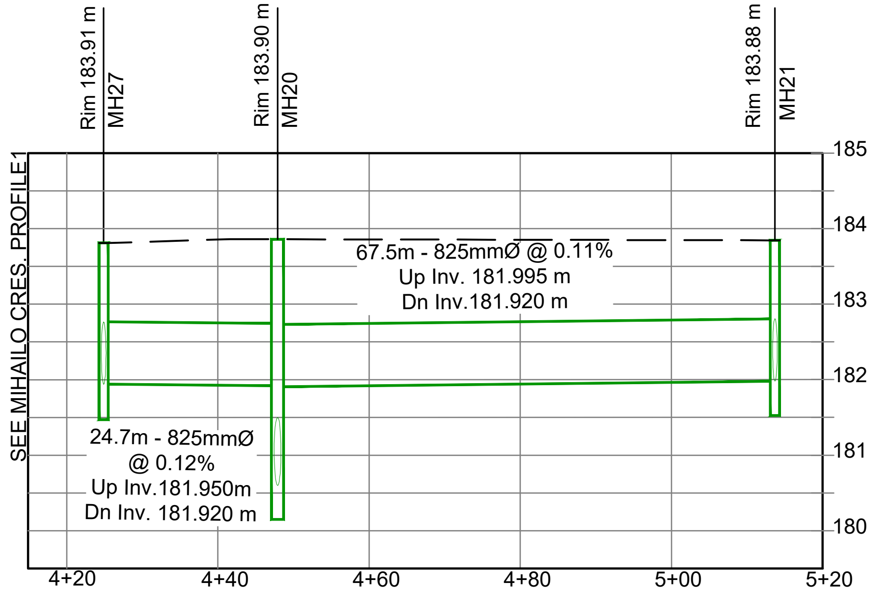
PROJECT NO.	18–743
FIGURE NO.	8
1:1500	



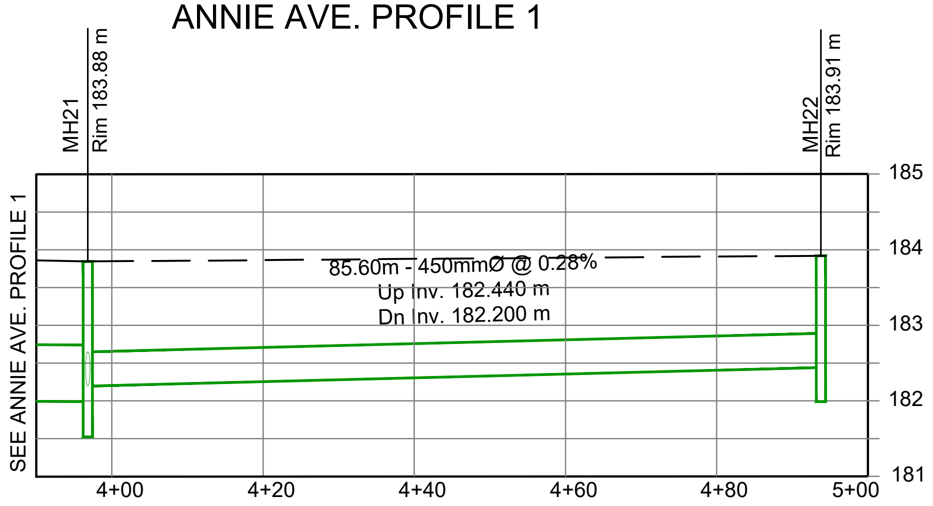
MIHAILO CRES. PROFILE 1



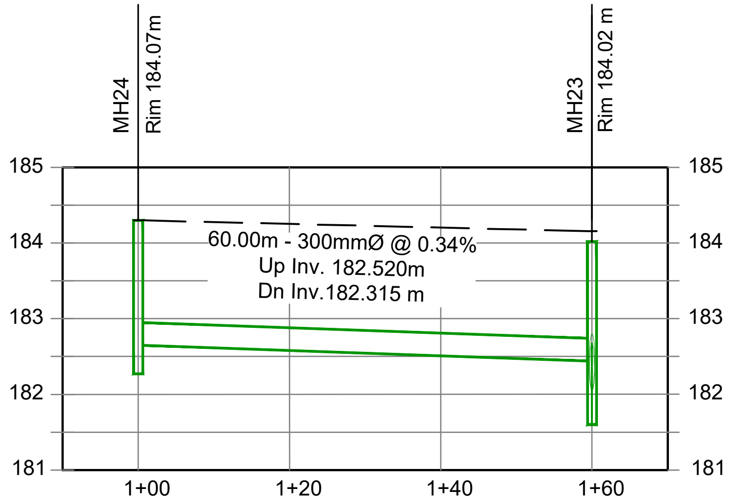
ANNIE AVE. PROFILE 1



MIHAILO CRES. PROFILE 2



ANNIE AVE. PROFILE 2



BOGDAN DR. PROFILE

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CANARD VALLEY ESTATES SUBDIVISION PH. 2
STORMWATER MANAGEMENT PLAN

PRELIMINARY STORM SEWER PROFILES

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FIGURE NO.
9

H 1:1000
V 1:100

3. STORM WATER MANAGEMENT POND DESIGN

3.1 Proposed Major System Layout

The flows generated by major storm events exceeding 1:5 year storm frequency are conveyed by the roadway overland flow routes into the storm water management facility designed as a wet pond. The oversized storage pipes constructed during phase one are considered to be a part of the major storm water management system. The modification to the existing outlet will provide the required release rate control into the municipal drainage system.

The preliminary grading plan (Figure 7) shows the clear overland flow paths with intermediate high points with an average 0.13m in depth.

The preliminary major storm water management system is shown on Figure 10, Figure11 and Figure 12.

Table 1
SWM Pond Stage-Storage Chart

ELEVATION (m)	DEPTH (m)	AREA (m^2)	VOLUME (m^3)	ACTIVE STORAGE (m^3)	NOTES
180.410	0	1250	0		Sediment forebays bottom
180.910	0.50	4900	1538		Storage cell bottom
181.410	1.00	5853	4226		
181.910	1.50	6843	7400	0	Normal Water Level (NWL)
182.410	2.00	7890	11083	3683	
183.110	2.70	9454	17153	9754	
183.710	3.30	10700	23200	15800	Maintenance access / top of bank

- The normal water level set at elevation of 181.91m
- The edge of trail is set at elevation of 183.71m (top of bank)
- Perimeter of pond is set at minimum elevation of 183.850
- The maximum pond spill elevation is 183.60m (to Romaro Drain)

3.2 Imperviousness Level

The proposed storm water management pond is design to service the Existing Phase 1 and the proposed Phase 2 of Canard Valley Estates Subdivision. The subdivision is a residential single family development. The recommended imperviousness percentage of **60%** (Table 3.7.5.1 of the Manual) was applied to the proposed Phase 2 of the development.

The majority of the existing Phase 1 lands are fully developed. The actual impervious percentage was calculated based on the 2019 aerial photo. The actual calculated imperviousness was applied for Phase1. The calculation details are demonstrated on Figure 13.

The lands under current analysis are located in the King Creek sub-watershed and release flows into McGregor Drain sewer. The imperviousness percentage of the existing McGregor Drainage area was also calculated for the purpose of dynamic modelling of the proposed system in conjunction with the existing. The calculation results are shown on Figure 14.

3.3 Allowable Release Rate

The allowable release rate is based on the capacity of the existing McGregor Drain, an enclosed 36" (910mm) diameter pipe.

The Schedule of Assessment of the Drainage Report *"Township of Anderdon, McGregor Drain''* prepared by C.G. Russell Armstrong Associates Ltd. in 1970 assesses 14.5 ac (5.87 ha.) for Emil Breschuk Lands. The current development is in the area of the former Emil Breschuk farmland; however, 3.3 ha of the lands are already developed and release the flows into McGregor Drain and should be excluded from the allowable release rate calculations, as shown on Figure 15.

The sewer sizing calculations for McGregor Drain are unavailable. The assumption was made that the sewer was designed to 1:2 year storm frequency as is common practice for municipal drain sizing.

Allowable release rate:

Area 'A' = 2.54ha

Runoff Coefficient 'C' = 0.2

Time of Concentration:

$T_c = 20\text{min} + T_{\text{upstream}} + T_{\text{pipe}} = 30\text{min}$

Intensity: $I = a/(T+b)^{c}$

$I = 854/(T+7)^{0.818}$ (Table 3.2.1.1 of the ERCA SWM Manual)

I = 44.53 (mm/hr)

Flow rate: Q= ACI/360

Q= (2.54 x 0.2 x 44.53)/ 360 = 0.063 m^3/s

The release rate from the development shall not exceed **0.063 m^3/s (0.003 m^3/s /ha)** during any storm event.

3.4 Active Storage Requirements

The flows from the subdivision storm drainage system are released into the existing enclosed municipal drain. In this situation backwater conditions are expected.

The required storage volumes are calculated assuming zero release rate. The standard 100-year Storage volume is calculated as the specific storage depth of runoff multiplied by the catchment area.

For hydrologic soil group D, the equation is as follows:

Storage Depth (mm) = 72+ 0.33x where x= imperviousness % > 50%
(Eq. 3.3.2.1.d ERCA SWM Manual)

Storage Volume Requirements for Phase 1:

To SWM pond: Storage Depth = (72+0.33x55) = 90.2mm
Area = 7.88ha
Storage Volume = (78800x0.0902) = 7108m^3

To swale: (calculated imperviousness 35%, use 50%)
Storage Depth = (72+0.33x50) = 88.5mm
Area = 0.84ha
Storage Volume = (8400x0.0885)= 743m^3

Swale: (grass)
Storage Depth = 72mm
Area = 0.51ha
Storage Volume = (5100x0.072) = 367m^3
Subtotal Ph. 1 Storage = 8218m^3

Storage Volume Requirements for Phase 2 and SWM pond:

To SWM pond: Storage Depth = (72+0.33x60) = 91.8mm
Area = 7.65ha
Storage Volume = (76500x0.0918) = 7023m^3

SWM pond (100% imp. at N.W.L .and Trail) => 65% imperviousness):
Storage Depth = (72+0.33x65) = 93.5mm
Area = 1.29ha
Storage Volume = (12900x0.0935) = 1206m^3
Subtotal Ph2. Storage = 8229m^3

Total Storage Required: 16,447m^3

Active Storage Provided:

Storage in SWM pond (to overflow weir elev. 183.600)		14,652m^3
Storage in sewer:	Phase 1 storm sewer	174m^3
	Phase 2 storm sewer	309m^3
	1600mm Dia. storage pipes and outlet	865m^3
	Swale incl. subdrain in sewer stone	200m
	Total Storage Provided:	**16,200m^3**

Total provided storage not including storage in the manholes, catchbasins, and on-road storage is 98.5% of the required 100-year storage with zero release rate.

Hydrologic and hydraulic modelling was conducted to demonstrate that the system has sufficient capacity for detention of the stormwater flows generated from the storm events up to 1:100 year frequency and that the proposed development will not have negative impact on the existing system and the adjacent lands during storm events exceeding 1:100year storm. The modelling approach and results are described in Section 4 of the current report.

4. HYDROLOGIC AND HYDRAULIC MODELLING

4.1 Software Application

The hydrologic and hydraulic analysis for this development was completed using the **PCSWMM Professional 2D software version 7.4**. PCSWMM is advanced modelling software for stormwater, wastewater and watershed systems. Surface flows routed overland (major systems such as roads, swales, street sags, storage areas) can easily and accurately be combined with underground flows (minor system, such as sewer infrastructure).

4.2 Design Storm Distributions

The selection of the design storm distribution was based on the recommendations of the current ERCA SWM Manual. The following storm distributions were used for the model:

- Water Quality Storm (Chicago 2-Year,4-Hour, 32mm Depth,1 5 min. Time Step)
- 5-Year Design Storm (Chicago 4-Hour, 49.5mm Depth, 15 min. Time Step)
- 100-Year Design Storm (Chicago 4-Hour, 81.6mm Depth, 15 min. Time Step)
- SCS Type II 24-Hour Design Storm, 100-Year, 108mm (15 min. Time Step)
- Urban Stress Test (Chicago 100 -Year 24-Hours,150mm Depth, 15min. Time Step)

The custom rainfall time series were created for each storm distribution based on data published in Appendix B of the ERCA SWM Manual.

4.3 Hydrologic Modelling

The Green-Ampt Infiltration method was selected to estimate infiltration losses. This method uses model parameters based on soil type. The model input parameters represent Hydrologic Soil Group D.

Flow Properties:	Suction Head	Su	180mm
	Conductivity	Ks	0.5mm/hr
	Initial deficit:		
	Minor storms	IMD, dry	0.21
	Major storms	IMD, normal	0.10
	Internal runoff routed		100%
	Internal routing		various

The study area includes the existing phase and the proposed phase of Canard Valley Estates Subdivision. The area was divided into number subcatchments representing backyard drainage directly to sewer and roadway and front yard drainage to catchbasins.

In order to estimate tail water conditions at the outlet to McGregor Drain, external McGregor Drain drainage area is represented by 7 subcatchments discharging directly into manholes.

Physical sub-catchment properties:

Drainage area (ha)	varies
Width (m)	varies
Impervious Area	
No Depression	25%
Manning Roughness	0.013
Depression Depth	2.5mm
Pervious Area	
Manning Roughness	0.24
Depression Depth	7.5mm (grass)
	10mm (open field)
Overall Imperviousness	
Future Single family homes	60%
Existing Development	as calculated

4.4 Hydraulic Modelling

4.4.1 General

A detailed dual drainage model was created to represent the proposed and the existing storm water management system of Canard Valley Estates Subdivision.
The model input parameters for the new development are shown on the attached Preliminary Grading Plan (Figure 7) and preliminary storm sewer design chart (Appendix A). The input parameter for the existing development was based on the As-Constructed Drawings for Canard Estates Subdivision, dated September 20, 2005.

The details of McGregor Drain upstream drainage system, as well as overland flow routes are uncertain. Thus, the assumption was made that all flows, after surcharging the underground system, by-pass the junctions and flow overland to King Creek Drain. It is assumed the junctions will surcharge 0.3m before by-passing the flows. The elements input data of the underground system is based on available topographic survey and the profile drawing for McGregor Drain, included in the 1970 Drainage Report. The McGregor Drain model is schematic and was created for SWM system tail water conditions analysis only.

The model is represented by:

1) Junctions representing existing tee manholes and road high points.
2) Storage nodes representing storm water management pond and existing and proposed manholes and catchbasins.
3) Conduits representing underground piping as well as open channels representing on-surface roadway drainage.
4) Outlets representing catchbasins grates flow capacity.
5) Orifice at subdivisions storm system outlet
6) Weirs representing overland flow openings
7) System outfalls.

4.4.2 Storage Nodes

A storage curve was created to model the storm water management pond. The manholes and catchbasins storage nodes are specified as functional storage equal to the given structure size.

4.4.3 Inlets Control

The proposed catchbasins for the development are grate inlet type. The model assumes the inlets are controlled by flow capacity of the catchbasins grate. The custom rating curves were created based on *Design Chart 4.19 of the MTO Drainage Management Manual.*

4.4.4 System Outlet

Model system outfall is located at King Creek Drain, which outlets to Canard River. The McGregor Drain outlet invert is 1.86m above King Creek Drain culvert invert and set at elevation of 181.00m. The Essex Region Floodline Mapping Study (Map No. ER5-34) notes floodline levels at Big Creek Drain Outlet as:

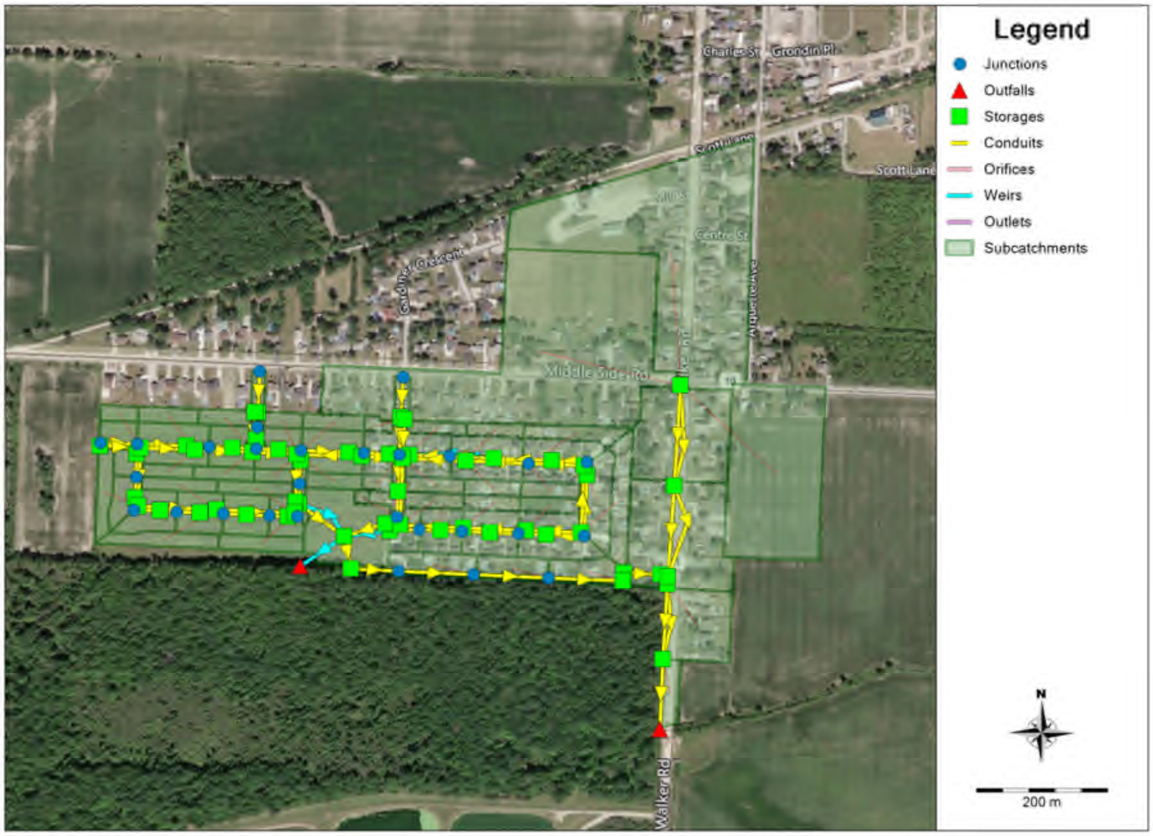
180.95m for 1:100 Year Flood Level
182.10m for Regional Storm Flood Level

The system outlet elevation is above a 1:100 year flood level, thus free outlet conditions were assumed for all storm events up to and including 1:100 year storm. The tailwater elevation of 182.100 was applied for Urban Stress Test model.
An additional outfall was specified representing overflow weir to Parent Trembley Drain for the storms exceeding 1:100-Year storm, such as Urban Stress Test.

4.4.5. Routing Method

Hydrodynamic routing method was chosen in the modelling. Hydrodynamic routing has the capability of modelling backwater effects, flow reversal, surcharging, looped connections, tidal outfalls and pressure flow throughout the drainage network.

The Schematic Model Representation is shown below:



4.5 Modelling Summary

The modelling results are summarized in following table:

Table 2

Design Storm	Peak Pond Inflow (m^3/s)	Max. Pond Elevation (m)	Min. Freeboard (m)	Active Pond Storage (m^3)	Peak Release Rate at Outlet to McGregor Drain (m^3/s)	Remarks
URBAN STRESS TEST	**4.386**	**183.65**	**0.06**	**15,202**	**0.058**	See Paragraph 6
SCS TYPE II 100-Yr, 24-Hr	**3.952**	**183.41**	**0.30**	**12,689**	**0.063**	Max. Water Level
CHICAGO 100-Yr, 4-Hr	**4.540**	**183.28**	**0.43**	**11,405**	**0.061**	Max. Pond Inflow
CHICAGO 5-Yr, 4-Hr	**2.750**	**182.72**	**0.99**	**6,196**	**0.050**	
CHICAGO 2-Yr, 4-Hr	**1.770**	**182.39**	**1.32**	**3,560**	**0.043**	Quality Storm

The modeling results show:

- The maximum water level in the pond occurs during the 100-Year, 24 hours duration storm (SCS Type II distribution). The maximum water level is 0.3m below pond top of bank and gives 0.54m of freeboard to the lowest house elevation of 183.95m (Unit 35, Phase 1). The rest of the houses are set at elevations between 184.20 m and 184.70, which gives at least 0.79m freeboard to the elevation at the house.

- The maximum conveyance system loading occurs during more intense 100-Year, 4 hours duration storm (Chicago distribution). The ponding above catchbasins will not exceed 0.3m during 1:100 -Year Storm event for all proposed and existing catchbasins except one set of the existing catchbasins.

- The Hydraulic Grade Line (HGL) in storm sewer system is at least 0.63m below the ground during 1:5-Year storm event for the proposed phase of development and will remain under ground for the existing phase.

- The maximum release rate from the subdivision will not exceed the allowable release rate of 0.063m^3/s under any conditions.

- Modelling of Urban Stress Test demonstrate the maximum water level in the pond is 0.3m below the lowest house opening.

The resultant modelling profiles are demonstrated in Appendix B. (B1 - Minor Conveyance System and Outlet Sewer Profiles; B2 - Major Conveyance System Profiles).

5. STORM WATER QUALITY CONTROL

5.1 Storage Requirements

The storm water management system is designed to protect the quality of receiving waters in most efficient way. The ‘first flush’ volume carrying the major load of the suspended solids and associated pollutants will be retained for later release. The ‘first flush’ will be directed via storm sewers to the storm water management pond. Catchbasins will have 600mm sumps with purpose of trapping the sediments and debris.

The storm water management pond is designed as a wet pond which releases water at a slower rate compared to the influent water. The pond is designed to provide a normal protection level with 70% long term suspended solids removal. Per Table 3.2 of the Ministry of the Environment SWM Planning and Design Manual, the required water quality storage is:

Phase 1 (55% imperviousness)	$110m^3/ha$	($40m^3/ha$ extended detention $70m^3/ha$ in permanent pool)
Phase 2 (60% imperviousness)	$117m^3/ha$	($40m^3/ha$ extended detention $77m^3/ha$ in permanent pool)

The required storage:

Phase 1 (7.88ha)	=>	$315m^3$	(extended detention)
		$552\ m^3$	(permanent pool)
Phase 2 (7.65ha)	=>	$306\ m^3$	(extended detention)
		$589\ m^3$	(permanent pool)
Total	=>	$621m^3$	(extended detention)
		$1141\ m^3$	(permanent pool)

The pond has sufficient storage volumes to provide storm water quality control.

5.2 Quality Storm Modelling

The water quality storm modelling was run using Chicago 2-Year, 4-Hour storm distribution for 32mm rainfall depth, as recommended in the ERCA Design Manual.
As demonstrated in Table 2 the model results are:

- Maximum elevation	182.40m
- Maximum storage	$3570m^3$
- Peak Discharge	$0.043m^3/s$
- Maximum pond outflow	$0.053\ m^3/s$
- Detention time	over 24 Hours

5.3 Sediment Forebay Design

The pond has two interconnected sediment forebays, one for each phase.
The sediment forebays are designed in accordance with the recommendations of 'Stormwater Management Planning and Design Manual' by Ontario Ministry of the Environment dated March 2003.

Forebays sizing check:pl

Forebay 1 volume	760 m^3
Forebay 2 volume	820m^3
Total	1580m^3

Total forebays volume is 21.4% of permanent pool volume (7400m^3) => ***OK***

Settling Length:

$Dist = \sqrt{(rQ_p/V_s)}$

Where: $Dist$ = *forebay length*
r = *length-to-width ratio* = *2.5(Forebay 1); 1.5(Forebay 2)*
Q_p = *peak flow rate from the pond during design quality storm* = $0.053 m^3/s$
V_c = *settling velocity* = $0.0003 m/s$

$Dist1 = \sqrt{(2.5\times 0.053/0.0003)} = 21.0m$ *Length of Forebay1 at bottom= 39m* => ***OK***
$Dist1 = \sqrt{(1.5\times 0.053/0.0003)} = 16.3m$ *Length of Forebay2 at bottom= 31.5m* => ***OK***

5.4 Facility Monitoring and Maintenance Requirements

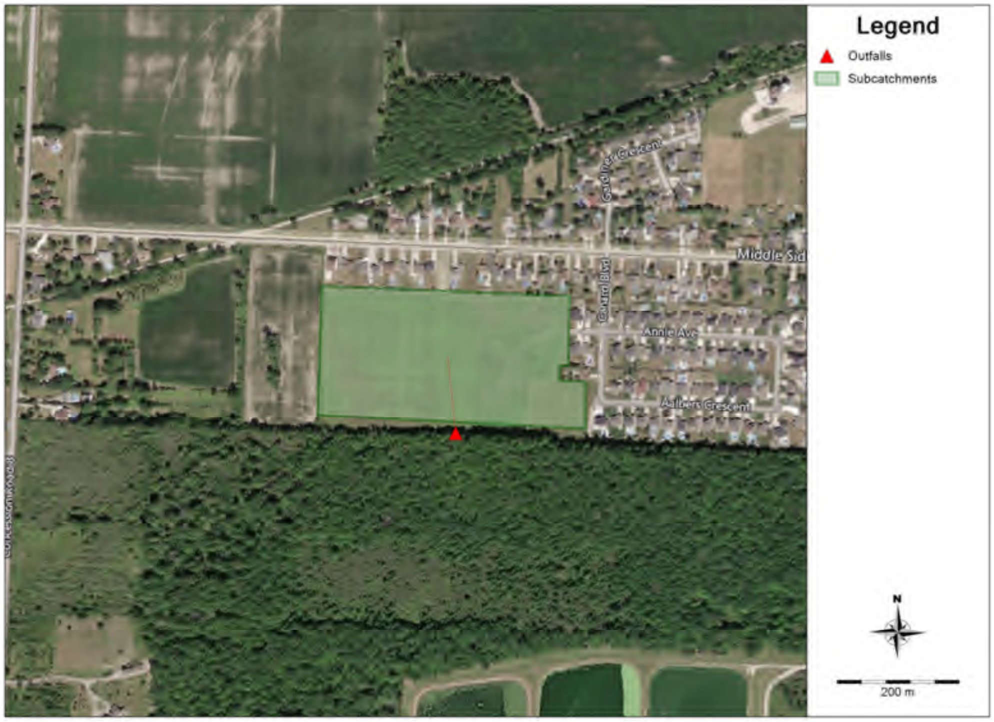
A periodic inspection of the facility is required for monitoring of sediment accumulation. During the first two years of operation, inspections should be made after every significant storm event. After this initial operation period is complete, then annual inspection is required. Schedule of maintenance should be identified by inspection results.

On a periodic basis, sediment deposited in the receiving forebays will accumulate to a depth such that cleanout is required to maintain an adequate depth. This will generally be indicated when aquatic plants emerge, or stand out above the water's surface. A period of 15 to 20 years is expected as the accumulation rates of clay are extremely slow.

A maintenance pipe is proposed to drain the pond using a portable lift pump, when maintenance is required. Sediment should be properly handled and disposed of.
Grass cutting and weed control is required as part of regular pond maintenance procedure.

6. MAJOR STORM EVENTS IMPACT ANALYSIS

The above described modelling show the proposed stormwater management system has sufficiency capacity to store the volumes generated by storm events up to and including 1:100 year storm frequency, applying the required downstream flow release restriction.
During major storm events exceeding 1:100 year storm frequency the stormwater volumes accumulated in the pond will overflow to Parent Trembley Drain when water level in the pond will reach the specified overflow weir elevation.
Currently the undeveloped site release all storm water flows to Parent Trembley Drain.
In order to analyze the impact of overflow volumes generated by the proposed development, an additional model for the existing conditions was created. It was assumed the subject area releases pre-developed flows to Parent Trembley Drain with conditions of 0% imperviousness and 0.1% average slope.
Below is the schematic model representation.



The results were compared with post-developed flows at overflow weir outlet for Urban Stress Test. The results are summarized in the table shown below.

Pre-developed Conditions		**Post-developed Conditions**	
Peak flow (m^3/s)	Total Flow Volume (m^3)	Peak Flow (m^3/s)	Total Flow Volume (m^3)
0.260	8,170	0.244	4,743

The results show that the post-developed peak flow will not exceed the pre-developed peak flow and the total stormwater volumes are reduced by 42%, therefore the proposed development will be beneficial to Parent Trembley Drain, and will not have any negative impact to adjacent lands.

7. SUMMARY

To satisfy the requirements of the storm water management plan for Canard Valley Estates Subdivision, it is proposed to implement both quantitative and qualitative protection measures to protect the quality of surface and groundwater resources.

i) On-Site Source Controls

- Following construction of houses, side yards and backyards of each lot to be graded toward a catchbasin in a depression in rear yard. All roofs to be discharged onto splash pads.
- Subdrains along the road (outside the pavement) will collect and treat runoff from the boulevard and road subgrades.
- Filtered building foundation drainage will be pumped onto splash pads.

ii) Conveyance System Controls

- Sumps of minimum 0.600 m depth in catchbasins will retain larger solid particles.
- CB Traps will aid in preventing oils from reaching the pond, allowing surface skimming.

iii) End-of-Pipe Controls

- An extended detention wet pond that will establish viable permanent habitat and restrict runoff to the existing municipal drainage to less than estimated allowable release rate.

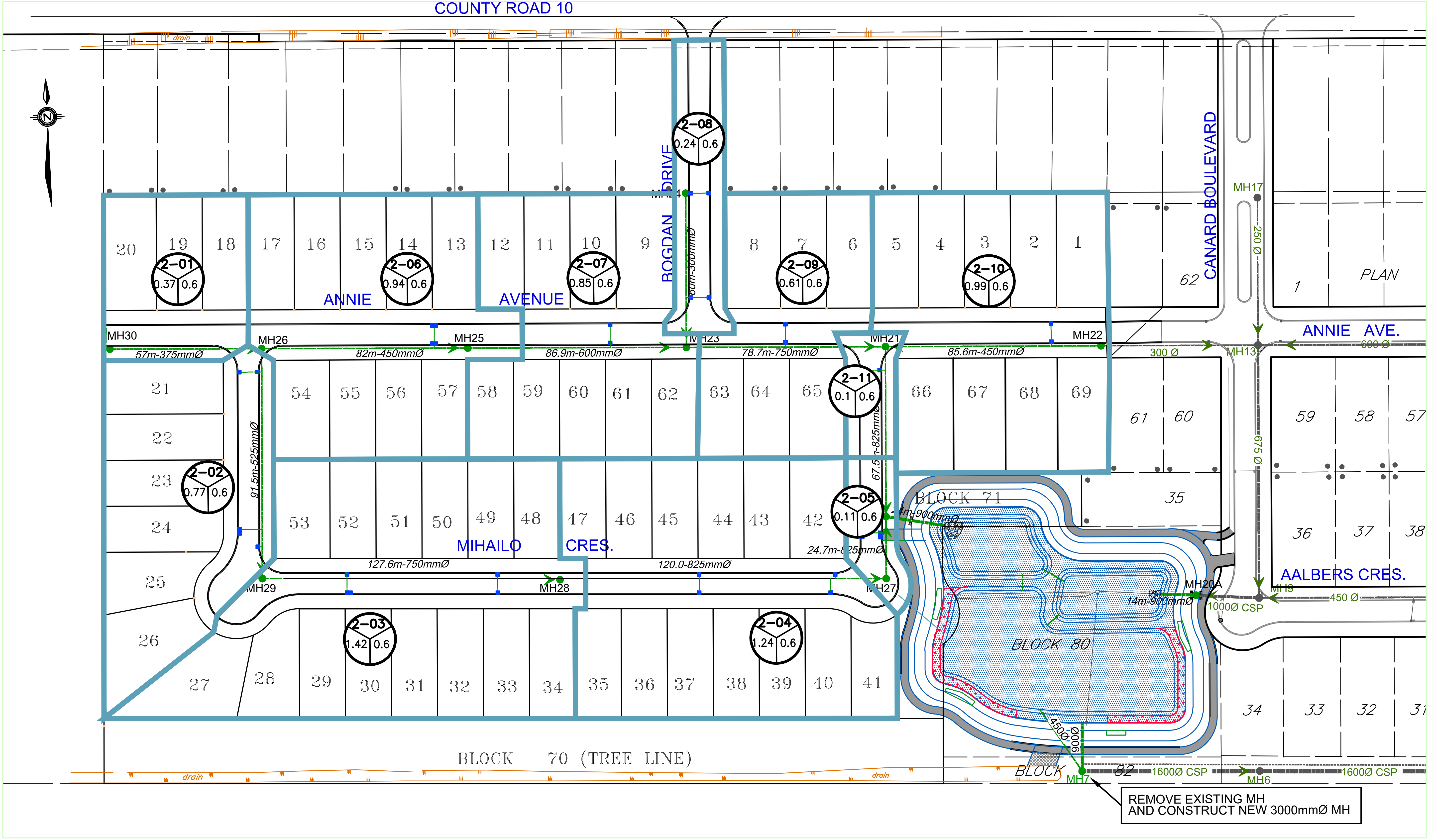
iv) Construction Period Runoff Quality Protection

- Topsoil stripping and stockpiling limitations and controls
- Silt intercepting filters to be installed in catchbasins to retain suspended solids
- Level boulevard grading behind curbs
- Silt fencing on development side of the wet pond
- Regular cleaning of roadways throughout the house building period

The proposed storm water management plan meets stormwater flood proofing, quantity and quality requirements and will allow an orderly development of Phase 2 of Canard Valley Estates Subdivision and will improve runoff treatment of the existing Phase 1 development.

APPENDIX A

Storm Sewer Design



RC SPENCER ASSOCIATES INC.
Consulting Engineers

LEGEND:

0-00 — DENOTES AREA NUMBER

0.00 — DENOTES AREA (Ha)

0.0 — DENOTES RUNOFF COEFFICIENT

CANARD VALLEY ESTATES SUBDIVISION PH. 2
STORMWATER MANAGEMENT PLAN

PROPOSED STORM DRAINAGE PLAN

PROJECT NO.
18-743

FIGURE NO.
8A

1:1500

TABLE 3

STORM WATER MANAGEMENT - METRIC UNITS

PROJECT NAME: **CANARD ESTATES SUBDIVISION PH.2**
PROJECT NO. 18-743
CLIENT:
DATE: June 2022

DESIGN CRITERIA

INTENCITY $I = a/(T + b)^C$

STORM CURVE: 5 YEAR C FACTORS: 0.60
ENTRY TIME: 20 MINUTES
MIN. VELOCITY: 0.8 m/s
MAX. VELOCITY: 3.0m/s n FACTOR: 0.013

a = 1259
b = 8.8
c = 0.838

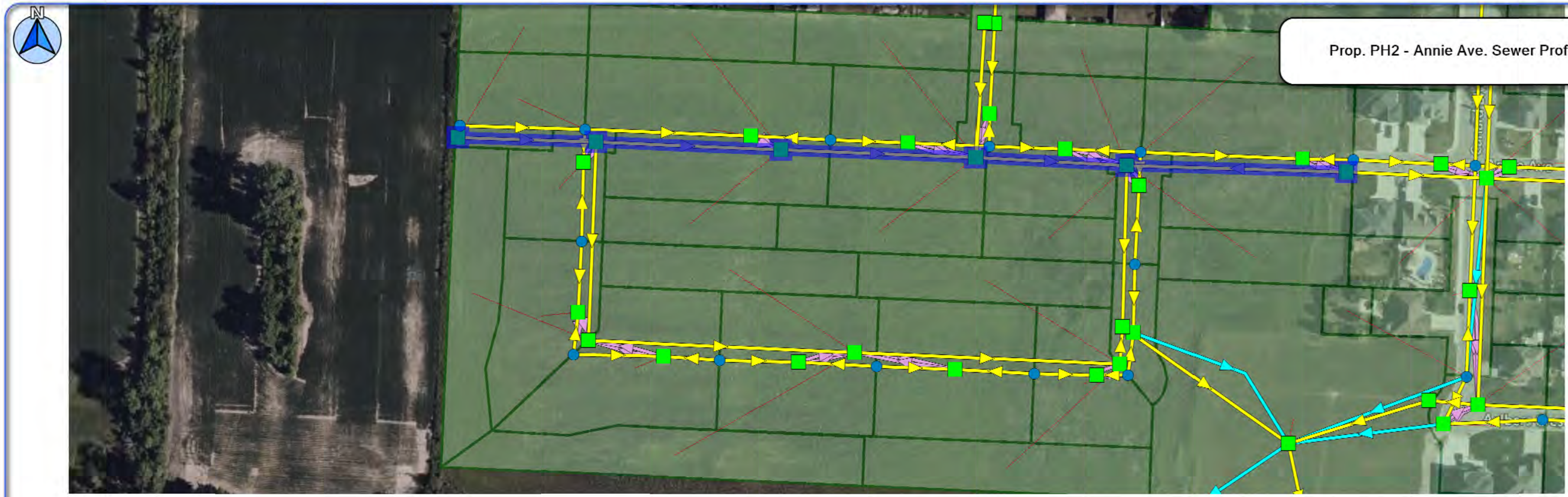
LOCATION				AREA		A X C				RAINFALL INTENSITY			Q	SEWER DESIGN									
STREET	Location	FROM	TO	AREA ha	TOTAL AREA ha	RUNOFF COEFF. "C"	INCR. AXC	TOTAL LAT. AXC	TOTAL SEWER AXC	FLOW SECT.	TIME ACCUM.	INTEN-SITY mm/hr	REQ'D cms	PIPE DIA.	SLOPE %	CAP. cms	VEL. m/s	LENGTH m	FLOW TIME MINS.	STO-RAGE CM	FALL In Sewer	INV. U.S.	INV. D.S.
Annie Ave.	2-01	MH30	MH26	0.37	0.37	0.60	0.222		0.222	1.19	15.00	88.40	0.054	0.375	0.26	0.089	0.80	57.0	1.19	6	0.145	182.705	182.560
Mihailo Cres.	2-02	MH26	MH29	0.77	1.14	0.60	0.462	0.222	0.684	1.93	16.19	84.87	0.161	0.525	0.16	0.171	0.79	91.5	1.93	20	0.145	182.485	182.340
Mihailo Cres.	2-03	MH29	MH28	1.42	2.56	0.60	0.852	0.684	1.536	2.64	18.12	79.74	0.339	0.750	0.10	0.356	0.80	127.6	2.64	56	0.130	182.265	182.135
Mihailo Cres.	2-04	MH28	MH27	1.24	3.80	0.60	0.744	1.536	2.280	2.22	20.76	73.72	0.466	0.825	0.11	0.482	0.90	120.0	2.22	64	0.136	182.110	181.975
Mihailo Cres.	2-05	MH27	MH20	0.11	3.91	0.60	0.066	2.280	2.346	0.44	22.98	69.39	0.451	0.825	0.12	0.497	0.93	24.7	0.44	13	0.030	181.950	181.920
Annie Ave.	2-06	MH26	MH25	0.94	0.94	0.60	0.564		0.564	1.38	15.00	88.40	0.138	0.450	0.30	0.157	0.99	82.0	1.38	13	0.249	182.520	182.270
Annie Ave.	2-07	MH25	MH23	0.85	1.79	0.60	0.510	0.564	1.074	1.63	16.38	84.31	0.251	0.600	0.17	0.251	0.89	86.9	1.63	25	0.145	182.245	182.100
Bogdan Dr.	2-08	MH24	MH23	0.24	0.24	0.60	0.144		0.144	1.25	15.00	88.40	0.035	0.300	0.34	0.056	0.80	60.0	1.25	4	0.205	182.520	182.315
Annie Ave.	2-09	MH23	MH21	0.61	2.64	0.60	0.366	1.218	1.584	1.63	18.02	79.99	0.351	0.750	0.10	0.356	0.80	78.7	1.63	35	0.080	182.090	182.010
Annie Ave.	2-10	MH22	MH21	0.99	0.99	0.60	0.594		0.594	1.50	15.00	88.40	0.145	0.450	0.28	0.151	0.95	85.6	1.50	14	0.240	182.440	182.200
Mihailo Cres.	2-11	MH21	MH20	0.10	3.73	0.60	0.060	2.178	2.238	1.26	19.65	76.13	0.472	0.825	0.11	0.478	0.89	67.5	1.26	36	0.075	181.995	181.920
OUTLET		MH20	POND		7.64	0.60	0.000	4.584	4.584	0.28	23.42	68.59	0.871	0.900	0.25	0.905	1.42	24.0	0.28	15	0.060	180.570	180.510
																				301			
																		Additional Storage:					
														1.600				411.0		826			
														0.600				71.0		20			
														0.150				411.0		7			
														0.900				19.0		12			

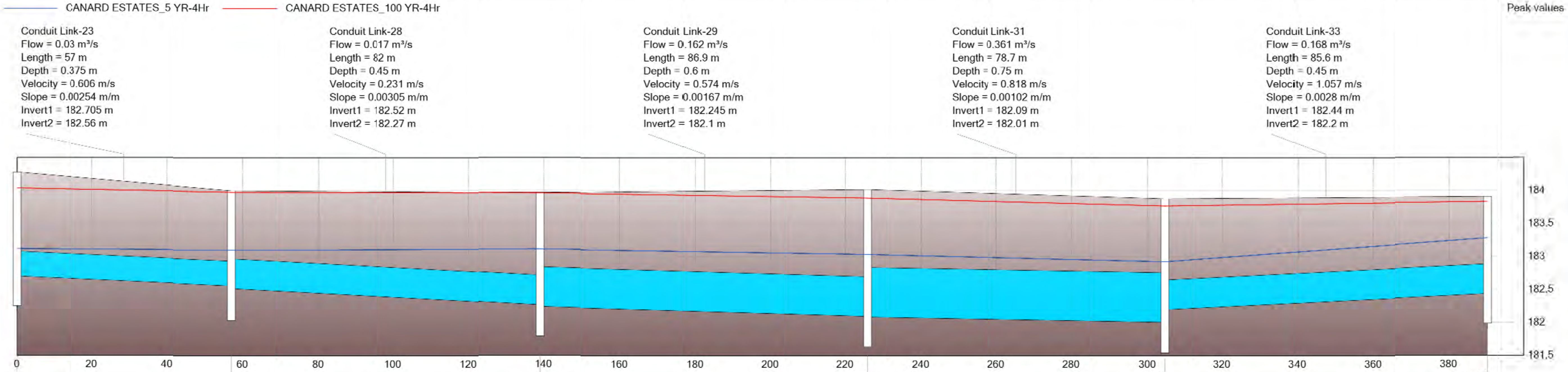
APPENDIX B

Hydrologic and Hydraulic Modelling Results

B1

Prop. PH2 - Annie Ave. Sewer Profile 1:5Yr. STM





Conduit Link-23
Flow = 0.03 m³/s
Length = 57 m
Depth = 0.375 m
Velocity = 0.606 m/s
Slope = 0.00254 m/m
Invert1 = 182.705 m
Invert2 = 182.56 m

Conduit Link-28
Flow = 0.017 m³/s
Length = 82 m
Depth = 0.45 m
Velocity = 0.231 m/s
Slope = 0.00305 m/m
Invert1 = 182.52 m
Invert2 = 182.27 m

Conduit Link-29
Flow = 0.162 m³/s
Length = 86.9 m
Depth = 0.6 m
Velocity = 0.574 m/s
Slope = 0.00167 m/m
Invert1 = 182.245 m
Invert2 = 182.1 m

Conduit Link-31
Flow = 0.361 m³/s
Length = 78.7 m
Depth = 0.75 m
Velocity = 0.818 m/s
Slope = 0.00102 m/m
Invert1 = 182.09 m
Invert2 = 182.01 m

Conduit Link-33
Flow = 0.168 m³/s
Length = 85.6 m
Depth = 0.45 m
Velocity = 1.057 m/s
Slope = 0.0028 m/m
Invert1 = 182.44 m
Invert2 = 182.2 m

Storage MH30
Max. CWSEL = 183.1202 m
Rim Elev. = 184.28 m
12/07/2021 01:50AM

Storage MH26
Max. CWSEL = 183.0991 m
Rim Elev. = 184 m
12/07/2021 01:50AM

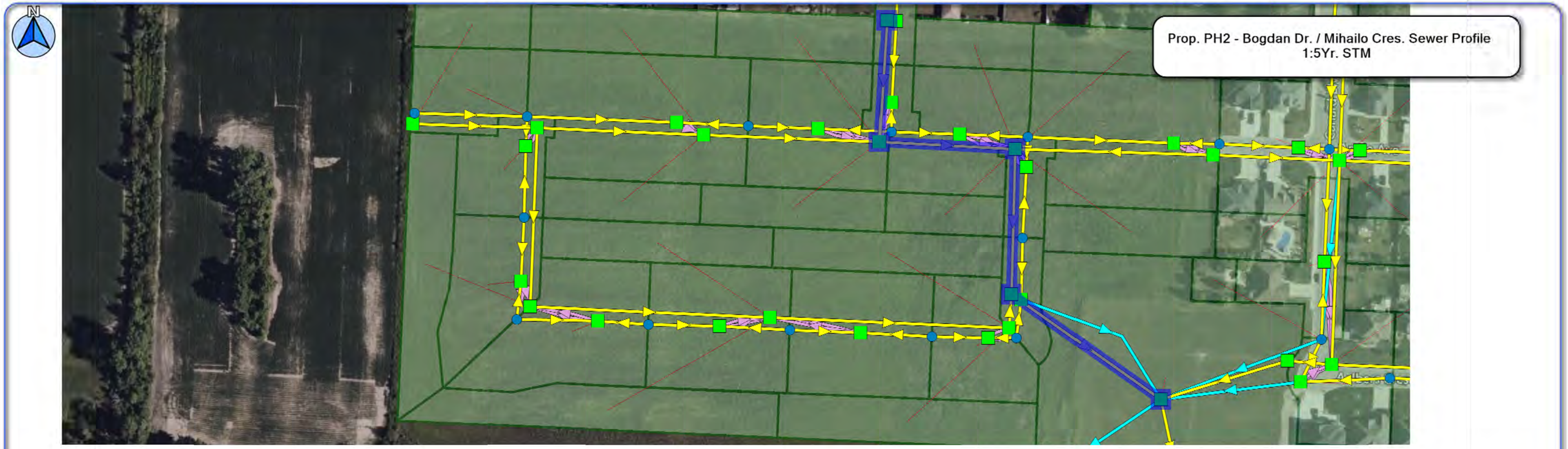
Storage MH25
Max. CWSEL = 183.1168 m
Rim Elev. = 183.97 m
12/07/2021 01:45AM

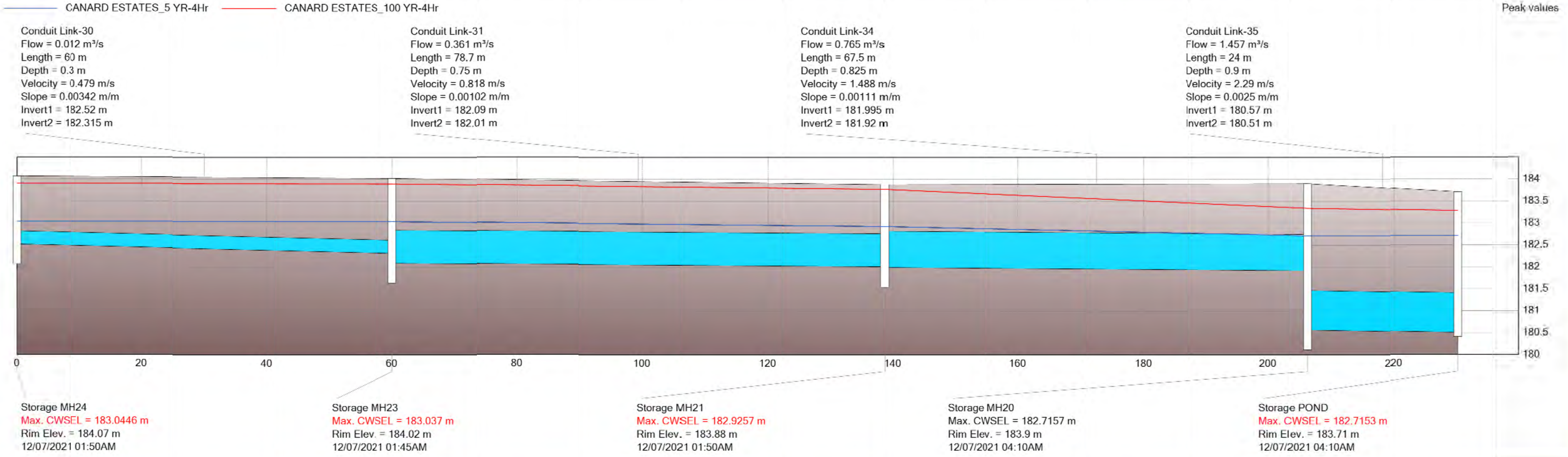
Storage MH23
Max. CWSEL = 183.037 m
Rim Elev. = 184.02 m
12/07/2021 01:45AM

Storage MH21
Max. CWSEL = 182.9257 m
Rim Elev. = 183.88 m
12/07/2021 01:50AM

Storage MH22
Max. CWSEL = 183.2843 m
Rim Elev. = 183.91 m
12/07/2021 01:45AM

Prop. PH2 - Bogdan Dr. / Mihailo Cres. Sewer Profile
1:5Yr. STM





Conduit Link-30
Flow = 0.012 m³/s
Length = 60 m
Depth = 0.3 m
Velocity = 0.479 m/s
Slope = 0.00342 m/m
Invert1 = 182.52 m
Invert2 = 182.315 m

Conduit Link-31
Flow = 0.361 m³/s
Length = 78.7 m
Depth = 0.75 m
Velocity = 0.818 m/s
Slope = 0.00102 m/m
Invert1 = 182.09 m
Invert2 = 182.01 m

Conduit Link-34
Flow = 0.765 m³/s
Length = 67.5 m
Depth = 0.825 m
Velocity = 1.488 m/s
Slope = 0.00111 m/m
Invert1 = 181.995 m
Invert2 = 181.92 m

Conduit Link-35
Flow = 1.457 m³/s
Length = 24 m
Depth = 0.9 m
Velocity = 2.29 m/s
Slope = 0.0025 m/m
Invert1 = 180.57 m
Invert2 = 180.51 m

Storage MH24
Max. CWSEL = 183.0446 m
Rim Elev. = 184.07 m
12/07/2021 01:50AM

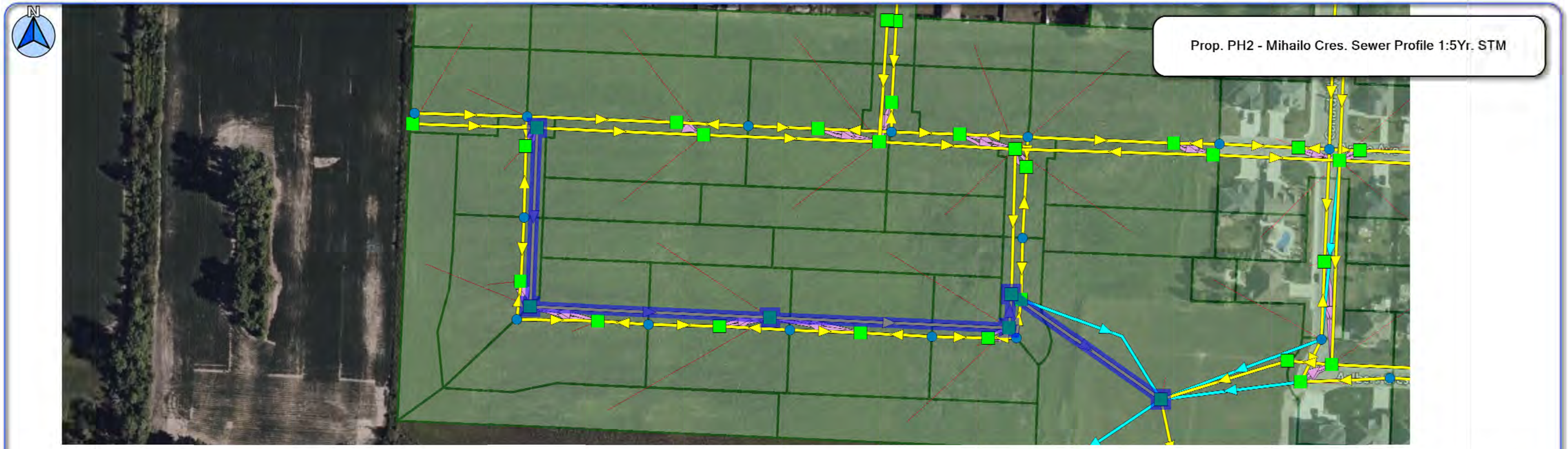
Storage MH23
Max. CWSEL = 183.037 m
Rim Elev. = 184.02 m
12/07/2021 01:45AM

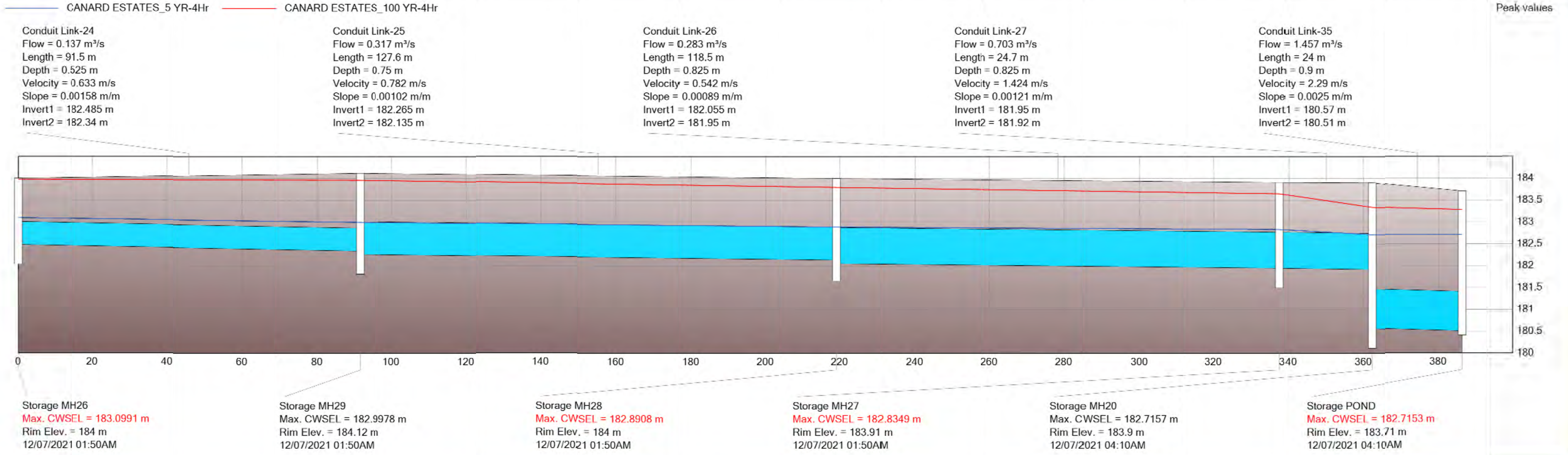
Storage MH21
Max. CWSEL = 182.9257 m
Rim Elev. = 183.88 m
12/07/2021 01:50AM

Storage MH20
Max. CWSEL = 182.7157 m
Rim Elev. = 183.9 m
12/07/2021 04:10AM

Storage POND
Max. CWSEL = 182.7153 m
Rim Elev. = 183.71 m
12/07/2021 04:10AM

Prop. PH2 - Mihailo Cres. Sewer Profile 1:5Yr. STM




CANARD ESTATES_5 YR-4Hr
CANARD ESTATES_100 YR-4Hr

Peak values

Conduit Link-24
Flow = 0.137 m³/s
Length = 91.5 m
Depth = 0.525 m
Velocity = 0.633 m/s
Slope = 0.00158 m/m
Invert1 = 182.485 m
Invert2 = 182.34 m

Conduit Link-25
Flow = 0.317 m³/s
Length = 127.6 m
Depth = 0.75 m
Velocity = 0.782 m/s
Slope = 0.00102 m/m
Invert1 = 182.265 m
Invert2 = 182.135 m

Conduit Link-26
Flow = 0.283 m³/s
Length = 118.5 m
Depth = 0.825 m
Velocity = 0.542 m/s
Slope = 0.00089 m/m
Invert1 = 182.055 m
Invert2 = 181.95 m

Conduit Link-27
Flow = 0.703 m³/s
Length = 24.7 m
Depth = 0.825 m
Velocity = 1.424 m/s
Slope = 0.00121 m/m
Invert1 = 181.95 m
Invert2 = 181.92 m

Conduit Link-35
Flow = 1.457 m³/s
Length = 24 m
Depth = 0.9 m
Velocity = 2.29 m/s
Slope = 0.0025 m/m
Invert1 = 180.57 m
Invert2 = 180.51 m

Storage MH26
Max. CWSEL = 183.0991 m
Rim Elev. = 184 m
12/07/2021 01:50AM

Storage MH29
Max. CWSEL = 182.9978 m
Rim Elev. = 184.12 m
12/07/2021 01:50AM

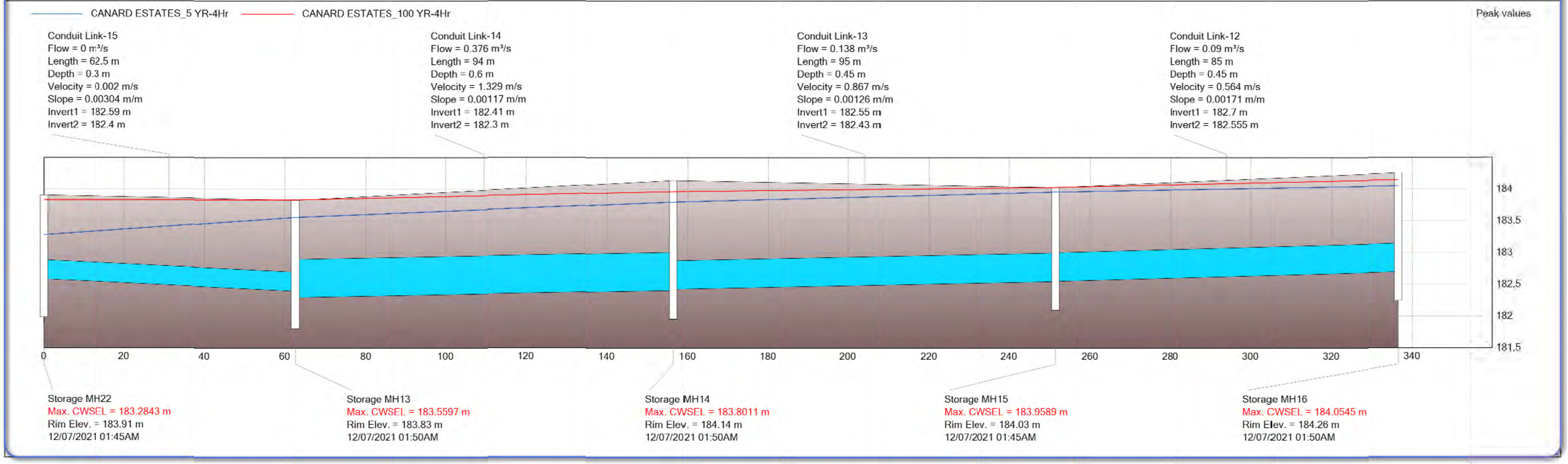
Storage MH28
Max. CWSEL = 182.8908 m
Rim Elev. = 184 m
12/07/2021 01:50AM

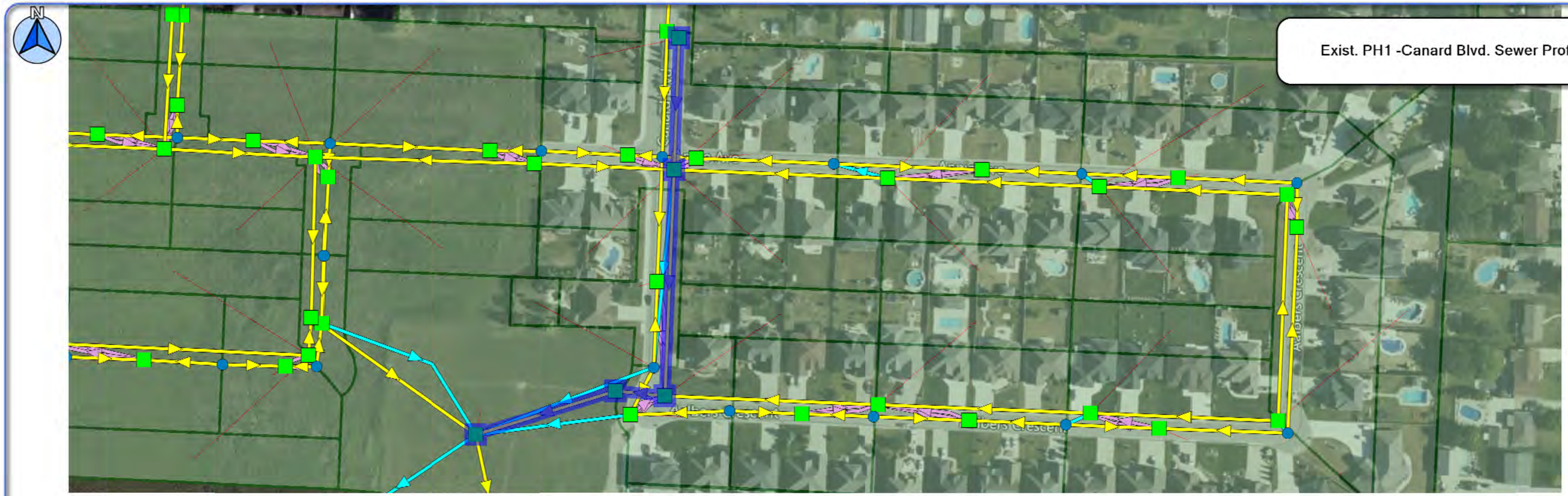
Storage MH27
Max. CWSEL = 182.8349 m
Rim Elev. = 183.91 m
12/07/2021 01:50AM

Storage MH20
Max. CWSEL = 182.7157 m
Rim Elev. = 183.9 m
12/07/2021 04:10AM

Storage POND
Max. CWSEL = 182.7153 m
Rim Elev. = 183.71 m
12/07/2021 04:10AM

Exist. PH1 - Annie Ave. Sewer Profile 1:5Yr. STM





Exist. PH1 -Canard Blvd. Sewer Profile 1:5Yr. STM

CANARD ESTATES_5 YR-4Hr
CANARD ESTATES_100 YR-4Hr

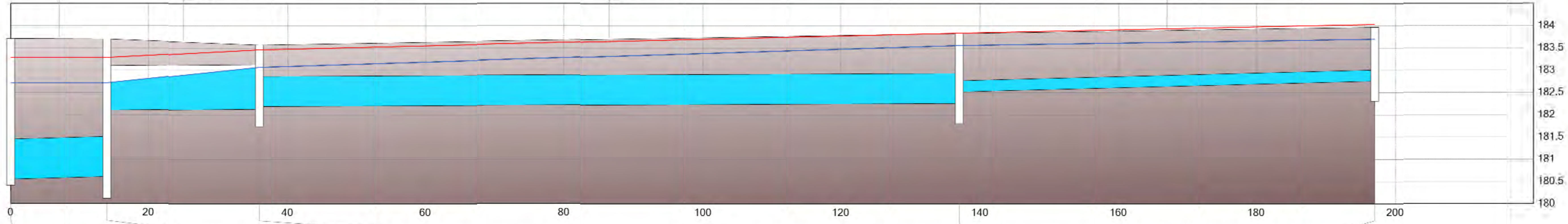
Peak values

Conduit Link-22
Flow = 1.005 m^3/s
Length = 14 m
Depth = 0.9 m
Velocity = 1.579 m/s
Slope = 0.00429 m/m
Invert1 = 180.61 m
Invert2 = 180.55 m

Conduit Link-21
Flow = 1.007 m^3/s
Length = 22 m
Depth = 1 m
Velocity = 1.578 m/s
Slope = 0.00136 m/m
Invert1 = 182.13 m
Invert2 = 182.1 m

Conduit Link-17
Flow = 0.515 m^3/s
Length = 101 m
Depth = 0.675 m
Velocity = 1.44 m/s
Slope = 0.00069 m/m
Invert1 = 182.26 m
Invert2 = 182.19 m

Conduit Link-16
Flow = 0.029 m^3/s
Length = 60 m
Depth = 0.25 m
Velocity = 0.584 m/s
Slope = 0.004 m/m
Invert1 = 182.76 m
Invert2 = 182.52 m



Storage POND
Max. CWSEL = 182.7153 m
Rim Elev. = 183.71 m
12/07/2021 04:10AM

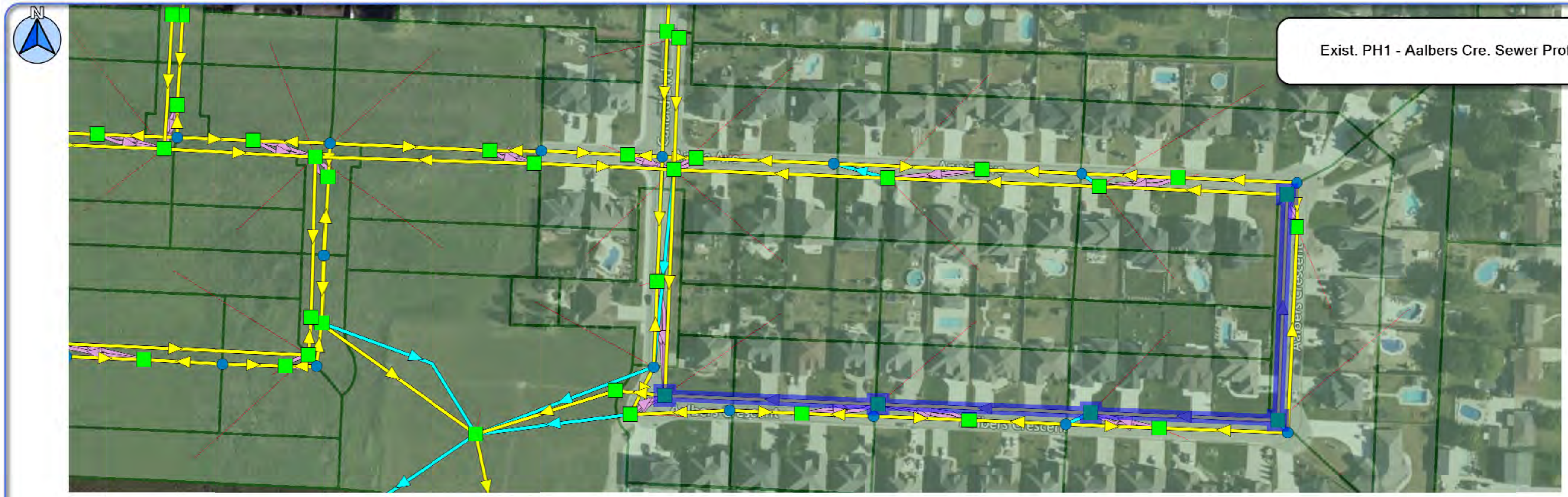
Storage MH20A
Max. CWSEL = 182.7153 m
Rim Elev. = 183.7 m
12/07/2021 04:10AM

Storage MH9
Max. CWSEL = 183.0701 m
Rim Elev. = 183.57 m
12/07/2021 01:50AM

Storage MH13
Max. CWSEL = 183.5597 m
Rim Elev. = 183.83 m
12/07/2021 01:50AM

Storage MH17
Max. CWSEL = 183.7014 m
Rim Elev. = 183.97 m
12/07/2021 01:45AM

Exist. PH1 - Aalbers Cre. Sewer Profile 1:5Yr. STM



CANARD ESTATES_5 YR-4Hr
CANARD ESTATES_100 YR-4Hr

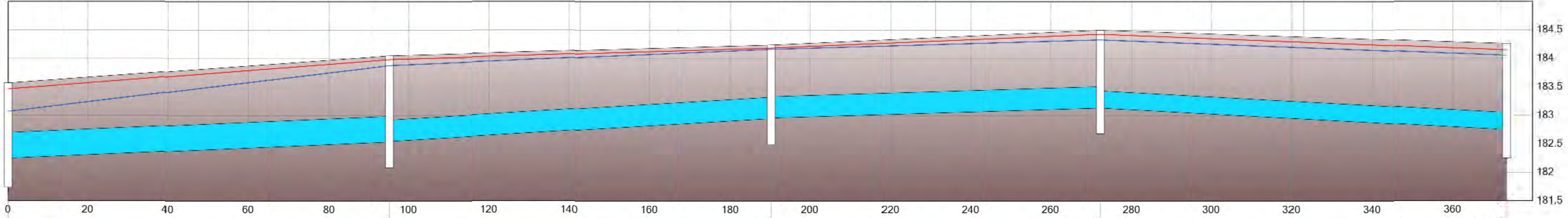
Peak values

Conduit Link-20
Flow = 0.248 m³/s
Length = 95.17 m
Depth = 0.45 m
Velocity = 1.556 m/s
Slope = 0.00305 m/m
Invert1 = 182.54 m
Invert2 = 182.25 m

Conduit Link-19
Flow = 0.092 m³/s
Length = 94.96 m
Depth = 0.375 m
Velocity = 0.833 m/s
Slope = 0.00421 m/m
Invert1 = 182.95 m
Invert2 = 182.55 m

Conduit Link-18
Flow = 0.072 m³/s
Length = 82 m
Depth = 0.375 m
Velocity = 0.655 m/s
Slope = 0.00213 m/m
Invert1 = 183.135 m
Invert2 = 182.96 m

Conduit Link-11
Flow = 0.048 m³/s
Length = 101.4 m
Depth = 0.3 m
Velocity = 0.676 m/s
Slope = 0.0038 m/m
Invert1 = 183.135 m
Invert2 = 182.75 m



Storage MH9
Max. CWSEL = 183.0701 m
Rim Elev. = 183.57 m
12/07/2021 01:50AM

Storage MH10
Max. CWSEL = 183.8788 m
Rim Elev. = 184.05 m
12/07/2021 01:50AM

Storage MH11
Max. CWSEL = 184.1646 m
Rim Elev. = 184.24 m
12/07/2021 01:45AM

Storage MH12
Max. CWSEL = 184.3297 m
Rim Elev. = 184.5 m
12/07/2021 01:45AM

Storage MH16
Max. CWSEL = 184.0545 m
Rim Elev. = 184.26 m
12/07/2021 01:50AM

Outlet Sewer Profile 1:5Yr. STM

Peak values

Conduit 1.0-01
Flow = 0.067 m³/s
Length = 18 m
Depth = 0.9 m
Velocity = 0.345 m/s
Slope = 0 m/m
Invert1 = 181.91 m
Invert2 = 181.91 m

Conduit 1.6-05
Flow = 0.054 m³/s
Length = 70.5 m
Depth = 1.6 m
Velocity = 0.172 m/s
Slope = 0.00034 m/m
Invert1 = 181.91 m
Invert2 = 181.886 m

Conduit 1.6-03
Flow = 0.048 m³/s
Length = 113.44 m
Depth = 1.6 m
Velocity = 0.075 m/s
Slope = 0.00041 m/m
Invert1 = 181.886 m
Invert2 = 181.84 m

Conduit 1.6-02
Flow = 0.049 m³/s
Length = 113 m
Depth = 1.6 m
Velocity = 0.154 m/s
Slope = 0.00035 m/m
Invert1 = 181.84 m
Invert2 = 181.8 m

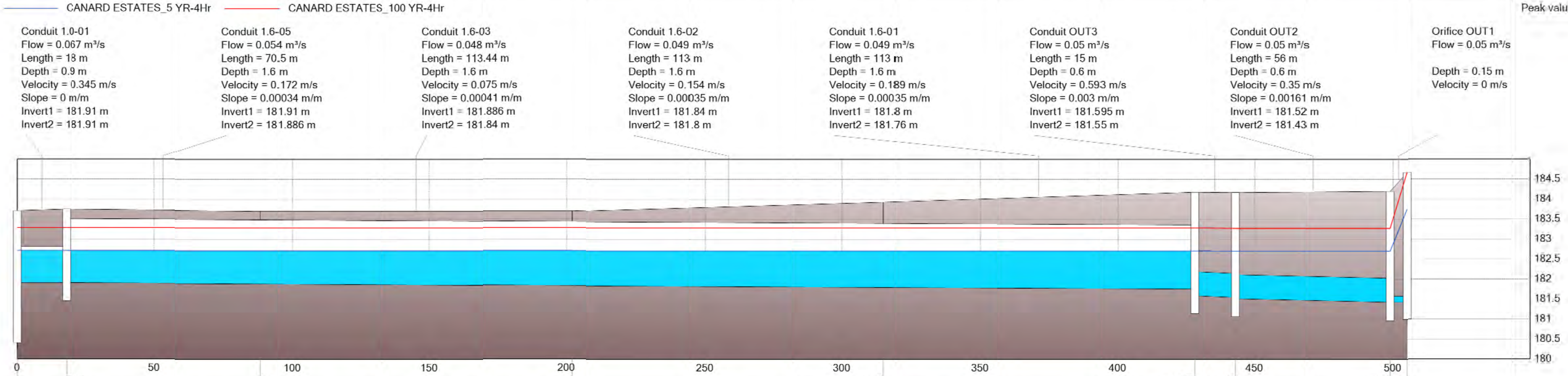
Conduit 1.6-01
Flow = 0.049 m³/s
Length = 113 m
Depth = 1.6 m
Velocity = 0.189 m/s
Slope = 0.00035 m/m
Invert1 = 181.8 m
Invert2 = 181.76 m

Conduit OUT3
Flow = 0.05 m³/s
Length = 15 m
Depth = 0.6 m
Velocity = 0.593 m/s
Slope = 0.003 m/m
Invert1 = 181.595 m
Invert2 = 181.55 m

Conduit OUT2
Flow = 0.05 m³/s
Length = 56 m
Depth = 0.6 m
Velocity = 0.35 m/s
Slope = 0.00161 m/m
Invert1 = 181.52 m
Invert2 = 181.43 m

Orifice OUT1
Flow = 0.05 m³/s

Depth = 0.15 m
Velocity = 0 m/s



Storage POND
Max. CWSEL = 182.7153 m
Rim Elev. = 183.71 m
12/07/2021 04:10AM

Storage MH7
Max. CWSEL = 182.7146 m
Rim Elev. = 183.75 m
12/07/2021 04:10AM

Junction MH6
Max. CWSEL = 182.7142 m
Rim Elev. = 183.7 m
12/07/2021 04:10AM

Junction MH5
Max. CWSEL = 182.7139 m
Rim Elev. = 183.7 m
12/07/2021 04:10AM

Junction MH4
Max. CWSEL = 182.7135 m
Rim Elev. = 183.93 m
12/07/2021 04:10AM

Storage MH3
Max. CWSEL = 182.7131 m
Rim Elev. = 184.18 m
12/07/2021 04:10AM

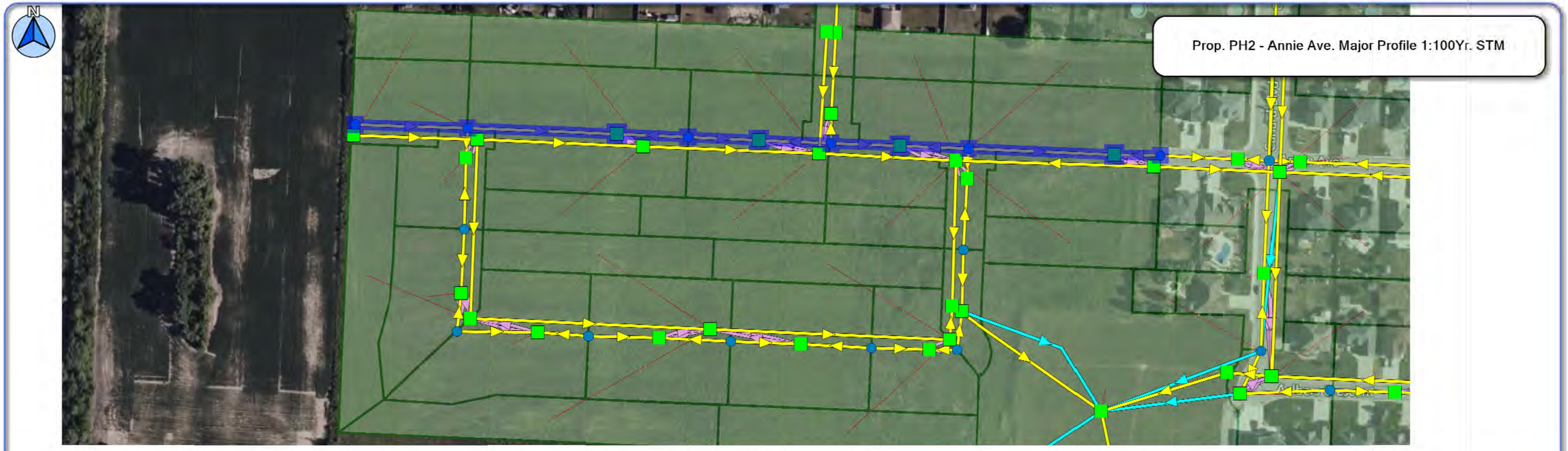
Storage MH2
Max. CWSEL = 182.7102 m
Rim Elev. = 184.17 m
12/07/2021 04:10AM

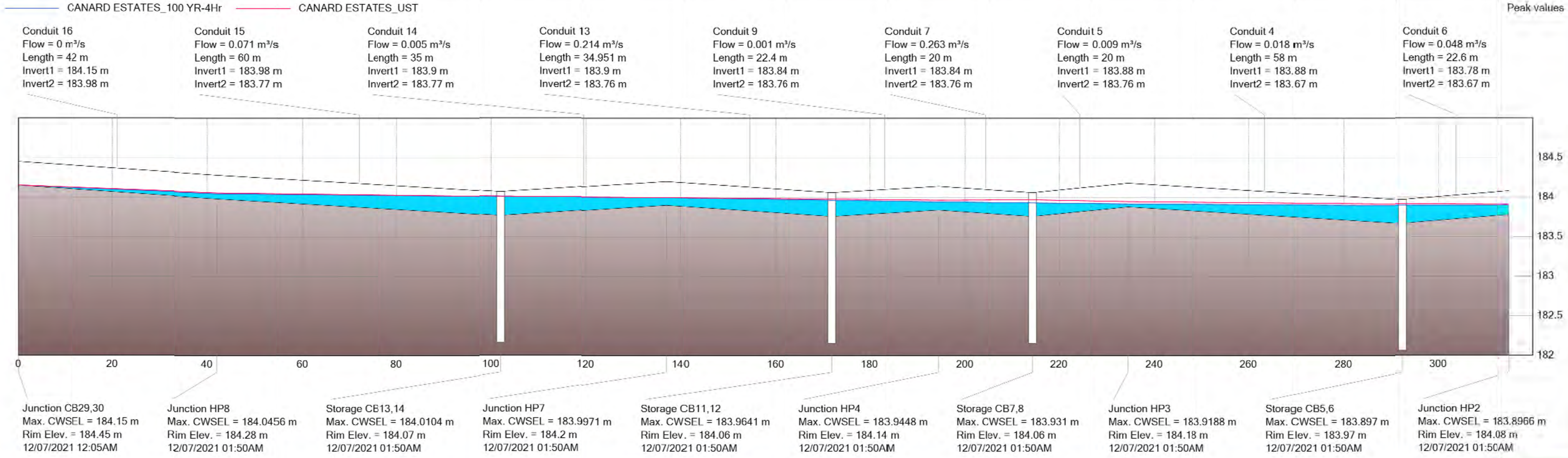
Storage MH1
Max. CWSEL = 182.7054 m
Rim Elev. = 184.2 m
12/07/2021 04:10AM

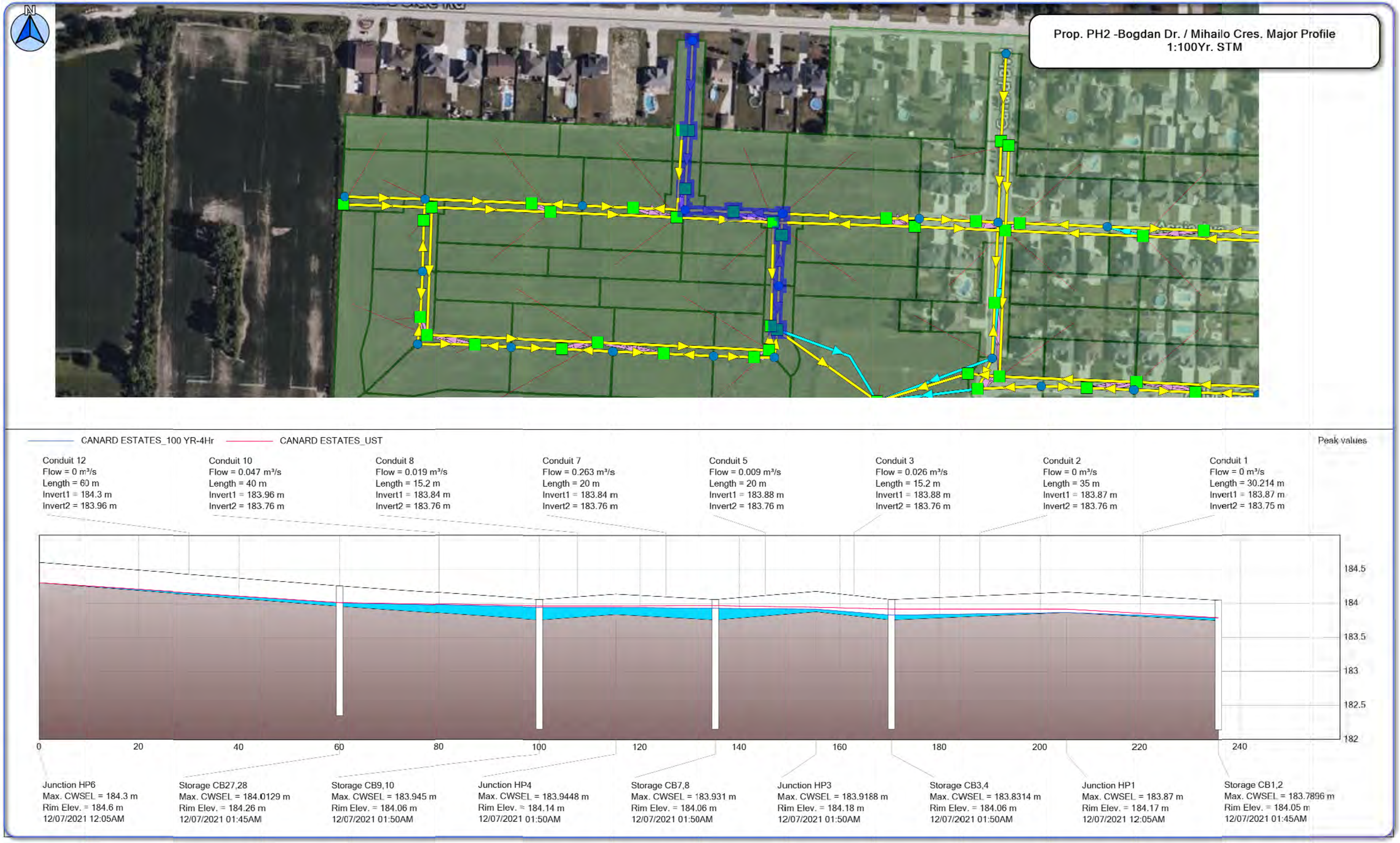
Storage MH1A
Max. CWSEL = 183.7559 m
Rim Elev. = 184.67 m
12/07/2021 02:00AM

B2

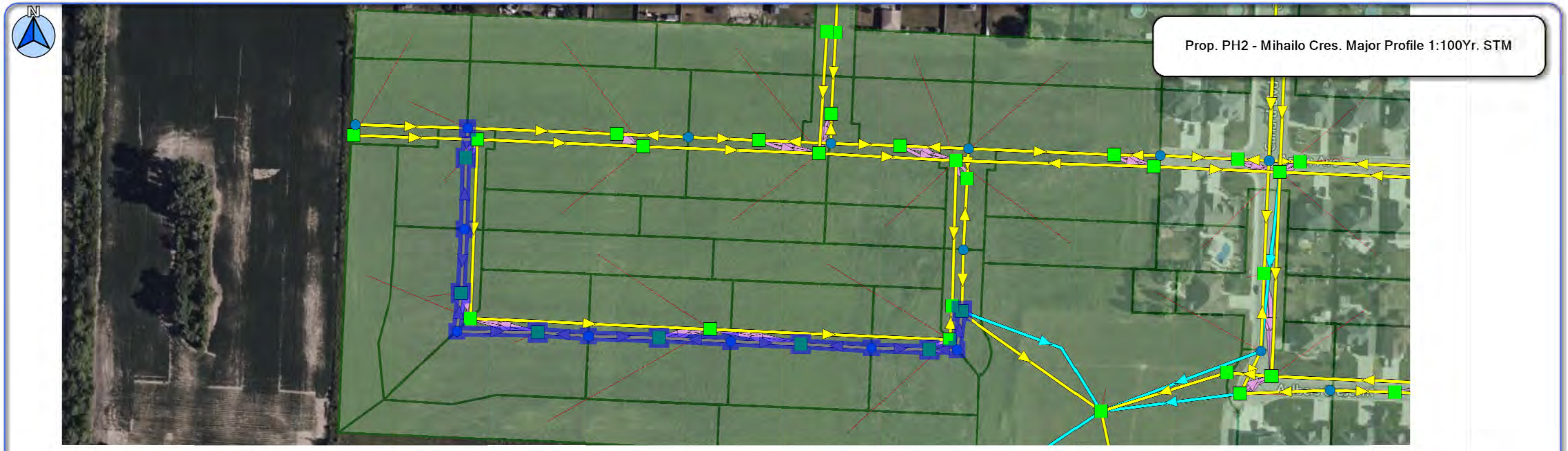
Prop. PH2 - Annie Ave. Major Profile 1:100Yr. STM

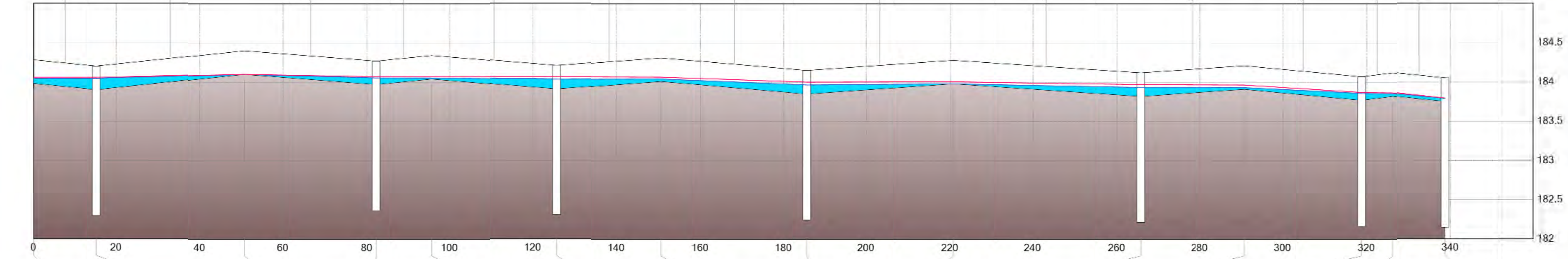





Prop. PH2 -Bogdan Dr. / Mihailo Cres. Major Profile
1:100Yr. STM
N
CANARD ESTATES_100 YR-4Hr
CANARD ESTATES_UST
Peak values
Conduit 12
Flow = 0 m³/s
Length = 60 m
Invert1 = 184.3 m
Invert2 = 183.96 m
Conduit 10
Flow = 0.047 m³/s
Length = 40 m
Invert1 = 183.96 m
Invert2 = 183.76 m
Conduit 8
Flow = 0.019 m³/s
Length = 15.2 m
Invert1 = 183.84 m
Invert2 = 183.76 m
Conduit 7
Flow = 0.263 m³/s
Length = 20 m
Invert1 = 183.84 m
Invert2 = 183.76 m
Conduit 5
Flow = 0.009 m³/s
Length = 20 m
Invert1 = 183.88 m
Invert2 = 183.76 m
Conduit 3
Flow = 0.026 m³/s
Length = 15.2 m
Invert1 = 183.88 m
Invert2 = 183.76 m
Conduit 2
Flow = 0 m³/s
Length = 35 m
Invert1 = 183.87 m
Invert2 = 183.76 m
Conduit 1
Flow = 0 m³/s
Length = 30.214 m
Invert1 = 183.87 m
Invert2 = 183.75 m
184.5
184
183.5
183
182.5
182
0
20
40
60
80
100
120
140
160
180
200
220
240
Junction HP6
Max. CWSEL = 184.3 m
Rim Elev. = 184.6 m
12/07/2021 12:05AM
Storage CB27,28
Max. CWSEL = 184.0129 m
Rim Elev. = 184.26 m
12/07/2021 01:45AM
Storage CB9,10
Max. CWSEL = 183.945 m
Rim Elev. = 184.06 m
12/07/2021 01:50AM
Junction HP4
Max. CWSEL = 183.9448 m
Rim Elev. = 184.14 m
12/07/2021 01:50AM
Storage CB7,8
Max. CWSEL = 183.931 m
Rim Elev. = 184.06 m
12/07/2021 01:50AM
Junction HP3
Max. CWSEL = 183.9188 m
Rim Elev. = 184.18 m
12/07/2021 01:50AM
Storage CB3,4
Max. CWSEL = 183.8314 m
Rim Elev. = 184.06 m
12/07/2021 01:50AM
Junction HP1
Max. CWSEL = 183.87 m
Rim Elev. = 184.17 m
12/07/2021 12:05AM
Storage CB1,2
Max. CWSEL = 183.7896 m
Rim Elev. = 184.05 m
12/07/2021 01:45AM

Prop. PH2 - Mihailo Cres. Major Profile 1:100Yr. STM





CANARD ESTATES_100 YR-4Hr — CANARD ESTATES_UST

Peak values

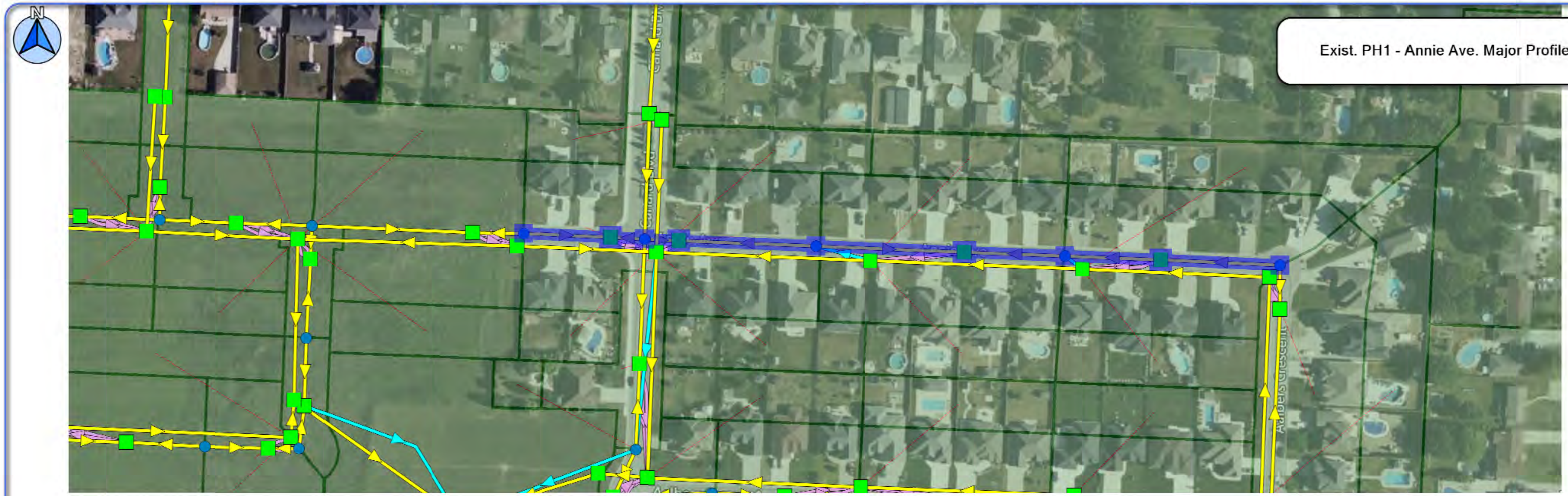
Links:

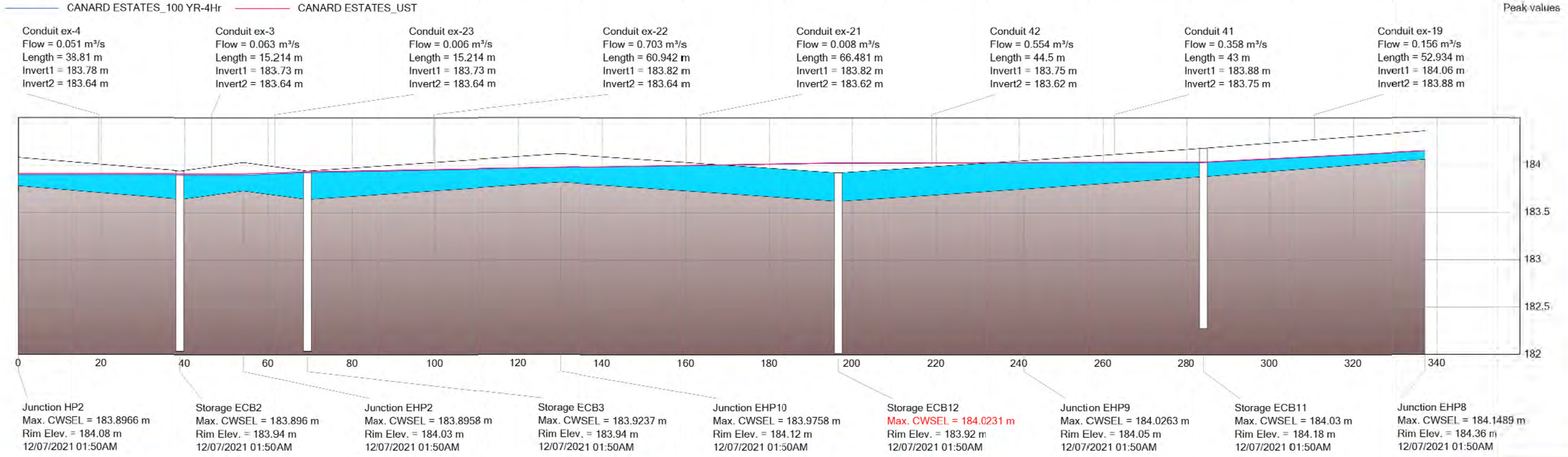
17	Q=0.042 m³/s	L=15.2 m	I1=183.98 m	I2=183.9 m
18	Q=0 m³/s	L=35.5 m	I1=184.1 m	I2=183.9 m
19	Q=0 m³/s	L=31.5 m	I1=184.1 m	I2=183.97 m
20	Q=0.001 m³/s	L=13.5 m	I1=184.04 m	I2=183.97 m
21	Q=0.002 m³/s	L=30 m	I1=184.04 m	I2=183.91 m
22	Q=0.007 m³/s	L=25 m	I1=184.01 m	I2=183.91 m
23	Q=0.012 m³/s	L=35 m	I1=184.01 m	I2=183.85 m
24	Q=0 m³/s	L=35 m	I1=183.98 m	I2=183.85 m
25	Q=0 m³/s	L=45 m	I1=183.98 m	I2=183.82 m
26	Q=0.001 m³/s	L=25 m	I1=183.91 m	I2=183.82 m
27	Q=0.005 m³/s	L=28 m	I1=183.91 m	I2=183.77 m
29	Q=0 m³/s	L=7.5 m	I1=183.82 m	I2=183.77 m
28	Q=0.016 m³/s	L=12.5 m	I1=183.82 m	I2=183.75 m

Nodes:

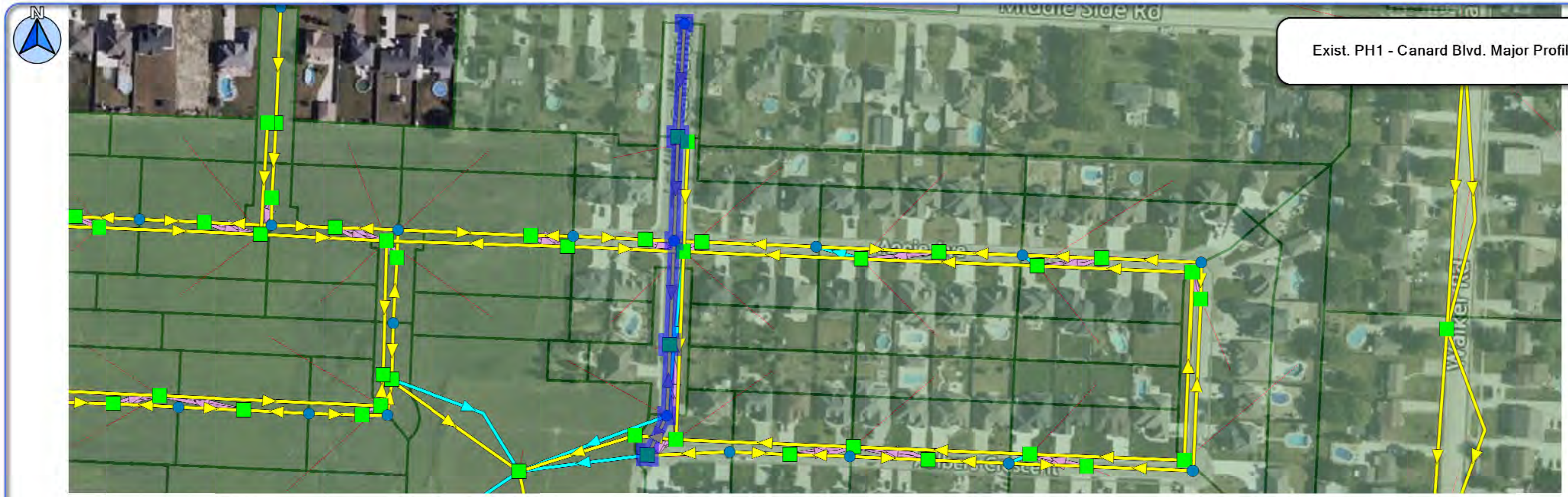
HP8	M=184.0456 m	R=184.28 m
CB25,26	M=184.0449 m	R=184.2 m
HP10	M=184.1 m	R=184.4 m
CB23,24	M=184.0555 m	R=184.27 m
HP11	M=184.0551 m	R=184.34 m
CB21,22	M=184.0375 m	R=184.21 m
HP12	M=184.0416 m	R=184.31 m
CB19,20	M=183.9662 m	R=184.15 m
HP13	M=183.98 m	R=184.28 m
CB17,18	M=183.9339 m	R=184.12 m
HP14	M=183.9328 m	R=184.21 m
CB15,16	M=183.8587 m	R=184.07 m
HP15	M=183.8542 m	R=184.12 m
CB1,2	M=183.7896 m	R=184.05 m

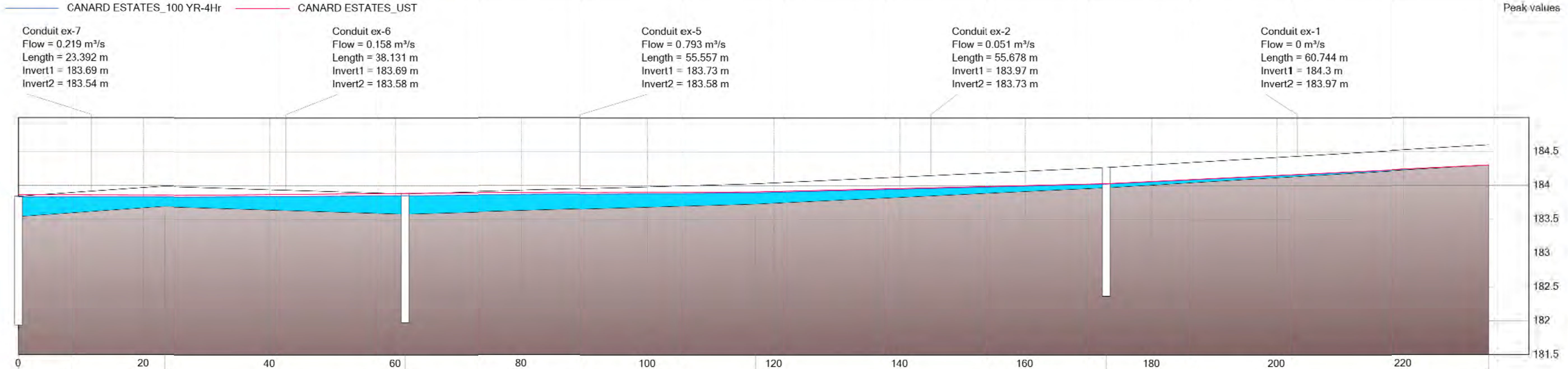
Exist. PH1 - Annie Ave. Major Profile 1:100Yr. STM





Exist. PH1 - Canard Blvd. Major Profile 1:100Yr. STM





Conduit ex-7
Flow = 0.219 m³/s
Length = 23.392 m
Invert1 = 183.69 m
Invert2 = 183.54 m

Conduit ex-6
Flow = 0.158 m³/s
Length = 38.131 m
Invert1 = 183.69 m
Invert2 = 183.58 m

Conduit ex-5
Flow = 0.793 m³/s
Length = 55.557 m
Invert1 = 183.73 m
Invert2 = 183.58 m

Conduit ex-2
Flow = 0.051 m³/s
Length = 55.678 m
Invert1 = 183.97 m
Invert2 = 183.73 m

Conduit ex-1
Flow = 0 m³/s
Length = 60.744 m
Invert1 = 184.3 m
Invert2 = 183.97 m

Storage ECB6
Max. CWSEL = 183.8334 m
Rim Elev. = 183.84 m
12/07/2021 01:50AM

Junction EHP3
Max. CWSEL = 183.8322 m
Rim Elev. = 183.99 m
12/07/2021 01:50AM

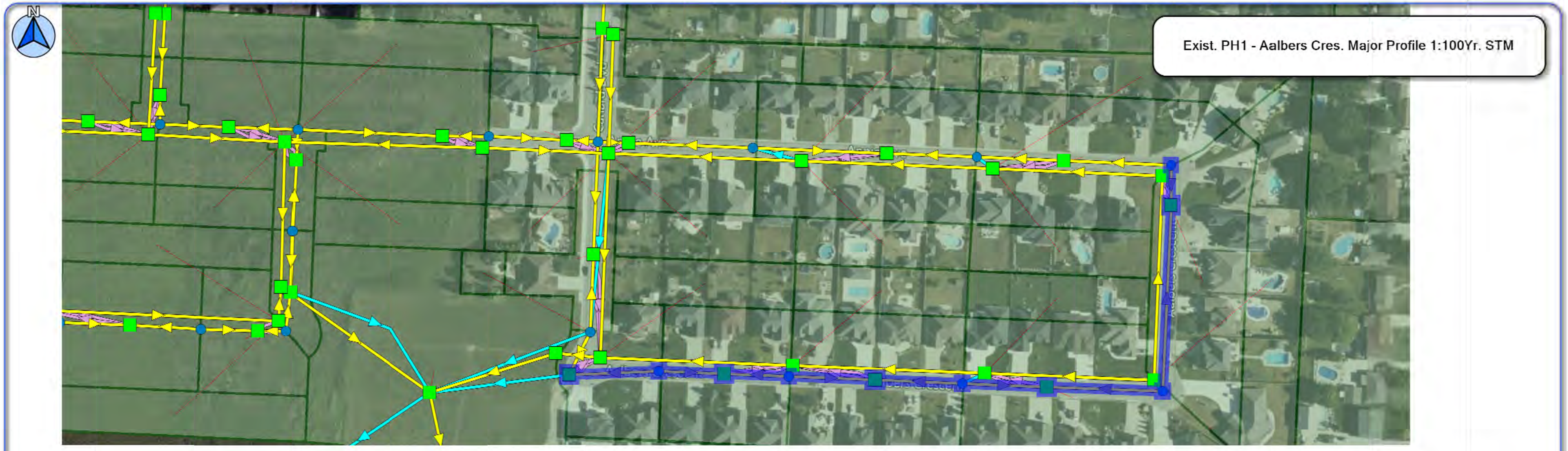
Storage ECB4
Max. CWSEL = 183.8527 m
Rim Elev. = 183.88 m
12/07/2021 01:50AM

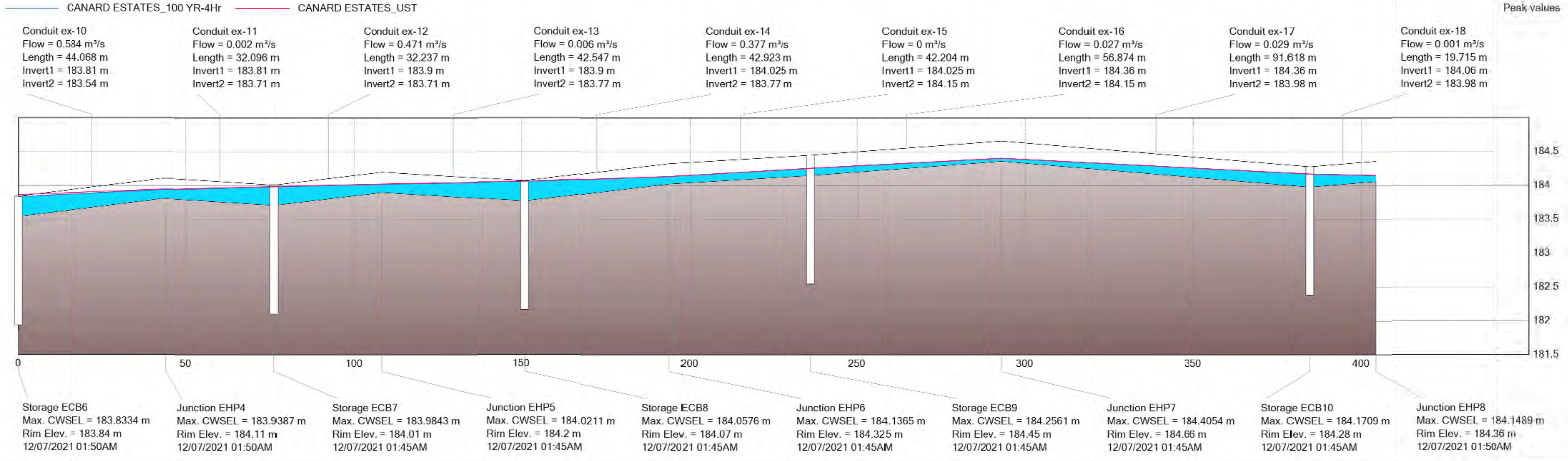
Junction EHP2
Max. CWSEL = 183.8958 m
Rim Elev. = 184.03 m
12/07/2021 01:50AM

Storage ECB1
Max. CWSEL = 184.0258 m
Rim Elev. = 184.27 m
12/07/2021 01:45AM

Junction EHP1
Max. CWSEL = 184.3 m
Rim Elev. = 184.6 m
12/07/2021 12:05AM

Exist. PH1 - Aalbers Cres. Major Profile 1:100Yr. STM





Conduit ex-10
Flow = 0.584 m³/s
Length = 44.068 m
Invert1 = 183.81 m
Invert2 = 183.54 m

Conduit ex-11
Flow = 0.002 m³/s
Length = 32.096 m
Invert1 = 183.81 m
Invert2 = 183.71 m

Conduit ex-12
Flow = 0.471 m³/s
Length = 32.237 m
Invert1 = 183.9 m
Invert2 = 183.71 m

Conduit ex-13
Flow = 0.006 m³/s
Length = 42.547 m
Invert1 = 183.9 m
Invert2 = 183.77 m

Conduit ex-14
Flow = 0.377 m³/s
Length = 42.923 m
Invert1 = 184.025 m
Invert2 = 183.77 m

Conduit ex-15
Flow = 0 m³/s
Length = 42.204 m
Invert1 = 184.025 m
Invert2 = 184.15 m

Conduit ex-16
Flow = 0.027 m³/s
Length = 56.874 m
Invert1 = 184.36 m
Invert2 = 184.15 m

Conduit ex-17
Flow = 0.029 m³/s
Length = 91.618 m
Invert1 = 184.36 m
Invert2 = 183.98 m

Conduit ex-18
Flow = 0.001 m³/s
Length = 19.715 m
Invert1 = 184.06 m
Invert2 = 183.98 m

Storage ECB6
Max. CWSEL = 183.8334 m
Rim Elev. = 183.84 m
12/07/2021 01:50AM

Junction EHP4
Max. CWSEL = 183.9387 m
Rim Elev. = 184.11 m
12/07/2021 01:50AM

Storage ECB7
Max. CWSEL = 183.9843 m
Rim Elev. = 184.01 m
12/07/2021 01:45AM

Junction EHP5
Max. CWSEL = 184.0211 m
Rim Elev. = 184.2 m
12/07/2021 01:45AM

Storage ECB8
Max. CWSEL = 184.0576 m
Rim Elev. = 184.07 m
12/07/2021 01:45AM

Junction EHP6
Max. CWSEL = 184.1365 m
Rim Elev. = 184.325 m
12/07/2021 01:45AM

Storage ECB9
Max. CWSEL = 184.2561 m
Rim Elev. = 184.45 m
12/07/2021 01:45AM

Junction EHP7
Max. CWSEL = 184.4054 m
Rim Elev. = 184.66 m
12/07/2021 01:45AM

Storage ECB10
Max. CWSEL = 184.1709 m
Rim Elev. = 184.28 m
12/07/2021 01:45AM

Junction EHP8
Max. CWSEL = 184.1489 m
Rim Elev. = 184.36 m
12/07/2021 01:50AM