Servicing and Stormwater Management Report
Garden Hill Estates

Mistral Land Development

April 14, 2022

MONUMENT GEOMATICS

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1 Background

Monument Geomatics and Estimating ("MG" or "Monument") was retained by Mistral Land Development to prepare a preliminary draft plan submission for a rural development in Garden Hill, Ontario, located in the Municipality of Port Hope. The development will consist of two rural roadways to allow frontage for 43 single family homes sitting on 3/4 acre lots, and a single residential block with frontage on Ganaraska Road and Frost Ave.

The development lands (hereto referred to as – "Mistral Lands" or "Property") is located northeast of Ganaraska Road and Mill Street intersection just east of Ganaraska Conservation Area and Garden Hill Dam (see Figure 1-1). The Municipal address for the property is located at 3852 Ganaraska Road, Garden Hill. The parcel of land is 36.6ha in area and zoned as future development. The south half of the property is comprised of various cropped fields and small pasture area, whereas the north portion is heavily forested with a high voltage hydro easement bisecting through the property east to west.

A geotechnical investigation was carried out by Terraspec Engineering Inc. on April 27, 2021. The soil layers encountered on site were silty topsoil, silty sand, and clay silt with groundwater encountered 1.5 - 3m below the ground surface. Bedrock was not encountered at the time of investigation. Terraspec's final report was published in May 2021 and was prepared under a separate cover. Monument prepared an excerpt from the report showing the borehole and water surface elevations in Appendix A.

Greer Galloway completed a Phase 1 Environmental Site Assessment (ESA) on May 31, 2020. The purpose of the study was to review the property's history and identify any environmental concerns or infractions that may have released significant contamination into the natural environment of the site. The study identified two potential contaminating activities (PCAs) in relation to the existing residence and farmstead fronting along Ganaraska Roadway. However, these PCAs were deemed at low risk for significant contamination and a Phase 2 ESA was not recommended. Further information into this investigation is provided under a separate cover.

A natural heritage investigation was conducted by Cambium Inc. in the summer of 2021 to determine any natural constraints for the development. Cambium's constraint map is provided in Appendix B. One major constraint indicated is the forested lands at the north half of the property. The land falls within the significant woodland protection criteria. Through stakeholder consultation the site plan was developed to reduce the building footprint within the forested areas to reduce the removal and negative impacts to the woodland. The significant woodland north of the hydro corridor falls outside the Garden Hill Hamlet and is not deemed suitable for development.

There are two significant drainage features located on the property. To the south, is a small tributary of the North Ganaraska River System that ravines through the southeast corner of the property. To the north, towering banks overlook a small stream within the forested lands draining southwest across the hydro easement and outlets onto the Ganaraska Conservation Area reservoir ("Reservoir") upstream of the Garden Hill Dam. For simplicity, the reach to the north will be referred to as the North Tributary and as such the south reach will be referred to as South Tributary. Ganaraska Region Conservation Authority (GRCA) met with Cambium on-site and it was identified that the South Reach was a warm-water system where the North Tributary is a cold-water system. The figure on the next page provides an overview of the subject property.

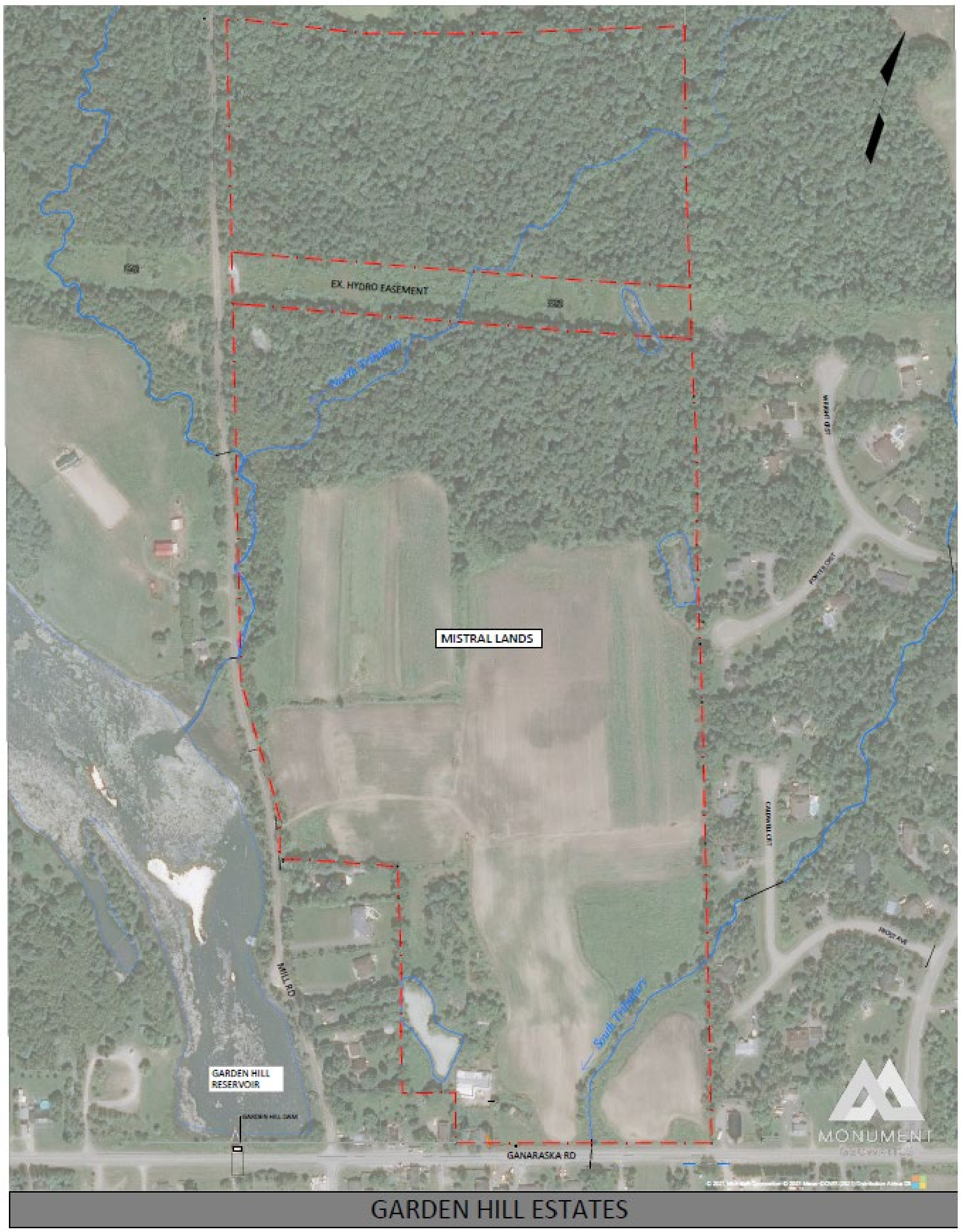

Figure 1: Site Overview

2 Proposed Development

The development will involve the construction of two rural roadways to allow frontage of 43 single family homes subdivided onto one 3/4 acre lots, and a single residential block with frontage on Ganaraska Road and Frost Ave. It is expected that each single-family lot will be developed with an estimated 2000 square-foot dwelling with a 2-bay garage, accessed with a 6m wide paved driveway and serviced with a private well and septic system. For runoff purposes, each lot will be assumed to have an additional 400sqft outbuilding (i.e. shed) to offset the variance of impervious area on each lot at full buildout. The single residential lot (Block 104) will be developed with an apartment/condo building. Site Plan Control will be required for the development of Block 104.

A draft plan of the development is provided in Appendix C illustrating the proposed lot fabric. On this plan it illustrates the regulated lands that are confined with watercourses and applicable setbacks. There are two stormwater management (SWM) facilities illustrated at the northwest and south portions of the site. As previously mentioned, the limits of the development will not extend past the hydro easement.

The development is proposed to be accessed by way of two new connections off Ganaraska Road and Porter Crescent. Safe access to the site, meeting Provincial Policy Statement 3.1.2c, will be provided from Porter Crescent. The second access for the site is provided from Ganaraska Road. The Site Access Letter prepared by Monument provides additional details to support the site's access locations.

Monument prepared a conceptual servicing plan (see Appendix D) to show a conceptual view on how each lot will meet the minimum servicing setbacks discussed in Section 3. The site will be fine graded to improve drainage and ensure runoff is contained within the development lands and conveyed to the required outlet points. Overall, runoff from the site will be conveyed using grassed swales, roadside ditches, and corrugated steel culverts.

Lot drainage will be directed where necessary using rear and side-yard swales to control overland flow along property boundaries. Proper lot grading techniques such as rear-to-front, front-to-rear, and split drainage will be implemented on a lot-per-lot basis around buildings to divert water away from foundations.

3 Water Supply and Sewage Disposal

The proposed residential lots will be serviced by individual drilled wells. Details of the groundwater supply will be provided in a Hydrogeological Investigation prepared by Greer Galloway Group under a separate study. The proposed stormwater management ponds are considered potential sources of contaminants, thus wells on lots adjacent to these ponds will have a minimum separation distance of 15m.

Sanitary servicing for the proposed development would be provided by individual on-site septic systems. As specified by the Ontario Building Code (OBC) Section 8 and the Ministry of Environment Water Supply Wells: Requirements and Best Practices. The proposed wells and septic systems will have a minimum separation distance of 15m. In addition, as specified by the OBC the septic systems will be at least 15m from the proposed stormwater management ponds, 5m from the proposed residential dwellings, and 3m from any property lines.

Fire suppression operations for the development will take advantage of the existing dry fire hydrant installed along the banks of the Garden Hill Conservation Reservoir on Mill Street. The location of this dry fire hydrant is illustrated on the Conceptual Servicing Plan in Appendix D.

4 SWM Design Objectives

The SWM design was prepared to meet the following objectives:

1) **Quantity Control** - The objective is to ensure that post-development peak flows meet the pre-development levels for all minor and major storm events up to the 100-yr return period. The Ganaraska Conservation Authority Region falls within Zone 1 of the Flood Hazard Criteria Zones for Ontario. Therefore, the Hurricane Hazel event will also be assessed to ensure the greatest peak flows are safely conveyed through the development in major overland flow routes.

2) **Quality Control** – Quality control will be provided using Best Management Practices (BMP's) to meet a level 1 total suspended solid (TSS) removal efficiency as an enhanced level of protection (80% long-term suspended solids removal), in accordance with the MECP's Table 3.2 in the 2003 SWM guideline.

3) **Sediment and Erosion Measure** - Prepare a sediment and erosion plan to control and mitigate release of sediment throughout the construction stage.

5 Hydrology

Monument used PCSWMM version 7.4.1 software to model the pre- and post-development conditions on site and delineate flood boundaries of the small stream located at the southeast corner of the property.

5.1 PCSWMM

PCSWMM version 7.4.1 is a powerful modeling platform companied with EPANET2 and EPA SWMM5 software that provides various hydrologic and hydraulic modeling capabilities. Within one platform, the user can analyze several SWM components such as stormwater, wastewater, water distribution system and watershed modeling.

Catchment areas are best represented based on the user defined hydrologic parameters carefully selected for each basin. Monument has selected to use the process modeling of rainfall/runoff combined with the flow routing option to assess the capacity of conduits (I.e. culverts & swales). Monument also selected to use the curve number option for the infiltration calculations. Based on the selected models the following parameters are required as inputs for each catchment:

- Area (ha) – Total area of each catchment
- Width (m) – Width of overland flow path; this is automatically determined based on the flow length
- Length (m) – Longest flow path of overland sheet flow
- Slope (%) – Average surface slope; MG determines the average slope using the 85/10 Method
- Imperv. (%) – Percent of Impervious area
- N imperv. – Manning's N for impervious area
- N Perv. – Manning's N for pervious area
- Dstore Imperv (mm) – Depth of depression storage on impervious area;
    - Selected 1mm
- Dstore Perv (mm) – Depth of depression storage on pervious area;
    - Selected 2.5mm for Agricultural lands and 5mm for grassed lands.
- Zero Imperv (%) – Percent of impervious area with no depression storage;
    - Selected 25%
- Curve Number – SCS runoff curve number

5.2 Precipitation Data

GRCA's 2014 Technical and Engineering Guidelines recommend that design storms be applied for the SCS type II, Chicago and AES distribution to determine the greatest runoff peaks using the 6-, 12-, and 24-hr durations. Precipitation data was selected using the Bowmanville Mostert rain gauge with over 32 years of collected precipitation data. The IDF curve and rainfall depths are provided in Appendix E.

The Ganaraska Conservation Watershed falls under Zone 1 of the *Flood Hazard Criteria Zones of Ontario and Conservation Authorities Map* in the Ministry of Natural Resources (MNR) 2002 *Technical Guide River & Stream Systems: Flood Hazard Limit.* For Zone 1, the Regulatory (“Regional”) storm will follow the greatest of either the 100-yr storm or Hurricane Hazel.

Hurricane Hazel

Hurricane Hazel occurred in the southwestern region of Ontario in October of 1954. The 48hr storm produced a total precipitation depth of 285mm. The Ministry of Transportation’s (MTO) 2008 Drainage Management Manual Design Chart 1.03 (see Appendix D) provides the synthetic storm data for this event. As illustrated in the table below, the first 36hrs produced only 25% (73mm) of the total rainfall, with the remaining 212mm occurring in the last twelve hours. Hurricane Hazel is typically modelled by applying the last twelve hours only and adjusting the antecedent soil moisture conditions (AMC) from AMCII (normal) to AMC III (saturated), to account for the previous 36hrs of precipitation and saturation of the ground voids.

5.3 Soils Conditions

Figure 2 illustrates the Ministry of Agriculture, Food and Rural Affairs AG Ontario Soil Survey Complex of the hydrologic soil groups (HSG) found within the site boundaries. The site’s soil composition consists of a mix of HSG A Ponty Pool Sandyloam and HSG B Bondhand Loam. The soil composition determined in the Terraspec April 27, 2021 field investigation found silty topsoil overlying silty sand and clay silt layers. The 12 boreholes averaged a topsoil thickness of 200mm and found groundwater depths between 1.5 to 3m below the ground surface.

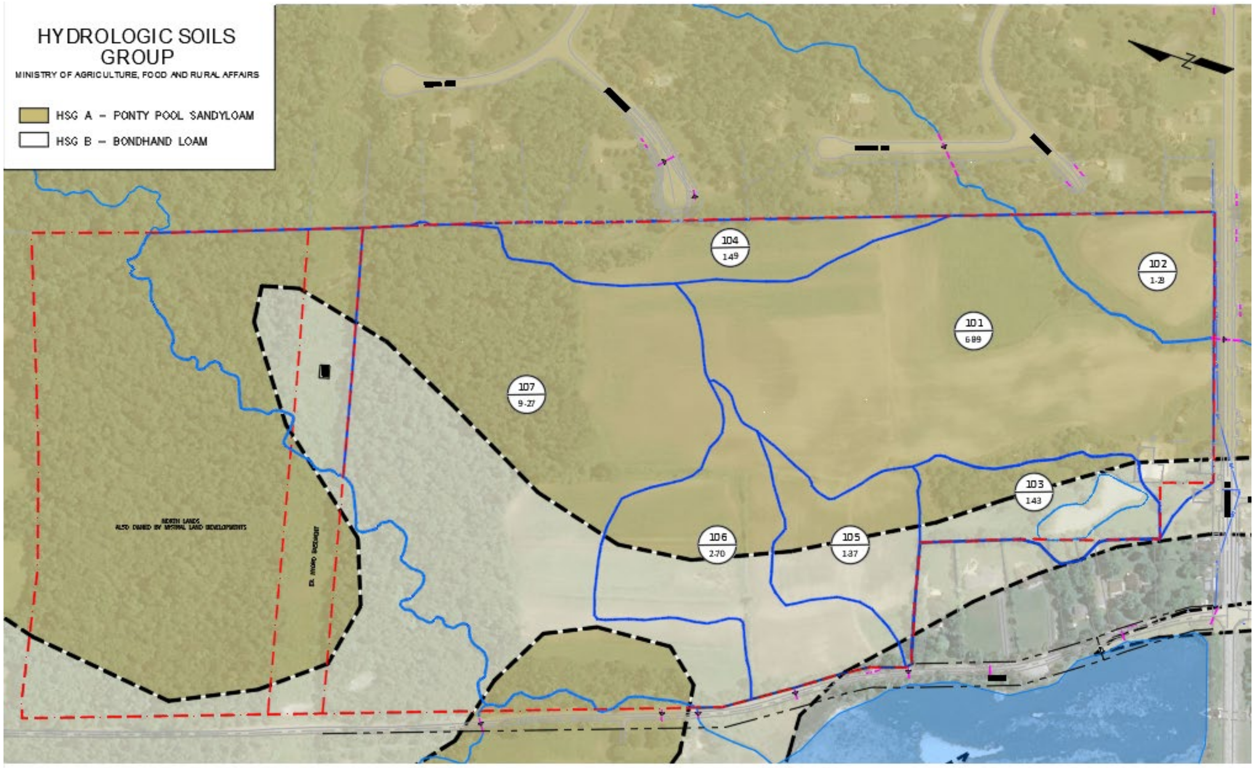

Figure 2: Overview of Hydrologic Soils Group

5.4 Curve Number

Monument selected to use the curve number model in PCSWMM to account for the infiltration potential of each catchment. The curve number is heavily based on the hydrologic soils group and the land use type. A weighted curve number was determined for each catchment using the equation shown below. The proposed curve numbers were used from Attachment 1 of the GRCA's Technical SWM design guidelines.

Equation 1: Weighted Curve Number Equation

(Eqn. 1)

$$*CN = \frac{A_1CN_1 + A_2CN_2 + \cdots}{A_t}$$

where:
**CN = composite curve number*
$A_{1,2,}$= area corresponding to specific land use or soil type, ha
$CN_{1,2,}$= curve number corresponding to $A_{1,2,\ldots}$
A_t = total drainage area, ha

6 Existing Conditions

6.1 Onsite Drainage

A schematic of the existing drainage pattern is provided on the Pre-development Catchment Area drawing shown in Appendix F. These catchments were delineated based on the existing site drainage patterns using the LIDAR derived contours from a combination of Monument survey, and contour data available in Ontario's Geohub database. Overall, the site drains to four separate outfalls along the south and west border of the property.

Outfall #1

Outfall #1 is a 900mm circular corrugated steel pipe (CSP) culvert at Ganaraska Road where a small creek is conveyed to the North Ganaraska River 254m downstream. The pre-development catchment areas contributing to this outfall are 101, 102 & 103. Catchment Area 104 also drains to this outfall which contains the small, isolated wetland pocket at the northeast boundary. This wetland receives runoff from the external lands 104EXT and then spills south and leaves the site into the roadside ditch of Caldwell Court. The objective for the external lands to the north is to receive this flow in the post-development conditions routed to OF#1 to maintain the existing drainage pattern.

Outfall #2

Outfall #2 is the furthest outfall to the north. This outfall point is located at a 1700mm CSP culvert crossing along Mill Street where two northern tributaries outlet into the Ganaraska Conservation Authority Area Reservoir. The contributing catchment areas onsite are 107B and 107 which both sheet flow to the North Tributary. A small external area is received in Catchment 107B within the hydro easement. Since development is not proposed past the hydro easement, Monument did not model these catchment areas in either the pre- or post-development conditions.

Outfall #3

Outfall #3 is located at the 500mm CSP culvert crossing south of Outfall #2 at Mill Street. Catchment 106 is the only contributing catchment area to this outfall as it is captured in the adjacent roadside ditches.

Outfall #4

Outfall #4 is a small 400 mm CSP culvert crossing Mill Street located south of Outfall #3. Monument completed an onsite inspection of this existing culvert and found it to be undermined with runoff not being conveyed through the culvert however underneath the structure through the roadbed. Catchment 105 is the only catchment area draining to this culvert.

6.2 Erosion Hazard Limit

River and stream systems are susceptible to erosion hazards due to flooding, erosion, and slope stability which may pose a threat to life and property. As per the Ministry of Natural Resources and Forestry (MNRF) 2002, Technical Guideline *River and Stream Systems: Flooding Hazard Limits*. Erosion hazard limits can be defined based on a combination of the following factors depending on the stream system:

Toe erosion allowance - applied to confined systems, determined by measuring 15m inland horizontally and perpendicular to the toe of the watercourse slope.

Stable slope allowance - applied to confined systems for locating the top of the stable slope and is defined as the top of a 3:1 slope measured from the more landward of either the toe of the valley or 15m landward from the river or stream bank.

Erosion access allowance – applied to confined and unconfined systems for providing access to an erosion prone area and defined as 6m landward from the top of the stable slope.

Flooding hazard limit – applied to unconfined systems to determine the extent of flooding in flat terrains where natural hazards may extend beyond the immediate channel.

The erosion hazard limit for the North Tributary was determined by adding the following:

- 15m landward from the tributary to apply the toe erosion allowance.
- The top of a 3:1 slope measured from the toe erosion allowance to apply the stable slope allowance.
- 6m from the top of the stable slope to apply the erosion access allowance.

To be conservative, an erosion hazard limit of 43m was applied to the entire length of the tributary as this was the greatest sum of the three allowances in a given reach. See Appendix G for erosion hazard limit drawing and cross-sections.

The erosion hazard limit for the South Tributary was determined by adding the following:

- The flooding hazard limit for Hurricane Hazel as outlined in the Ministry of Natural Resources Technical Guide – River and Stream Systems, Flooding Hazard Limit. See section below.
- 6m from the flooding hazard limit to apply the erosion access limit.

7 Floodplain Analysis

The purpose of this Section is to determine the 100-yr and Regional flood elevations and delineate the flood limit inundating the adjacent banks of the developable lands. The study will incorporate anticipated future impervious area from the development and follow the guidelines of the Ministry of Natural Resources and Forestry (MNRF) 2002 Technical Guideline *River and Stream Systems: Flooding Hazard Limits.*

7.1 North Tributary

The North Tributary creek outlets into a larger tributary just upstream of Garden Hill Conservation Area Reservoir ("GHCA Reservoir") east of Mill Street. Downstream, the two tributaries are conveyed through Outfall #2. The North Tributary has an approximate contributing area of 88.71ha, a total channel length of 886m and general longitudinal slope of 2.4%. Within the subject property, the creek banks are generally steep with elevation differences from the channel to top of bank of up to 5m in some locations. As described in the previous Section, the North Tributary is considered a confined river system due to its location within the valley corridor.

GRCA provided Fill and Flood Plain Maps of the North Ganaraska River that was completed by Totten Sims Hubicki Associates in January 1990. The maps illustrate the 100-yr and Regional flood limits south of Ganaraska Road and upstream of the GHCA Reservoir dam. The mapping revealed flooding occurring halfway in the reach length of the North Tributary within the property limits.

The 100-yr and Regional flood elevations were extracted from the maps just north of the Mill Street crossing and were determined to be 173.79m and 176.41m, respectively (River Station 6.0705 – see Appendix H).

Monument delineated the reach of this flood event as shown on the Post-development Drainage Plan in in Appendix D. Given the steep banks of the confined river valley and the setback from which the development is proposed, a floodplain analysis was not completed for this reach. Monument also assumes that the minor increase in impervious area from the development will peak at a time less than that peaked for the entire catchment area of the North Ganaraska River resolving in a very insignificant impact to the flood elevations. Therefore, no further floodplain analysis was completed.

7.2 South Tributary

The small tributary at the southeast corner extends north into the Woodland Gardens development intersecting Caldwell Court and Wright Crescent. South of Ganaraska road, it directly outlets into the North Ganaraska River approximately 254m downstream. Monument prepared a Catchment Area drawing provided in Appendix G delineating 4 sub-catchments to the tributary's outlet point.

7.2.1 Existing Floodline

A PCSWMM model was prepared to determine the 100-yr and Regional peak flows draining to each culvert crossing. Weighted curve numbers, watershed lengths, impervious area and watershed slopes were determined and supplied to the PCSWMM model. These values are summarized in the Table below and further details provided in Appendix I.

Table 7-1: PCSWMM Input Summary

Name	FLD1	FLD2	FLD3	FLD4
Outlet Point	OF1	OF1	OF1	OF1
Area	8.70	7.76	14.87	2.47
Flow Length	442	206	540	95
Slope	2.4%	3.2%	2.5%	4.2%
Percent Impervious (%)	13%	12%	5%	41%
N Impervious	0.013	0.013	0.013	0.013
N Pervious	0.24	0.24	0.13	0.13
Curve Number (AMC II)	44	38	68	46
Curve Number (AMC III)	57	53	80	59

Runoff determined at each node was routed through the model in a representative channel cross-section. A trapezoidal channel with a bottom width of 0.5m, side slope of 2H:1V and overall depth of 1m was selected. For the 270m where the channel intersects the Mistral property, onsite data supported the use of a 2m channel bottom.

Monument did not complete a topographic survey of the Caldwell Court and Wright Crescent crossings. Therefore, the conduits shown in the model at these junctions were selected as "Dummy conduits" and allow the model to flow freely downstream. This was conservatively selected ignoring any potential storage occurring at the upstream of the crossing.

The crossing at Ganaraska Road was modelled with a broad-crested weir to account for any overtopping of the road sag. The weir invert is set at the road sag elevation of 177.54m and estimated with a length of 30m. Results showed that in the event of Hurricane Hazel some roadway overtopping would occur with a depth of 0.19m. Monument's pre-development 100-yr floodline model determined the greatest

100-yr peak flows for the 24-hr SCS Type II distribution and the Hurricane Hazel event at the applicable nodes shown in Table 7-1.

Table 7-2: Pre-development Peak Flows

Pre-development Peak Flows (cms)		
Crossing	**100-yr**	**Regional**
Wright CRST	0.352	0.727
Caldwell CRT	0.632	1.385
U.S. Ganaraska RD	1.102	2.938
D.S. Ganaraska RD	0.825	2.776
N. Ganaraska River	1.036	3.000

Monument prepared a HEC-RAS model from the property down to the junction at the North Ganaraska River. The overall reach has an approximate channel length of 1750m, contributing area of 33.25ha, and an average longitudinal slope of 1.7%. The model was built with a reach length of 542m, one culvert crossing (Ganaraska Road) and 14 river stations (R.S.) to the outlet of North Ganaraska River (illustrated in image below). The ground data for the cross-sections within the development were cut from a terrain model from Monuments LIDAR survey and inputted into HEC-RAS. The remaining cross sections downstream were extracted from Ontario’s Digital Elevation Model LIDAR data.

Figure 3: Excerpt of HEC-RAS Model

Downstream approximately 150m from the Ganaraska Road crossing, the 100-yr and Regional flood elevations of the North Ganaraska River are met in the channel at elevations of 170.71m and 171.39m, respectively (see TSH floodplain mapping in Appendix H). These elevations were applied in the flow data *downstream constraints* as known water surface elevations (WSELs). The peak flows in Table 7-1, were applied at river stations 470, 271, 232 and 72 and WSEL results in Table 7-2.

Table 7-3: HEC-RAS Pre-development 100-yr and Regional Water Surface Elevations

	Pre-development	
River Station (m)	100-yr WSEL (m)	Regional WSEL (m)
477	177.1	177.60
436	176.56	177.60
409	176.29	177.60
384	176.06	177.60
340	175.44	177.60
304	175.46	177.60
291	175.46	177.60
283	175.46	177.60
271	175.44	177.60
254	Ganaraska Crossing	
237	174.3	174.52
222	174.09	174.32
204	173.31	173.53
152	172.29	172.47
109	171.56	171.73
72	170.71	171.39

The results show that in the event of the 100-yr, the crossing conveys the peak flow with minimal ponding occurring at the upstream side of the Ganaraska Rd Crossing. However, in the event of Hurricane Hazel, the Regional WSEL floods up the full extent of the reach length causing ponding to occur to the upstream neighboring property.

7.2.2 Post-development

Monument prepared a “100-yr Post Floodline” model to determine an increased peak flow at the Ganaraska Rd Crossing from the impervious area proposed for the development. An increase in the %impervious of Catchment Area FLD3 was applied. In the existing conditions, the %impervious for FLD3 was determined to be 5% of the total area. Monument conservatively estimated that the proposed development would result in a 20% imperviousness for the 10.88ha portion of the development lands. This increased FLD3’s total imperviousness to 19%. The following post-development peak flows were determined and supplied to the HEC-RAS model:

Table 7-4: 100-yr and Regional Post-development Peak Flows

Post-development Peak Flows (cms)		
Crossing	100-yr	Regional
Wright CRST	0.352	0.730
Caldwell CRT	0.632	1.385
U.S. Ganaraska RD	1.102	3.063
D.S. Ganaraska RD	0.825	2.792
N. Ganaraska River	1.036	3.016

Note that the 100-yr peak flows were not adjusted since quantity control measures will maintain pre-development 100-yr flow levels. Similar to the pre-development WSELs Hurricane Hazel creates a large amount of ponding upstream of the crossing as it overtops the Ganaraska Road. Monument plotted these WSELs on the drawing provided in Appendix J as a Post-development Floodplain Drawing.

Table 7-5: HEC-RAS Post-development 100-yr and Regional Water Surface Elevations

	Post-development	
River Station (m)	100-yr WSEL (m)	Regional WSEL (m)
477	177.10	177.60
436	176.56	177.60
409	176.29	177.60
384	176.06	177.60
340	175.44	177.60
304	175.48	177.60
291	175.48	177.60
283	175..48	177.60
271	175.45	177.60
254	Ganaraska Crossing	
237	174.3	174.52
222	174.09	174.32
204	173.31	173.53
152	172.29	172.47
109	171.56	171.73
72	170.71	171.39

7.2.3 Proposed Culvert Replacement

With the extent of the flooding occurring in the development lands shown on the Floodline Drawing provided in Appendix H, Monument recommends that the culvert crossing at Ganaraska Road be upsized to improve the efficiency of the Ganaraska Road crossing. This would result in less flooding on the development lands and eliminate overtopping of the road in the Regional Event.

The Floodplain Mapping prepared by Totten and Sims (1990) did not account for this additional storage upstream as illustrated on the Floodplain Map provided by GRCA (see Appendix H). This was a conservative assumption at the time to account for all water flowing freely from the South Tributary to North Ganaraska River. Therefore, in the event of the 100-yr or Hurricane Hazel upsizing the crossing would not impact the original floodline. However, by upsizing the culvert it would result in greater peak flows immediately downstream at the Ganaraska Rd Crossing due to the enlarged opening not creating as much energy loss. To better understand the impacts, Monument prepared a “Post-development Upgraded Culvert” model in PCSWMM.

Under the existing conditions the 900mm culvert has an invert elevation of 174.32m and a resulting obvert elevation of 175.22m. The depth of cover over the crossing is 2.1m with the edge of travelled lane elevation directly above the crossing at 177.32m. Therefore, cover over an enlarged culvert would not be an issue.

In the proposed conditions, Monument selected a 1400mm CSP culvert with an embedded invert depth of 0.30m. With the selected configuration, the PCSWMM model determined the following Post-development Upgraded Culvert peak flows.

Table 7-6: Peak Flows under Post-development Conditions with Culvert Replacement

Post-development Culvert Upgrade Peak Flows (cms)		
Crossing	**100-yr**	**Regional**
Wright CRST	0.352	0.730
Caldwell CRT	0.632	1.385
U.S. Ganaraska RD	1.102	3.063
D.S. Ganaraska RD	0.859	2.945
N. Ganaraska River	1.099	3.196

A comparison between the existing configuration and proposed is provided in the Table 7-6.

Table 7-7: Pre-development Peak Flows vs. Post-development Peak Flows with Culvert Replacement

Peak Flow Comparison Pre to Post New Culvert						
Crossing	100-yr			Regional		
	Pre	Post (N.CLVT)	Increase	Pre	Post (N.CLVT)	Increase
Wright CRST	0.352	0.352	0.000	0.727	0.730	0.003
Caldwell CRT	0.632	0.632	0.000	1.385	1.385	0.000
U.S. Ganaraska RD	1.102	1.102	0.000	2.938	3.063	0.125
D.S. Ganaraska RD	0.825	0.859	0.034	2.776	2.945	0.169
N. Ganaraska River	1.036	1.099	0.063	3.000	3.196	0.196

As illustrated in Table 7-6, an enlarged culvert would slightly increase peak flows at the outlet of the new culvert and ultimately downstream to North Ganaraska River. These peak flow values were then adjusted in the flow data in HEC-RAS generating new WSELs shown in Table 7-7.

Table 7-8: HEC-RAS Water Surface Elevations in Post-Development Condition with Culvert Replacement

	Post-Development (N. CLVT)	
River Station (m)	100-yr WSEL (m)	Regional WSEL (m)
477	177.1	177.19
436	176.56	176.60
409	176.29	176.39
384	176.06	176.06
340	175.44	176.02
304	175.20	176.02
291	175.20	176.02
283	175.20	176.02
271	175.15	175.93
254	Ganaraska Crossing	
237	174.31	174.54
222	174.09	174.33
204.0	173.32	173.54
152.0	172.29	172.48
109.0	171.57	171.74
72.0	170.71	171.39

As illustrated in the results above, the enlarged culvert drastically lowers WSELs upstream and eliminates the extent of backwater effect to only 70m within the reach onsite. The max flood elevation at the culvert crossing is 176.02 m which is now 1.52m below Ganaraska Road sag, eliminating water from overtopping the road. A comparison of the WSELs downstream of the crossing are provided below.

Table 7-9: Comparison in HEC-RAS Water Surface Elevations between Post-development vs. Post-development with Culvert Replacement

River Station (m)	100-yr WSELs (m)			Regional WSELs (m)		
	Pre	Post (N.CLVT)	Change WSEL	Pre	Post (N.CLVT)	Change WSEL
Ganaraska Crossing						
237	174.3	174.31	0.01	174.52	174.54	0.02
222	174.09	174.09	0	174.32	174.33	0.01
204	173.31	173.32	0.01	173.53	173.54	0.01
152	172.29	172.29	0	172.47	172.48	0.01
109	171.56	171.57	0.01	171.73	171.74	0.01
72	170.71	170.71	0	171.39	171.39	0

The results shown in Table 7-8 provide a comparison of the Post WSELs downstream with the existing culvert in place against the Post WSELs with the new culvert crossing. The results show that when upsizing the culvert, WSELs will be minimally impacted to less than 2cms. These WSELs have been plotted on the Post-development Floodline Drawing in Appendix H. The drawing reveals that in both the pre- and post-development conditions, the existing residential house at 3893 Ganaraska Road is within the 100-yr and Regional Floodline. Downstream from this property, the banks hold the flows in both event within the channel.

Provided the following review, Monument believes that the recommendation of upsizing the culvert would benefit the development, the County roadway, and eliminate potential risk of flooding of the upstream neighbours. Therefore, the proposed conditions discussed herewithin this report will be based on the 100-yr and Regional flood elevations generated with the culvert replacement. Further design details on the new culvert crossing will be provided at the detailed design stage.

7.3 Proposed Entrance

The concept plan for this development proposes that a roadway be built to provide a secondary access from Ganaraska Road. Due to separation distances for arterial intersections, the proposed access will intersect Ganaraska Road through the small floodplain area filling a small rounded low area on the west side of the watercourse (see Floodplain Drawing Appendix H).

To allow this access to proceed, a new geometry was created in HEC-RAS under the proposed conditions to add an obstruction to the applicable river stations. An obstruction was added in the river stations to the extent of the right-of-way to river stations 271, 283, 291, and 304. The obstruction height was set to an elevation of 177.60m. The cross section in which this ponding occurs is 291. See the below screen shot of this cross section with the obstruction in place. The simulation was run separately to see if this would increase the proposed WSELs.

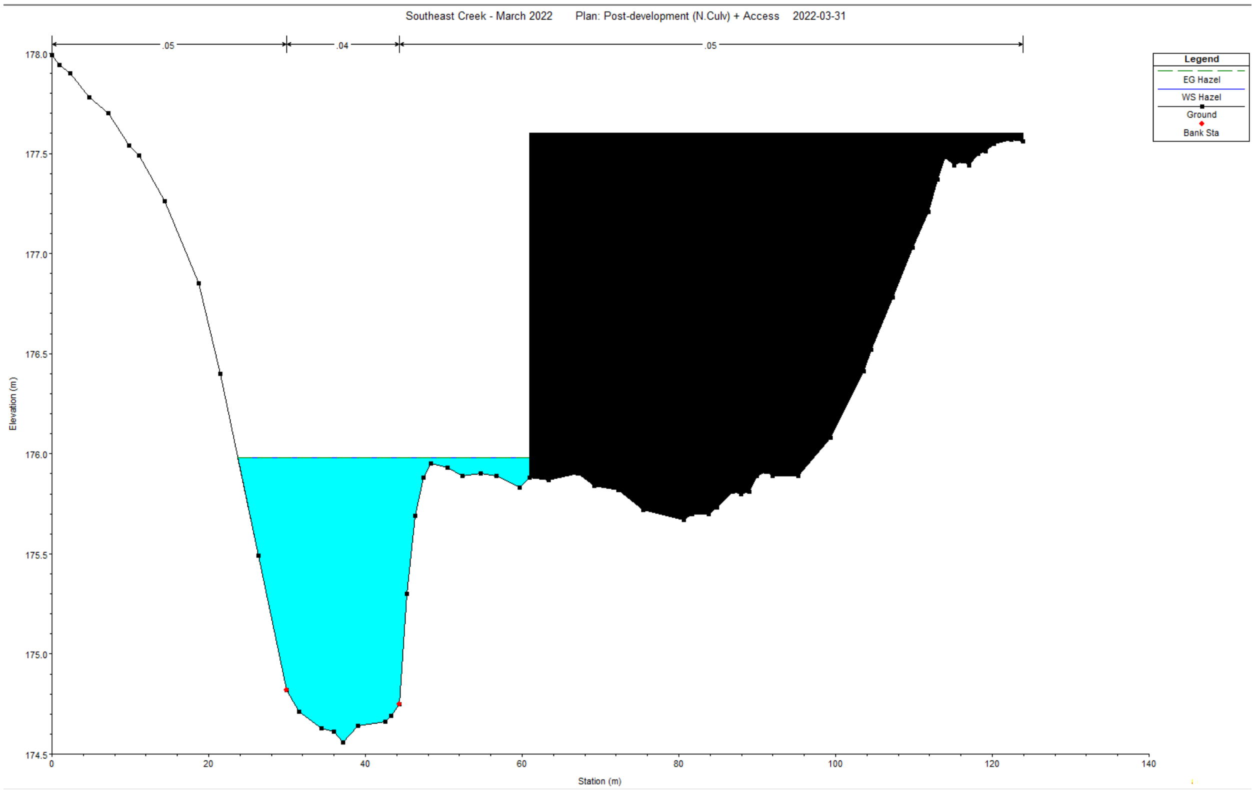

Figure 4: HEC-RAS Rivers Station 291 with User Defined Obstruction

The results showed no change and therefore is not a concern. To gauge an understanding of the storage volume loss in this instance, Monument prepared a cut and fill analysis in Civil3D. The analysis determined a loss of approximately 80m^3.

8 Peak Flows

8.1 Pre-development

Monument created a "Pre-development" PCSWMM model to determine runoff to each outfall. As discussed in Section 4, the parameters are selected based on their land use type and soil condition. Table 8-1 provides a summary of the PCSWMM input parameters for each catchment. A detailed look into the calculations is provided in the Methodology Section of Appendix F.

Table 8-1: Pre-development PCSWMM Input Summary

Name	101	102	103	104	104EXT
Outlet Point	OF1	OF1	OF1	OF1	OF2
Area	6.99	1.23	1.43	1.49	1.10
Flow Length	540	100	255	131	157
Slope	2.5%	2.0%	3.1%	4.1%	2.1%
Percent Impervious (%)	0	0	0	0	0
N Impervious	0.013	0.013	0.013	0.013	0.013
N Pervious	0.06	0.06	0.13	0.13	0.20
Curve Number	72	64	46	67	34

Name	105	106	107
Outlet Point	OF4	OF3	OF2
Area	1.37	2.70	9.52
Flow Length	232	267	393
Slope	4.3%	4.7%	2.5%
Percent Impervious (%)	0	0	0
N Impervious	0.013	0.013	0.013
N Pervious	0.06	0.06	0.13
Curve Number	82	82	55

As recommended by the GRCA SWM Technical Guideline, Monument determine peak flows for the 6hr, 12hr and 24hr storms for both SCS Type II and Chicago distributions. As shown in Table 8-1, the greatest peaks were determined in the 24hr SCS Type II distribution.

Table 8-2: Pre-development Flows

Duration	Pre-development Peak Flows (cms)				
	OF1	OF2	OF3	OF4	Reservoir
SCS Type II					
24hr	**0.602**	**0.122**	**0.382**	**0.198**	0.706
12hr	0.450	0.084	0.323	0.165	0.566
6hr	0.407	0.086	0.246	0.122	0.449
Chicago					
24hr	0.377	0.093	0.246	0.152	0.497
12hr	0.291	0.072	0.187	0.119	0.379
6hr	0.259	0.058	0.172	0.110	0.346

Table 8-3: Post-uncontrolled Peak Flows to each Outlet:

Duration	Uncontrolled Peak Flows (cms)				
	OF1	OF2	OF3	OF4	Reservoir
SCS Type II					
24hr	0.889	0.067	0.764	0.075	0.905
12hr	0.751	0.045	0.625	0.061	0.730
6hr	0.559	0.046	0.526	0.055	0.625
Chicago					
24hr	0.687	0.050	0.583	0.071	0.688
12hr	0.562	0.039	0.483	0.056	0.558
6hr	0.534	0.032	0.451	0.052	0.519

Note: AES distribution for 6hr, 12hr and 24hr durations are not offered in PCSWMM and therefore were not considered for this analysis.

8.2 Post-development

A proposed drainage plan has been provided in Appendix J. As illustrated on this plan, Monument determined a total of 10 different post-development catchment areas draining to the four different Outfall locations. Catchment areas 200, 201, 202A, 202B and 203 will drain to Outfall #1.

Catchment 200

Delineates the rear yards of lots 1-4. It also contains the small external area draining into the old farmers pond shown on the Existing Catchment Area Drawing. A rear yard swale is proposed to collect runoff from this area conveyed directly to Outfall #1.

Catchment 201
Contains majority of Street ‘A’ and the front yards of lots 1 to 5 and 21 to 41. This area will drain within the roadside ditch directly to the stormwater management facility at the entrance to the south (SWMF#1).

Catchment 202A
This catchment area contains the small wetland compensation area adjacent to the South Tributary. It also includes a small portion of lots 41 to 43 that will drain off the rear uncontrolled.

Catchment 202B
Will direct drainage from the external lands 104EXT direct to the South Tributary. A cross culvert in Street ‘C’ will convey water from the external lands in a rear yard swale behind lots 33 to 40. Runoff from the rear of these lots will also contribute and be released uncontrolled. Further details of this swale will be provided in the Conveyance Section.

Catchment 203
Access to this portion of the property will be achieved through Frost Avenue. This block has been designated for increased density such as townhouses or a small condo complex. At this time, it is unknown the amount of hardened area that will be proposed. For simplicity, Monument assumed that 25% of this area will be hardened and allowed to runoff uncontrolled. Quality control devices will need to be reviewed at the time of site plan.

The remaining catchment areas will be directed to the Reservoir through Outfalls#2, #3 and #4.

Catchment 300A
At the north end of Street ‘A’ a cul-de-sac will push development into the woodland area past Porter Crescent. This will extend past the existing drainage divide of Outfall #1. To maintain existing drainage patterns and eliminate the need for a third SWM facility, a drainage easement has been proposed to convey runoff west to Street ‘B’. A high point will be constructed at the intersection of Street ‘A’ and Street ‘C’ where the roadside ditches will tie into a proposed grassed swale in the easement. Runoff will be conveyed in a grassed swale and into the second SWMF (“SWMF#2”). Further discussion into the conveyance swale will be discussed in Section 11.

Catchment 300B
A high point will be constructed at the south end of Street ‘B’ to direct drainage north to SWMF#2. Rear yard swales will split behind lots 16 and 19 to direct drainage into the roadside ditches.

Catchment 400 & 500

Are small catchments that cannot drain to a localized SWMF. By containing the ditches along the side of Street 'B' the drainage area to outlet #3 and outlet #4 have been reduced and runoff will drain uncontrolled. A rear yard swale starting behind Lot 5 and continue along the perimeter of the property to lot 8 will control runoff directly to the existing 400mm CSP at Mill Street.

Catchment 600

Contains the ending cul-de-sac at of Street 'B'. The remaining area contains the wooded portions at the rear of lots 12 to 14 and 28 to 32. Drainage at these rears will not be controlled and will drain off to the North watercourse. The houses in these lots will be pushed as far forward to the frontage to minimize impacts to the woodlands. A ditch will perimeter around the cul-de-sac and drain out at the property line of Lot 12 and 13. Further details on this swale will be provided in Section 10.

Subarea Routing

Post-development catchments are divided into pervious and impervious subareas. Surface runoff can infiltrate in pervious surfaces represented by the curve number, where impervious areas will directly runoff. Overland flow is then generated from each subarea by approximating them as non-linear reservoirs (see image below).

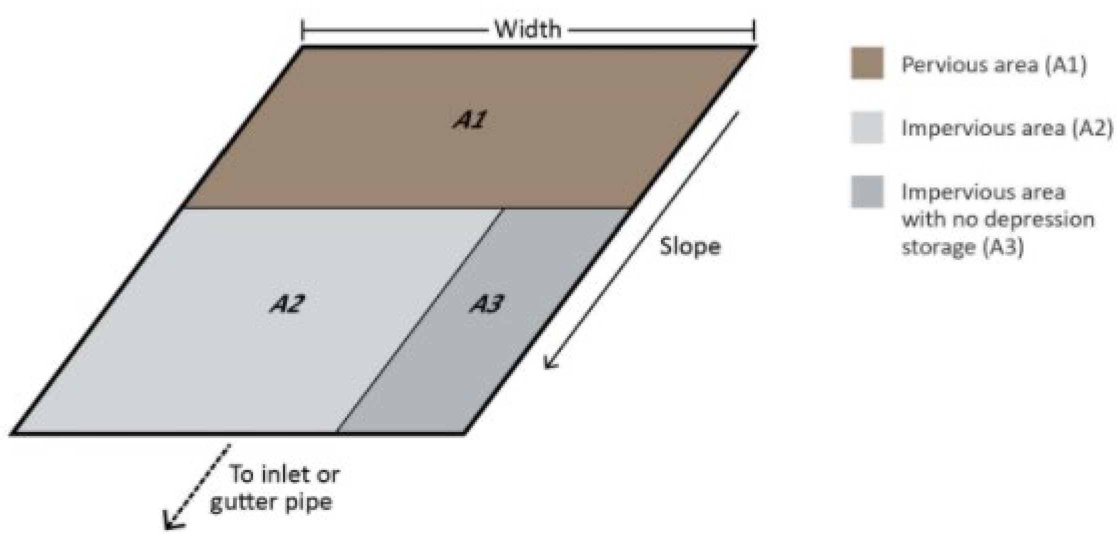

Figure 5: PCSWMM Support Excerpt describing Subarea Routing

(https://support.chiwater.com/80217/subarea-routing)

Typically, the overland flow from each subarea is independently routed to the outlet, however, PCSWMM allows the user to further subdivide runoff between subareas using the subarea routing tool. This creates internal routing between pervious and impervious surfaces. (e.g. roofs onto lawn surfaces). There are three selection options for the subarea routing tool:

- **IMPERV:** some percentage of the runoff from the pervious area is directed to the impervious area and then to outlet,
- **PERV:** some percentage of the runoff from the impervious area is directed on the pervious area and then to outlet,
- **OUTLET**: runoff from each subarea is routed directly to the outlet. (e.g. no subarea routing)

The image below illustrates the difference between the IMPERV (left) and PERV (right) routing.

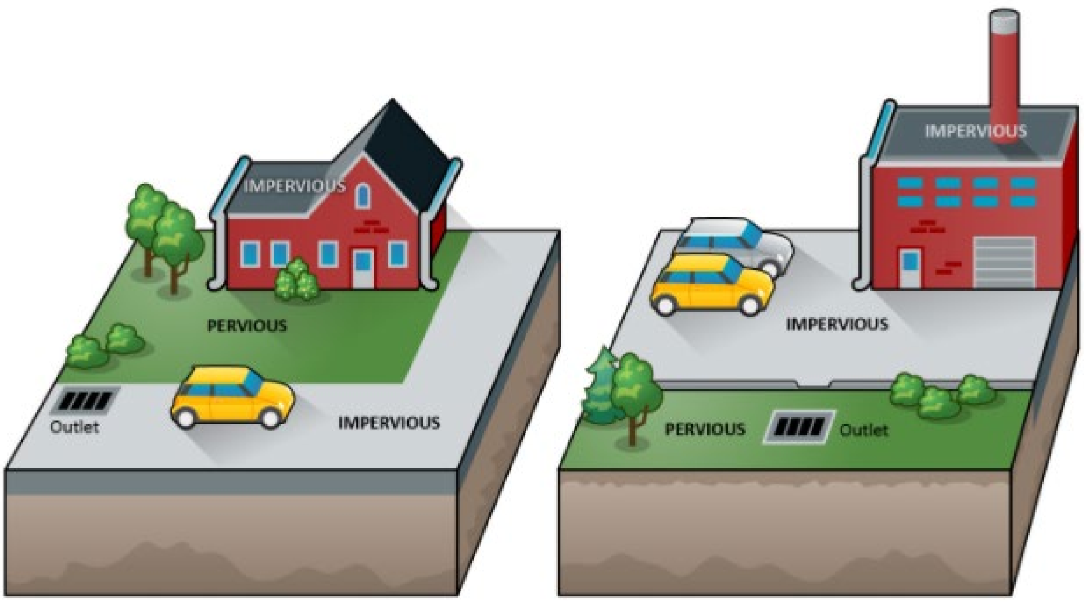

Figure 6: Excerpt from PCSWMM Support website illustrating Subarea Routing

(https://support.chiwater.com/80217/subarea-routing)

Monument selected to use the PERV command for each catchment to route runoff from rooftops onto the grassed areas. This is represented by expressing a percent routed for each catchment (i.e. area of rooftop over the total catchment area). For Catchment Areas 201, 300A and 300B, a conservative percent routed of 30% was selected. The remaining catchments were conservatively set to 80% as runoff from these areas are the back half of the roof tops to the rear yards.

The input summary in Table 8-4 provides the selected parameters used for these catchments. More details are provided in the Methodology Section of the Appendix.

Table 8-4: Post-development PCSWMM Input Summary

Name	200	201	202A	202B	203
Outlet Point	OF1	OF1	OF1	OF1	OF1
Area	1.31	3.90	2.25	1.62	1.23
Flow Length	100	150	120	255	58
Slope	1%	3.5%	4%	1%	2%
Percent Impervious (%)	4.4%	27.3%	7.7%	7.2%	25.0%
N Impervious	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.25	0.06
Subarea Routing (%)	80%	30%	80%	80%	-
Curve Number	60	73	72	72	72

Name	300A	300B	400	500	600
Outlet Point	OF3	OF3	OF4	OF3	OF2
Area (ha)	5.60	2.43	0.78	0.65	4.70
Flow Length (m)	140	66	50	75	239
Slope	2.5%	2.0%	2.0%	3.0%	2.3%
Percent Impervious (%)	18.7%	30.6%	15.6%	4.4%	4.9%
N Impervious	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.25	0.25
Subarea Routing	30%	30%	80%	80%	80%
Curve Number	75	75	78	78	56

Site Imperviousness

Based on the anticipated lot grading, Monument used the proposed drainage plan to determine an impervious area for each catchment. As illustrated in Table 8-5, it is anticipated that the overall site will have a percent impervious of 16%.

Table 8-5: Estimated Site Imperviousness within each Catchment Area

Catchment	Area (ha)	Impervious Area (ha)	Percent Impervious
200	1.31	0.06	4.4%
201	3.90	1.07	27.3%
202A	2.25	0.17	7.7%
202B	1.62	0.12	7.2%
203	1.23	0.31	25.0%
300A	5.60	1.04	18.7%
300B	2.43	0.74	30.6%
400	0.78	0.12	15.6%
500	0.65	0.03	4.4%
600	4.70	0.20	4.3%
Total:	**24.46**	**3.86**	**15.8%**

A "Post-development Uncontrolled" model was simulated to look at the magnitude in which the proposed development would increase runoff. The uncontrolled peak flows generated from this model are displayed in Table 8-6.

Table 8-6: Post-controlled Peak Flows to each Outfall

Duration	Uncontrolled Peak Flows (cms)				
	OF1	OF2	OF3	OF4	Reservoir
SCS Type II					
24hr	**0.908**	**0.067**	**0.764**	**0.075**	**0.905**
12hr	0.716	0.045	0.625	0.061	0.730
6hr	0.563	0.046	0.526	0.055	0.625
Chicago					
24hr	0.632	0.050	0.583	0.071	**0.688**
12hr	0.493	0.039	0.483	0.056	0.558
6hr	0.462	0.032	0.451	0.052	0.519

Note: Peak flows from catchment area 300A and 300B will be routed to Outfall #3 in the post-development conditions. Further details are provided below.

Comparing the pre- to post-uncontrolled peak flows, quantity control is required for outfall #1 and outfall #3 (see tables 8-7 and 8-8). The peak flows shown in red signify that the pre-development levels were not met and therefore quantity control is required.

Note: To avoid over-controlling runoff under the post development conditions, the flow rates were balanced between the three outlets that ultimately discharge to the reservoir instead of matching predev rates for each outlet. Outlet conveyance capacity is reviewed in Section 11 to confirm the balanced flow rates across the 3 outlets can safely be conveyed to the reservoir.

Table 8-7: Pre-development Peak Flow Comparison to Outfall #1 and #2

Peak Flow Comparison (cms)				
Duration	OF#1		OF#2	
	Pre	Post Uncontrolled	Pre	Post Uncontrolled
SCS Type II				
24hr	0.602	0.889	0.122	0.067
12hr	0.450	0.751	0.084	0.045
6hr	0.407	0.559	0.086	0.046
Chicago				
24hr	0.377	0.687	0.093	0.050
12hr	0.291	0.562	0.072	0.039
6hr	0.259	0.534	0.058	0.032

Table 8-8: Pre-development Peak Flow Comparison to Outfall #3 and #4

Peak Flow Comparison (cms)						
Duration	OF#3		OF#4		Reservoir	
	Pre	Post Uncontrolled	Pre	Post Uncontrolled	Pre	Post Uncontrolled
SCS Type II						
24hr	0.382	0.764	0.212	0.075	0.706	0.905
12hr	0.323	0.625	0.180	0.061	0.566	0.730
6hr	0.246	0.526	0.130	0.055	0.449	0.625
Chicago						
24hr	0.294	0.583	0.168	0.071	0.497	0.688
12hr	0.231	0.483	0.135	0.056	0.379	0.558
6hr	0.213	0.451	0.125	0.052	0.346	0.519

8.3 Storage Requirements

The storage calculator in PCSWMM computes the volume of storage required to reduce uncontrolled peak flows to a user-defined design flow (pre-development levels). This can be used to provide a quick estimate by assuming a linear relationship between storage volume and discharge. The storage calculator was used to estimate the greatest storge that would be required for each of the two stormwater management facilities.

Table 8-9: Storage Requirement Estimate to each Outfall

Duration	Storage Volume (m^3)	
	OF1	Reservoir
SCS Type II		
24hr	414	416
12hr	407	421
6hr	469	**616**
Chicago		
24hr	**531**	499
12hr	522	567
6hr	486	489

The greatest storage requirements for Outfall #1 were estimated in the 24hr Chicago storm requiring 531m^3 of active storage. At the Reservoir, the greatest storage will be generated in the 6hr SCS Type II storm with 616m^3. Therefore, the pre-development model was re-run to determine the 2-100yr peak flows for each storm. These will be the target release rates for each outfall.

Table 8-10: Pre-development Release Rates

Duration	Target Flows (cms)	
	OF1	Reservoir
2-yr	0.046	0.071
5-yr	0.102	0.143
10-yr	0.153	0.201
25-yr	0.236	0.291
50-yr	0.302	0.365
100-yr	0.376	0.449

Note:

OF#1 target flows were determined in 24hr Chicago storm
OF#2 target flows were determined in 6hr SCS Type II storm

9 Stormwater Management

To meet the quantity control design objective, two separate stormwater management facilities have been proposed for the site. The first stormwater management facility (SWMF#1) is the proposed pond located at the new Ganaraska Road entrance. The second facility (SWMF#2) is located further north near the existing farm entrance from Mill Street. This section will focus on the conceptual design of these facilities to satisfy the design objective. A conceptual layout has been provided on the post-development Catchment Area Drawing in Appendix J.

9.1 Stormwater Management Facility #1

Monument is proposing that this facility be a continuous flow dry pond. The roadside ditch will inlet at the pond invert of 176.30m and slope down at an average pond bottom slope of 0.80% to the outlet. At the outlet the total pond depth will reach 1.2m with a bottom of pond elevation of 175.40m and top of berm elevation of 176.60m. A 3.0m wide access trail will be constructed at the top of pond off Street 'A' for maintenance purposes.

A centralized 1.0m wide landscape berm will be constructed in the pond to extend the overall flow length to the outlet. Side slopes for this berm will be 3H:1V interior while the exterior side slopes for the remainder of the pond will be 5H:1V. A small attenuation berm will be constructed at the end of the landscape berm. This small berm will have a total height of 0.30m constructed from heavy granular material similar to a rock check dam. A small 150mm deep low flow channel will also be installed along the center of the flow path to the outlet of the pond. Further details will be provided at the time of detailed design.

Quantity Control

Monument extracted preliminary storage volumes from Civil 3D to determine a stage-storage-discharge relationship. This relationship will be used to size the outlet structures and then supplied to PCSWMM in the form of a stage-area relationship (see Appendix K).

The conceptual design resulted in an active storage volume of 696m^3 with an additional storage volume of 499m^3 available to the top of berm. This is greater than the storage requirement estimated for in section 8.3.

Quality Target

The objective for dry facilities is to meet TSS removal efficiency of 60% (Basic - Level 3) as per Table 3.2 of the MECP 2003 SWM Guideline. Table 3.2 provides a quick calculation to determine a required extended detention storage based on the impervious area and a 24hr drawdown time. The total contributing area to the pond is 3.91ha (Catchment 201) with a total impervious area of 1.09ha. This results in the contributing area to have a total %imperviousness of 28%. However, runoff from rooftops will not be credited in this calculation and deducted from the impervious area. The remaining area is 0.87ha with an adjusted %impervious of 22% to be used in Table 3.2.

Table 3.2 does not have a pre-determined multiplication factor for impervious levels less than 35%. Therefore, Monument extrapolated a value of 51m^3/ha of storage from the 35% and 55% assigned factors. Multiplied by the contributing area (3.91ha,) the required detention volume is 220m^3.

Outlet Size

The outlet will need to restrict runoff to the required release rates while providing a 24hr drawdown time for the extended detention volume. These were modelled in PCSWMM using the weir and orifice structure sized accordingly to the stage-storage-discharge sheet. To try and meet the drawdown time requirement the minimum orifice size of 75mm was selected. This results in an approximate 16hr drawdown time, 0.42m max depth and storage volume of 154m^3 in the event of quality storm. An excerpt of the quality storm routed through the pond storage volume from PCSWMM graph tab is provided below.

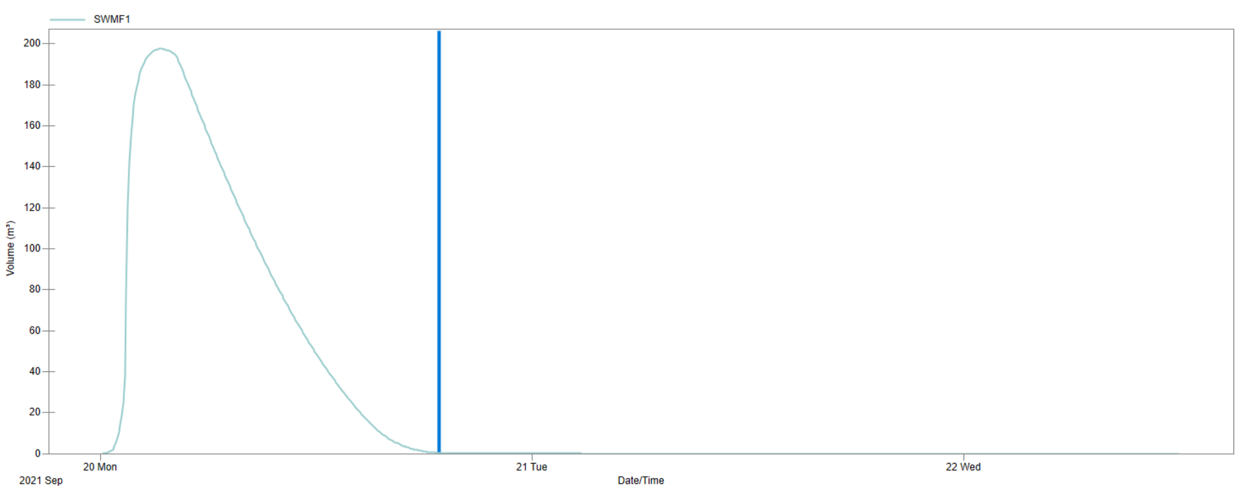

Figure 7: PCSWMM Excerpt of Quality Storm Routed through SWMF#1

The dry pond does not meet the quality objectives as described above. This is expected since the contributing area to the pond is less than 5ha. With a 75mm orifice size an exception of a 12hr drawdown time is acceptable for these facilities to prevent clogging. The MECP 2003 SWM guideline also recommends that dry ponds with drainage areas less than 5ha receive pre-treatment to help reach the targeted TSS removal efficiency. To enhance the quality treatment, the inlet of the pond will be lined with river stone and filter fabric to the end of the attenuation berm.
The active storage will be controlled using an in-structure sharp-crested weir. The sharp-crested weir will be set at an invert elevation of 176.05m and have opening length of 0.60m. Further details on the outlet structure will be provided at the time of detail design.

With the two theoretical outlet structures set, the following release rates were determined at the downstream node of the pond, and further overall controlled flows at Outfall #1:

Table 9-1: SWMF#1 Pond Release Rates and Quantity Control Check

		Peak Flow at Outfall#1		
Duration	Pond Outfall	Target Flows	Controlled Flow	Achieved Quantity Control
2-yr	0.015	0.035	0.031	✓
5-yr	0.039	0.092	0.084	✓
10-yr	0.056	0.152	0.135	✓
25-yr	0.109	0.243	0.236	✓
50-yr	0.136	0.320	0.316	✓
100-yr	0.171	0.407	0.398	✓

An overall review of the ponds function under each event is summarized in Table 9-2.

Table 9-2: Function of SWMF#1 under each return period event

Duration	Inflow	Storage (m^3)	Max Depth (m)	WSEL (m)
2-yr	0.118	357	0.29	176.01
5-yr	0.170	460	0.34	176.11
10-yr	0.210	503	0.39	176.15
25-yr	0.270	562	0.42	176.20
50-yr	0.319	607	0.45	176.23
100-yr	0.371	655	0.49	176.27

Therefore, the conceptual SWM facility is adequately sized to attenuate all return period events to the pre-development levels to the quantity control objective.

Emergency Spillway

An emergency spillway is provided at the top of pond set at the invert of the active storage (176.30m). The objective for the spillway is to provide safe conveyance out of the pond in the event of the Regional Storm or in case of emergency events (i.e. blockage) and to contain the flow from overtopping the berm. The post-controlled model was simulated with the Regional Storm event and curve numbers increased to AMC III saturated soil conditions (Catchment 201 only). The inflow in this condition was determined to be 0.534cms. The model also allows the user to define an initial depth in the storage node. This option was used to model the backwater effect from the regional floodline (175.98m) which would result in a depth of 0.58m in the pond. As per the stage-storage-discharge sheet the Emergency Spillway has a flow capacity of 0.823m^3/s to the top of berm. The PCSWMM results show that in the event of the

Regional flood elevation and inflow to the pond that the broad-crested weir would be engaged with a total flow depth of 0.14m and overflow over weir of 0.30m^3/s.

Street 'A' Culvert Crossing

The proposed culvert crossing in Street 'A' is the final outlet structure for SWMF#1. The Regional Storm will also create a backwater effect in this culvert potential causing the structure to operate under outlet control. A 1000mm CSP culvert was modelled in PCSWMM for this crossing. The upstream and downstream nodes at this crossing were also set with an initial depth of 0.68m and simulated to determine the upstream hydraulic grade line. The results showed that the max WSEL would be 176.03m which is 0.24m lower than the broad-crested weir. Therefore, in the event of the Regional Flood the outlet structure from the pond would drain to Outfall #1.

9.2 Stormwater Management Facility #2

The second SWMF is proposed to be an extended detention wet pond. This was selected to allow the outflows of the pond to be pulled from the bottom of the permeant pool to discharge cold water downstream. The total contributing area to this facility is 7.35ha with a percent impervious of 24% from Catchment 300A and 300B.

Borehole #5 and #7 of Terraspec's geotechnical investigation revealed ground water depths of 1.5 to 1.6m from ground surface (see excerpt in Appendix A). As per the MECP 2003 guidelines, stormwater management ponds (Wet & Dry ponds) can be constructed within groundwater depths if they are lined with a clay liner. This is to stop potential groundwater contamination through infiltration. The ground water depth is estimated to be at an elevation 178.50m at the inlet of the pond.

Monument has proposed that the top of permeant pool be set at an elevation of 178.80m. The top of active storage will be set to 179.50m which is 0.30m lower than the top of berm elevation of 179.80m. The permeant pool depth is 1.0m with side slopes of 3H:1V and the remaining depth to the top of berm having a side slope of 5H:1V.

The centerline of Street 'B' has been designed to allow major flows into the pond at a road sag elevation of 180.60m. A 3.0m wide maintenance trail will also be provided along the perimeter of the pond accessed from Street 'B'. A third access point can be provided from the existing Mill Street farm entrance alongside the facilities outlet, provided fill limits for this access remain outside of the Regional floodline.

Quantity Volumes

Monument extracted preliminary storage volumes from Civil 3D to determine a stage-storage-discharge relationship. This relationship will be used to size the outlet structures and then supplied to PCSWMM in the form of a stage-area relationship (Appendix K).

The conceptual design resulted in an active storage volume of 962m^3 with an additional storage volume of 533m^3 available to the top of berm. This is greater than the storage requirement estimated for in Section 8.3. The total permanent pool will have a volume of 757.5m3 with 324m^3 subsiding in the forebay.

Quality Target

The wet pond is to provide a total TSS removal efficiency of 80% following Table 3.2 of MECP's Technical Guideline. The total contributing area to the pond is 7.35ha (Catchment 300A + 300B) with a total assumed impervious area of 1.78ha. This results in the contributing area to have a total %imperviousness of 25%. However, runoff from rooftops will not be credited in this calculation and deducted from the impervious area. The remaining impervious area is 1.15ha with an adjusted %impervious of 16% to be used in Table 3.2.

Monument extrapolated a value of 30m3/ha of storage from the 35% and 55% assigned factors from MECP Table 3.2. Multiplied by the contributing area (7.35ha) the required permanent pool volume is 220m^3. The total extended detention volume is determined using 40m^3/ha. Therefore, the required extended detention volume target will be 295m^3. Further details on quality controls are provided in Section 10.

Outlet Size

Two outlet structures have been sized to meet both the quality and quantity control targets. An orifice pipe will act as the first outlet to attenuate minor storms and the 25mm, 4hr Chicago quality event. The main objective of this outlet is to meet the 24hr detention time.

The orifice was sized to have a diameter of 75mm set at the top of the permeant pool elevation. With this orifice a detention time of 35hr was achieved as displayed in the PCSWMM excerpt in Figure 8.

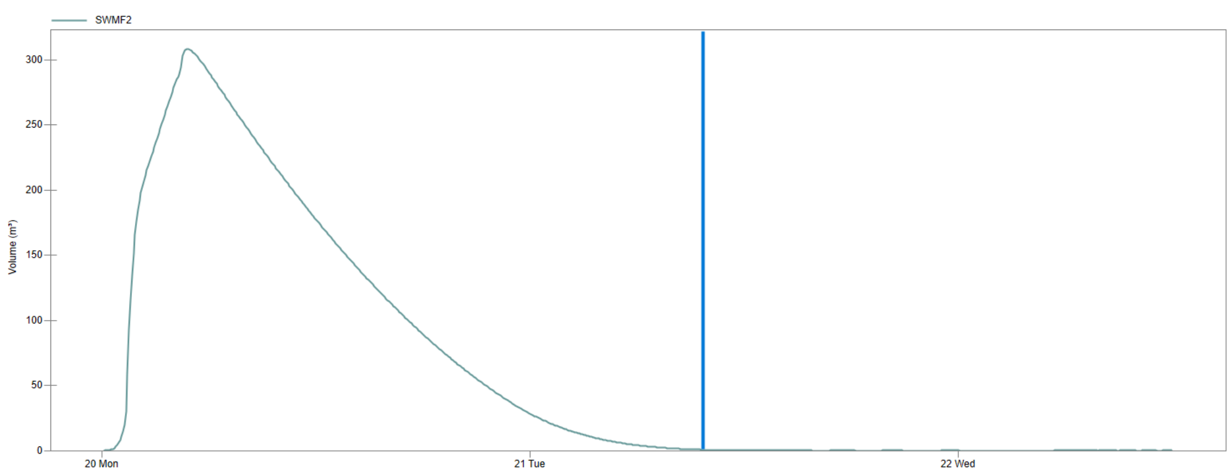

Figure 8: PCSWMM Excerpt of Quality Event through SWMF#2

The results showed that the total storage volume used in this event was 308m^3 with a maximum depth of 0.30m. Therefore, SWMF#2 provides the 24hr drawdown time and exceeds the required extended detention volume of 290m^3 satisfying both quality control targets.

Monument proposed that the second outlet be designed using a 0.8m long sharp-crested weir set to the invert of 179.10m. This outlet will attenuate the major storm events inletting into the pond. With the two theoretical outlet structures modelled in the Post-development Controlled model the following flows were determined at the reservoir.

Table 9-3: SWMF#2 Pond Release Rates and Quantity Control Check

		Peak Flow at Reservoir (cms)		
Duration	Pond Outfall	Target Flow	Controlled Flow	Achieved Quantity Control
2-yr	0.036	0.071	0.044	✓
5-yr	0.087	0.143	0.111	✓
10-yr	0.135	0.201	0.174	✓
25-yr	0.204	0.291	0.268	✓
50-yr	0.261	0.365	0.345	✓
100-yr	0.321	0.449	0.430	✓

An overview of the ponds function is provided in Table 9-4.

Table 9-4: Function of SWMF#2 under each return period event

Duration	Inflow	Storage (m^3)	Max Depth (m)	WSEL (m)
2-yr	0.134	407	0.37	179.17
5-yr	0.186	506	0.44	179.24
10-yr	0.252	583	0.49	179.29
25-yr	0.349	685	0.56	179.36
50-yr	0.426	764	0.61	179.41
100-yr	0.509	845	0.66	179.46

Therefore, the conceptual SWM facility is adequately sized to attenuate all return period events to their pre-development levels and meet the quantity control objective.

Emergency Spillway

Similar to SWMF#1, a broad-crested weir will be constructed at the top of berm to safely convey the Regional storm through the pond to Outfall #3. The Regional Storm simulated on the post-controlled model under saturated soil conditions (AMCIII) resulted in an inflow to the facility of 1.15m^3/s. The emergency spillway will be set to the top of active storage (179.50m) and have a 3.0m weir length. The model resulted in the weir being engaged in this event with a total depth of 0.10m over the invert and discharge of 0.112m^3/s. Therefore, the Regional Storm will be safely conveyed through the SWMF. Outlet details will be required to Outfall #3 at the time of detailed design.

Outfall #3 Culvert

As described in Section 8, peak flows from the SWM facility will be conveyed to the 500mm corrugated plastic pipe at Outfall #3. As discussed, quantity control was provided for the Reservoir as it is the ultimate receiver from Outfalls 2, 3 and 4. This will result in slightly increased flows as illustrated in Table 9-5. Note: Peak flows correspond to the 6hr SCS Type II storm.

Table 9-5: Pre-development Peak Flows at Outfall #3 vs Post-controlled Peak Flows

Duration	Pre-development			Post-development (Controlled)		
	Inflow	HW Depth	WSEL	Inflow	Depth	WSEL
2-yr	0.05	0.16	173.77	0.04	0.16	173.77
5-yr	0.09	0.29	173.90	0.09	0.29	173.90
10-yr	0.13	0.36	173.97	0.15	0.39	174.00
25-yr	0.17	0.45	174.06	0.22	0.59	174.20
50-yr	0.21	0.58	174.19	0.28	Overtop	-
100-yr	0.25	Overtop	-	0.35	Overtop	-

As per the MTO Highway Drainage Manual WC-1, Mill Street is classified as a local roadway with the required design storm to be conveyed through the crossing without overtopping the road in the event of 10-yr return period storm. This criterion is satisfied in both the pre-development and post-development-controlled conditions. As per WC-13, the relief flow depth cannot exceed 0.30m over the road sag in the event to the Regional flood. This is not applicable, as the entire Mill Street right-of-way is within the Regional Floodplain as pre-determined in Section 7. Therefore, the increased peak flows will not adversely impact the hydraulic condition of the crossing.

10 Quality Matrix

As determined in Section 8.2, the development is expected to increase imperviousness of the site by approximately 16%. These surfaces include rooftops, driveways, gravel areas, and the roadway. It is common practice that runoff from rooftops is considered clean water and does not require quality treatment if they are directly connected to pervious surfaces.

To meet the overall quality control objective, the use of Low Impact Development (LIDs) treatment train approach will be implemented. A treatment train typically requires up to two different LID devices in the same flow path. This is practical way of minimizing sediment accumulation in one isolated location (i.e. end of pipe facility).

Vegetated filter strips and enhanced grass swale design considerations are referenced from Credit Valley Conservation and Toronto Region Conservation's 2010 *Low Impact Development Stormwater Management Planning and Design Guide.* Further design considerations are discussed below.

10.1 Vegetated Filter Strips

Vegetated filter strips are gently sloping vegetated areas that treat runoff from adjacent impervious areas in the form of sheet flow. Typically grassed, these devices transition runoff from one surface texture to another by reducing velocity and filtering out suspended sediment. A conservative runoff reduction for these devices is estimated to be 40% for HSG A and B soils.

Design Considerations:
The following design considerations are provided in the Low Impact Development SWM Planning and Design Guide (CVC and TRCA, 2010) for these devices.

Table 10-1: Low Impact Development Design Guidance

Design Element	Unit	Notes:
Minimum Space	5m	Minimum flow path across strip
Slope	1% to 5%	Match driveway slope
Groundwater Separation	1m	Below ground surface
Soil Type	Any type	Preferred HSG A & B
Max Flow length	25m	Maximum recommended length
Performance Rate:	50%	For soils under HSG A & B

10.2 Enhanced Grassed Swale

Enhanced grass swales are vegetated channels designed to convey, treat, and reduce stormwater runoff. Simple grass channels or ditches have long been used for stormwater conveyance, particularly for roadway drainage. Enhanced grass swales incorporate design features, such as check dams, to improve containment removal and runoff reduction functions.

Design Considerations

The most favorable design parameters for these swales are recommended using a trapezoidal channel with a bottom width of 0.75m to 3.0m, a longitudinal slope between 0.5% and 4%, with 3H:1V side slopes. The swales should be designed with check dams to maintain a velocity of 0.5m/s with a flow depth of 100mm in the event of the 4hr, 25mm Chicago storm.

Table 10-2: Enhanced Grassed Swale Design Guidance

Design Element	Unit	Notes:
Minimum Space	2m	Consume about 5 to 15% of drainage area
Slope	0.5% to 6%	Slopes greater than 3%, use check dams
Groundwater Separation	1m	Below ground surface
Soil Type	Any type	
Maximum drainage area	2ha	Ratio of impervious area to swale area 5:1 to 10:1
Performance Rate:	76%	Based on favorable design parameters

The typical road cross-section provided in Appendix D illustrates the proposed ditch structure. The ditches are proposed to have a maximum depth of 0.93m with a 0.5m bottom width and 2.5H:1V side slopes. They will act as major overland flow route and minor flow conveyance to each SWMF. Since the configuration of these ditches do not meet the favorable design parameters a reduced removal efficiency of 50% will be assigned for these swales.

10.3 Design Matrix

The design matrix below describes the proposed LIDs for the development.

Table 10-3: Proposed Treatment Train Removal Efficiencies

Treatment Number	Treatment Type	SWM Device	Assumed TSS Removal Rate	Target Surface for Treatment
1)	Pre-treatment	Vegetated Filter Strip	40%	Driveways
2)	Pre-treatment	Enhanced Grass Swale	50%	Roadway
3)	End-of-system	Dry Pond & Wet Pond	60-80%	Road & Driveways

Catchment Areas 201, 300A, and 300B include the areas requiring quality treatment. The matrix for each of these areas is explain as follows:

Catchment 201
Driveways will receive quality treatment from all three devices. The roadway will receive pre-treatment from the grassed swales and ending treatment from the dry pond. With combined treatment, the last treatment efficiency is conservatively reduced to account for water that is cleaned in the pre-treatment device. Therefore, the dry pond will have a reduced theoretical efficiency of 30% for runoff from the roadway. Simply adding this adjusted removal efficiency to the 50% efficiency from pre-treatment device, the overall expected TSS removal rate will reach 80%.

Catchment 300A & 300B
Similar to the quality matrix discussed for Catchment 201, these catchments are anticipated to reach a greater removal efficiency rate with the proposed wet pond. As discussed in Section 9.2, the wet pond has been designed to reach a Level 1 treatment removal efficiency. With the combined treatment, the adjusted theoretical efficiency of the Wet Pond will be 24% with the overall removal efficiency of the development expected to reach 84% overall.

11 Conveyance

The major advantage of PCSWMM is its capability to import geodetic elements straight from AutoCAD/Civil 3D allowing the user to easily build a representative drainage model. The post-controlled model was developed to incorporate the drainage swales and major culvert crossings shown on the Post-development Catchment Area drawing in Appendix J.

To assess the major flow routes the Regional Storm was simulated in the Post-Control Model as described in Section 9. The peak flows to the SWMFs are shown below. These peak flows will be used to provide a quick comparison of the capacities determined for each conveyance structure.

SWMF #1: Regional Inflow = 0.534cms (Catchment 201)
SWMF #2: Regional Inflow = 1.15cms (Catchment 300A and 300B)

Roadway Ditches

The roadway and ditches will act both as the major and minor flow routes. The roadside ditches illustrated on the typical cross section (Appendix D) were tested using Manning's Open Channel Flow equation to determine the full flow capacity with a minimum longitudinal slope of 0.50%. With a depth of 0.93m and 0.5m bottom width the roadside ditch will have a full flow capacity of 3.24m^3/s. The open channel flow calculation sheet is provided in Appendix L.

Conveyance Swale

The conveyance swale from Catchment Area 300B to 300A will be constructed within the 6m storm easement. This swale will have a longitudinal slope of 3.5% from Street 'A' to Street 'B'. The depth of the swale will be 0.60m with a 0.50m bottom width and 3H:1V side slopes. Check flow dams will be required and determined at detailed design. Assuming the check flow dams will have a height of 0.15m the full flow capacity of the swale (channel depth set to 0.45m) will be 1.63m^3/s.

Roadway and Entrance Culverts

The largest culvert size that can be accommodated in the typical road cross section will be a 600mm CSP with 300mm of cover from the edge of the travel lane (excluding road crossing of Street 'A' at Ganaraska Road), assuming the culverts function under inlet control. Design Chart 2.32 from the MTO Drainage Management Manual was used to understand the inlet capacity of this culvert size. A headwater to depth ratio of 1.5m was selected as shown on the Nomograph provided in Appendix M. Under these conditions a 600mm culvert has an inlet capacity of 0.50m^3/s before overtopping the roadway. This capacity is just under the 100-yr inlet peak flow to SWMF#2 and should result in minimum road overtopping in the event of the Regional Storm.

Culvert crossings within the development will be required to meet the Ministry of Transportation (MTO) 2008 Highway Drainage Design Standards. Further review at each culvert crossing will be required at time of detailed design to ensure that each crossing satisfies these standards.

12 Sediment and Erosion Control

Erosion and Sediment control measures are required at the time of construction to eliminate sediments leaving the site. This is an import process as runoff over undisturbed areas can adversely impact downstream infrastructure and the natural environment. Based on the anticipated level of work, a detailed sediment and erosion control plan will be required at the time of detailed design.

13 Maintenance

Maintenance procedures are necessary to ensure long term care of both SWMFs are provided. A 3.0m maintenance path has been included in the conceptual designs to facilitate these procedures. Further details will be provided at detailed design.

14 Conclusion

Monument Geomatics and Estimating ("MG" or "Monument") was retained by Mistral Land Development to prepare a preliminary Servicing and Stormwater Management Report in support of the draft plan submission for a rural development in Garden Hill, Ontario, located in the Municipality of Port Hope. The development will consist of two rural roadways to allow frontage for 43 single family homes sitting on 3/4 acre lots, and a single residential block with frontage on Ganaraska Road and Frost Ave.

The report provides a high-level review of the following items:

1) **Draft Plan** layout with two separate road access points from Ganaraska Roadway and Porter Crescent creating 43 single family lots and one large residential block.

2) **Water Supply and Sewage Disposal** setbacks to support private servicing of each lot.

3) **Erosion Hazard Limits** of the confined and unconfined tributaries located on the property.

4) **Floodplain Analysis and Delineation** to delineate the 100-yr and Regional flood elevations of each tributary.

    A floodplain analysis was conducted for the small Tributary to North Ganaraska River that intersects the southeast corner of the development lands. The tributary outlets the property through a 900mm CSP culvert in Ganaraska Road (CTY Rd 9). The watershed boundaries were delineated in 4 different catchment areas to each culvert crossing upstream to the confluence of North Ganaraska River. A hydrologic model in PCSWMM was created to determine pre- and post-development peak flows at each crossing. A conservative increase of 20% impervious area was applied for the development lands to determine the post-development peak flows.

    A HEC-RAS model was built from the Ganaraska River up to the eastern boundary of the development lands to determine water surface elevations (WSELs) both upstream and downstream of Ganaraska Road. The results revealed that in both the pre- and post-development conditions the event of the Regional Storm, large amounts of flooding would occur in the banks of the development lands. The Regional event would also overtop the roadway.

    Monument proposed that the culvert crossing in Ganaraska Road be upsized to a larger 1400mm circular culvert or equivalent to reduce the storage being created upstream of the crossing. As illustrated on the North Ganaraska River Floodplain Mapping provided in Appendix H. The original Floodplain Study did not account for this additional storage upstream of the

Crossing. Therefore, upsizing the culvert would not impact original flood elevations in the North Ganaraska River.

The HEC-RAS model was simulated with the new replacement option to improve the hydraulic efficiency of the crossing and reduced upstream WSELs. The results from this option drastically reduced the ponding and returned the Regional flood elevations back within the banks of the watercourse. A Proposed Floodline drawing (FLD-3) is provided in Appendix H illustrating the flooding boundaries both upstream and downstream of the crossing.

5) **Stormwater Management** to meet the design objectives following Ganaraska Region Conservation Authorities (GRCAs) and Ministry of Environment, Conservation and Parks stormwater management (SWM) guidelines.

After pre-consultation with the Conservation Authority, the following SWM objectives were established for the development:

1) **Quantity Control** - The objective is to ensure that post-development peak flows meet the pre-development levels for all minor and major storm events up to the 100-yr return period. The Regional Storm (Hurricane Hazel) will also be assessed to ensure that all flows are safely conveyed through the development in appropriately sized major overland flow routes.

2) **Quality Control** – Quality controls were provided using Best Management Practices (BMP's) to meet a level 1 total suspended solid (TSS) removal efficiency as an Enhanced level of protection (80% long-term suspended solids removal), in accordance with the MECP's Table 3.2 in the 2003 SWM guideline.

3) **Sediment and Erosion Measure** - Prepare a sediment and erosion plan to control and mitigate release of sediment throughout the construction stage. Further details would be provided at detailed design.

Two (2) stormwater management facilities are proposed for the development due the undulating topography of the land separating the site into two drainage boundaries. The first facility (SWMF#1) will be adjacent to the Ganaraska Road and outlet into the South Tributary (Outfall#1). This facility is proposed to be a continuously flowing dry pond. A conceptual design is provided on the Post-development Catchment Area drawing and discussed in Section 9.1. This facility has been sufficiently sized within the dedicated SWM block to meet the quantity control objective while following the design considerations provided in GRCA's SWM Guidelines. The facility will also be accredited with a long-term TSS removal efficiency of 60% as per the MECP's 2003 SWM guidelines.

SWMF#2 is located adjacent to Mill Street near the existing farm entrance of the property. This facility has been proposed as an extended detention wet pond. This was selected after GRCA identified that the downstream receiver was a cold-water system. The permanent pool in the facility

will allow the water to be extracted from the bottom of the pond therefore discharging cold water downstream.
The conceptual design of the facility has been sufficiently sized to attenuate all flows to their pre-development levels. The facility will also be designed to meet the extended detention volumes and drawdown time to provide a total long term TSS removal efficiency of 80% for the contributing area.

The quality control objective for the site has been designed to meet the targeted long term TSS removal efficiency of 80% as per GRCA's SWM Guidelines. A quality matrix has been proposed using best management practices by accrediting low impact development devices (LIDs). The matrix is proposed to follow the treatment train approach. For the development, the hardened areas such as driveways, roadways and gravel areas will require treatment before ultimately discharging downstream. Monument has proposed that pre-treatment for these surface areas be provided using vegetated filter strips and enhanced grassed swales with the ending of system treatment as the applicable Stormwater Management Facilities. Further explanation is provided in Section 10.

Therefore, the conceptual design provided in this report have been prepared to meet each design objective and satisfy servicing requirements to support the proposed draft plan.

Prepared by:

Patrick Quinn, E.I.T.

Reviewed by:

LICENSED PROFESSIONAL ENGINEER
C. J. ORAM
100140005
2022-04-14
PROVINCE OF ONTARIO

Cody Oram, P.Eng
Senior Project Manager

15 References

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Ministry of Transportation (2008). Highway *Drainage Design Standards*

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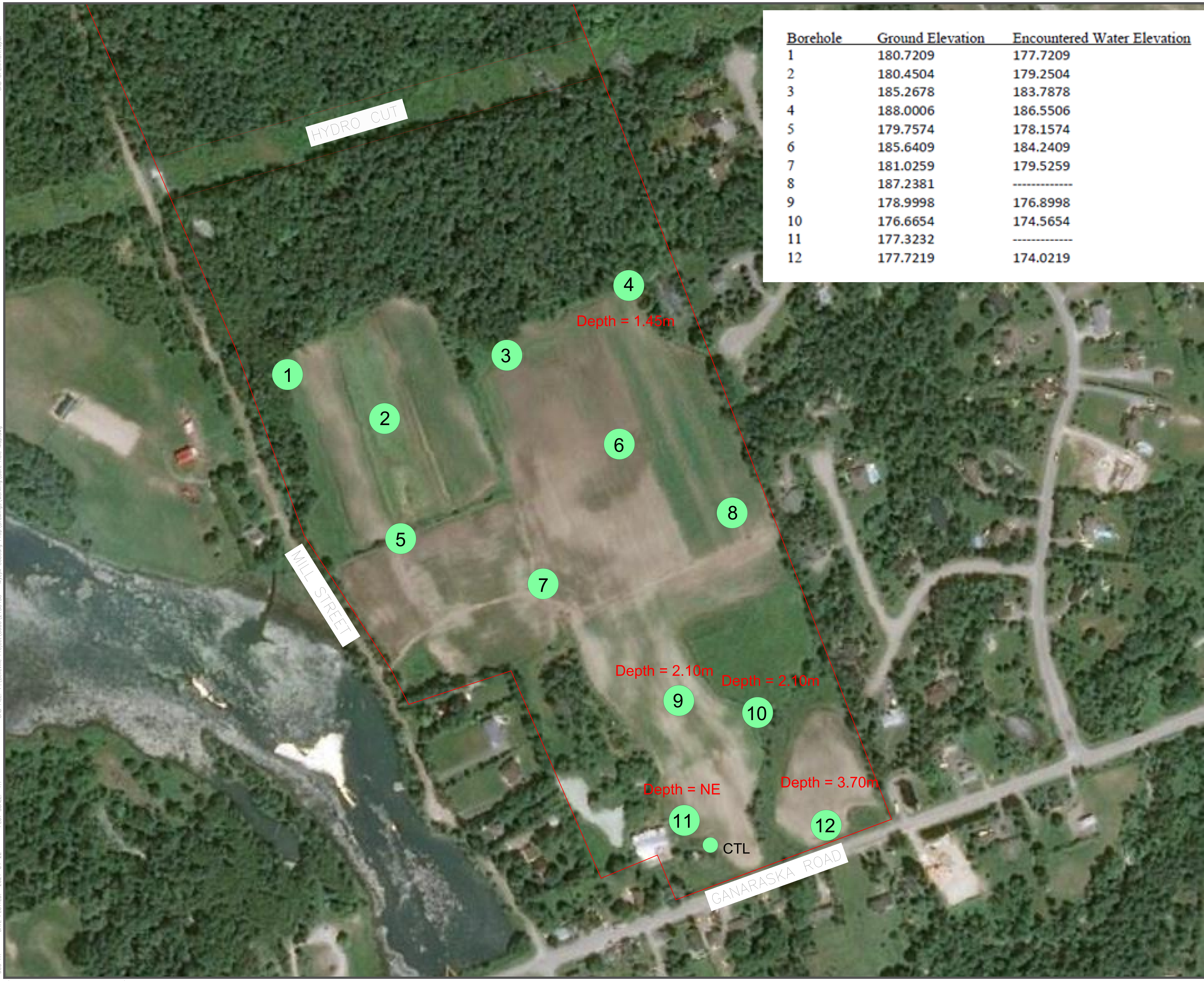
Ontario.ca (2012) *The Ontario Building Code*

Toronto and Region Conservation Authority, Credit Valley Conservation Authority (2010). *Low Impact Development Stormwater Management Planning and Design Guide.*

APPENDIX A – BOREHOLE LOG EXCERPT



Borehole	Ground Elevation	Encountered Water Elevation
1	180.7209	177.7209
2	180.4504	179.2504
3	185.2678	183.7878
4	188.0006	186.5506
5	179.7574	178.1574
6	185.6409	184.2409
7	181.0259	179.5259
8	187.2381	-------------
9	178.9998	176.8998
10	176.6654	174.5654
11	177.3232	-------------
12	177.7219	174.0219

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FAX: 613-966-3087

NOTES:

1. ALL WORK SHALL BE IN ACCORDANCE WITH RELEVANT CODES AND GUIDELINES.
2. ALL DRAWINGS AND ADDENDA ARE TO BE READ AS, AND IN CONJUNCTION WITH THE SPECIFICATIONS.
3. ALL EQUIPMENT SHALL BE INSTALLED AS SPECIFIED OR APPROVED EQUIVALENT.
4. CONTRACTOR MUST CHECK AND VERIFY ALL DIMENSIONS BEFORE PROCEEDING WITH WORK AND BE RESPONSIBLE FOR SAME.
5. CONTRACTOR MUST REPORT ANY DISCREPANCIES TO ENGINEER FOR RESOLUTION BEFORE COMMENCING THE WORK.
6. ANY CHANGES MUST BE APPROVED BY THE ENGINEER.

A/B — A DETAIL NO.; B DRAWING NO. – WHERE DETAILED

LEGEND

NORTH

STAMP

PROJECT
GARDEN HILL SUBDIVISION
GARDEN HILL, ONTARIO

DRAWING TITLE
SITE PLAN
BORE HOLE LOCTAIONS

REVISION		DATE
05		
04		
03		
02		
01		

DESIGNED BY

DRAWN BY TL

REVIEWED BY

APPROVED BY

PROJECT DATE 06/05/2021 (DD/MM/YYYY)

PROJECT # 21-3-8438

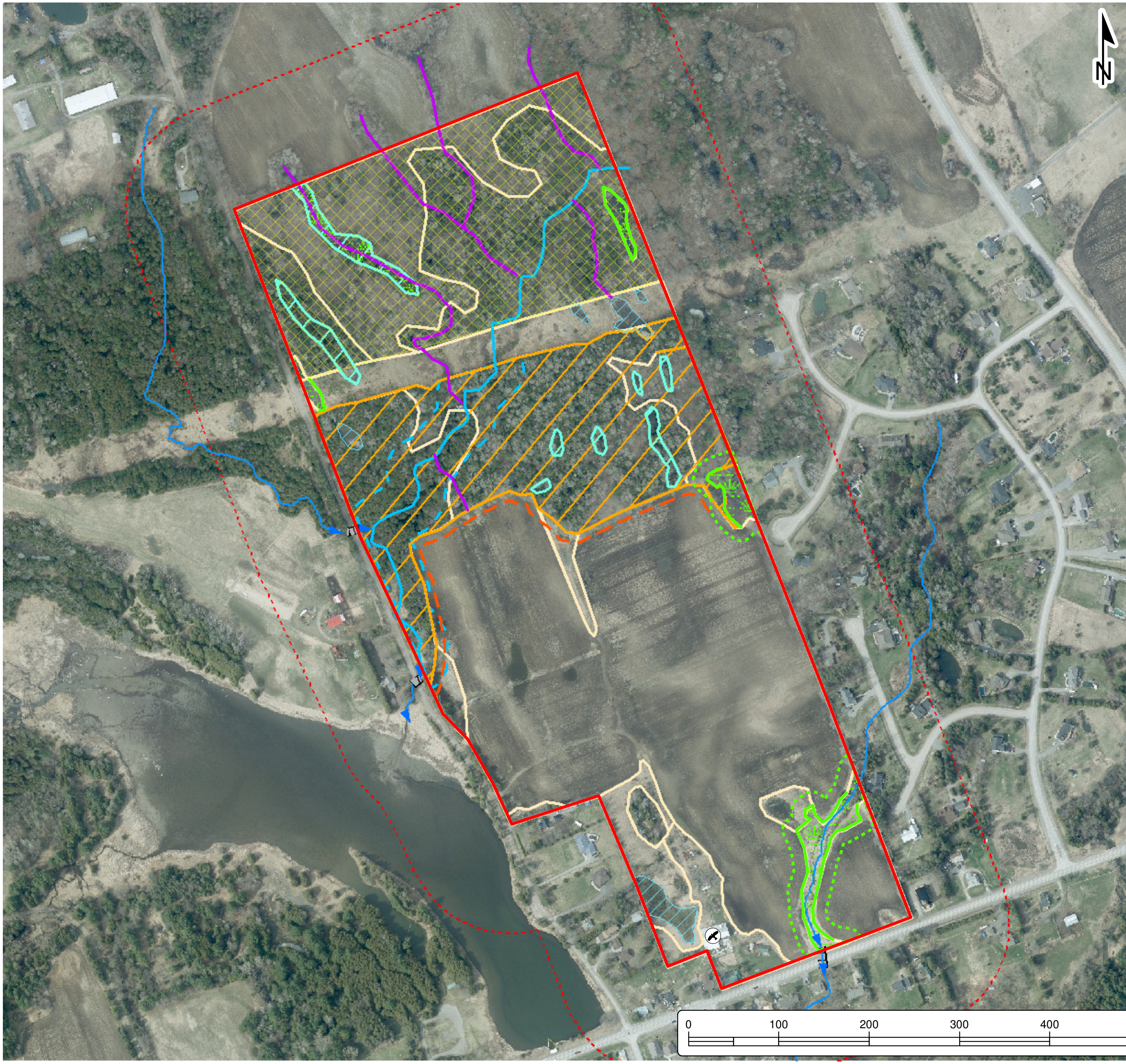
SCALE HOR: 1:200 VER: N/A

DRAWING # SP-1

GGG-A1 DATE PLOTTED: 2021-04-22 PLOT SCALE: 1:? CAD FILE: P:\Belleville Project\8000\2138438 - Mycyk Cobourg Prop\Drawings\Working\Bore Hole Map.dwg

(METRIC SCALE – ALL DIMS IN MILLIMETERS)

APPENDIX B – CONSTRAINTS MAP



ENVIRONMENTAL IMPACT STUDY

CHRISTOPHER DONEGAN
3852 Ganaraska Road
Port Hope, Ontario

LEGEND

- 120 m Adjacent Lands
- Site (37 ha)
- 15 m Wetland Setback
- Wetland Boundary (Delineated with GRCA)
- Wetland
- Vegetation Communities
- 10 m Woodland Dripline Setback
- Significant Woodlands (within Settlement Area) (7.7 ha)
- Significant Woodlands (Outside Settlement Area)
- Field Verified Watercourse
- 30 m Watercourse Setback
- Culvert
- Drainage Feature
- Pond
- Vernal Pool
- Watercourse, Permanent (Ontario Hydro Network Mapping)
- Barn Swallow Nesting Habitat

Notes:
- Base mapping features are © Queen's Printer of Ontario, 2019 (this does not constitute an endorsement by the Ministry of Natural Resources and Forestry or the Ontario Government).
- Distances on this plan are in metres and can be converted to feet by dividing by 0.3048.
- Cambium Inc. makes every effort to ensure this map is free from errors but cannot be held responsible for any damages due to error or omissions. This map should not be used for navigation or legal purposes. It is intended for general reference use only.

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NATURAL HERITAGE CONSTRAINTS - DRAFT

Project No.: 12728-001	Date: September 2021 Rev.:
ale: 1:6,000	Projection: NAD 1983 UTM Zone 17N
eated by: DJL	Checked by: JPP
Figure: 3	

O:\GIS\MXDs\12700-12799\12728-001 Christopher Donegan - EIS - 3852 Ganaraska Rd, Port Hope\2021-09-14 FIG 3 - Natural Heritage Constraints - Draft.mxd

APPENDIX C – DRAFT PLAN

LAND USE SCHEDULE

LAND USE	AREA	SITE (%)
LOTS 1–43 – 3036m² MIN. RURAL RESIDENTIAL	15.95ha	43.6%
BLOCK 100, 101 – SWM POND	0.72ha	2.0%
BLOCK 102 – DRAINAGE EASEMENT	0.11ha	0.3%
BLOCK 103 – WETLAND COMPENSATION/WATERCOURSE	1.82ha	5.0%
BLOCK 104 – 10–21 UNIT CONDOMINIUM/APARTMENT BUILDING	0.67ha	1.8%
BLOCK 105, 107 – REGULATED LANDS	12.90ha	35.3%
BLOCK 106 – HYDRO EASEMENT	1.87ha	5.0%
20.0m MUNICIPAL ROAD ALLOWANCE – STREET A, B, C, COUNTY ROAD 9 WIDENING	2.56ha	7.0%
SITE TOTAL	36.60ha	100%

DRAFT PLAN OF SUBDIVISION

GARDEN HILL ESTATES

PLAN 39R-14329 - PART OF LOT 16, CONCESSION 8
MUNICIPALITY OF PORT HOPE
COUNTY OF NORTHUMBERLAND

SCALE 1:1250

0 30 60 Meters

NO.	DATE (D/M/Y)	REVISION	BY
1.	18/04/22	FIRST SUBMISSION	C.O.

ADDITIONAL INFORMATION REQUIRED – SECTION 51(17), PLANNING ACT

a) AS SHOWN ON THE DRAFT PLAN
b) AS SHOWN ON THE DRAFT PLAN
c) AS SHOWN ON THE DRAFT PLAN
d) SEE LAND USE SCHEDULE
e) AS SHOWN ON THE DRAFT PLAN
f) AS SHOWN ON THE DRAFT PLAN
g) AS SHOWN ON THE DRAFT PLAN
h) WELL AND SEPTIC
i) CLAY SILT TO SILTY SAND
j) AS SHOWN ON THE DRAFT PLAN
k) FULL MUNICIPAL SERVICES
l) AS SHOWN ON THE DRAFT PLAN

OWNER'S CERTIFICATE

I AUTHORIZE MONUMENT GEOMATICS TO PREPARE AND SUBMIT THIS DRAFT PLAN FOR APPROVAL.

OWNER

DATE

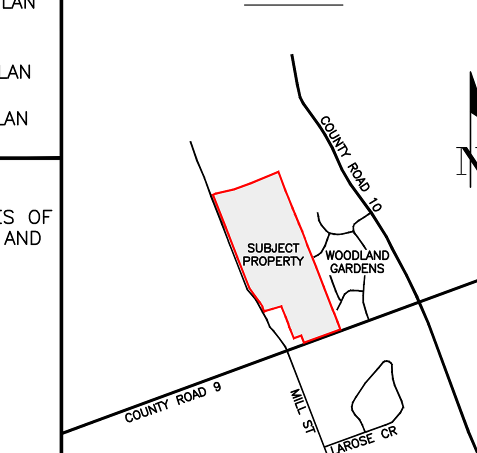
SURVEYOR'S CERTIFICATE

I CERTIFY THAT THE BOUNDARIES OF THE LAND TO BE SUBDIVIDED AND CORRECTLY SHOWN.

ONTARIO LAND SURVEYOR

DATE

KEY PLAN



PROJECT No.: 21–0135
DRAWN: T.M.
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DATE: 18/04/22

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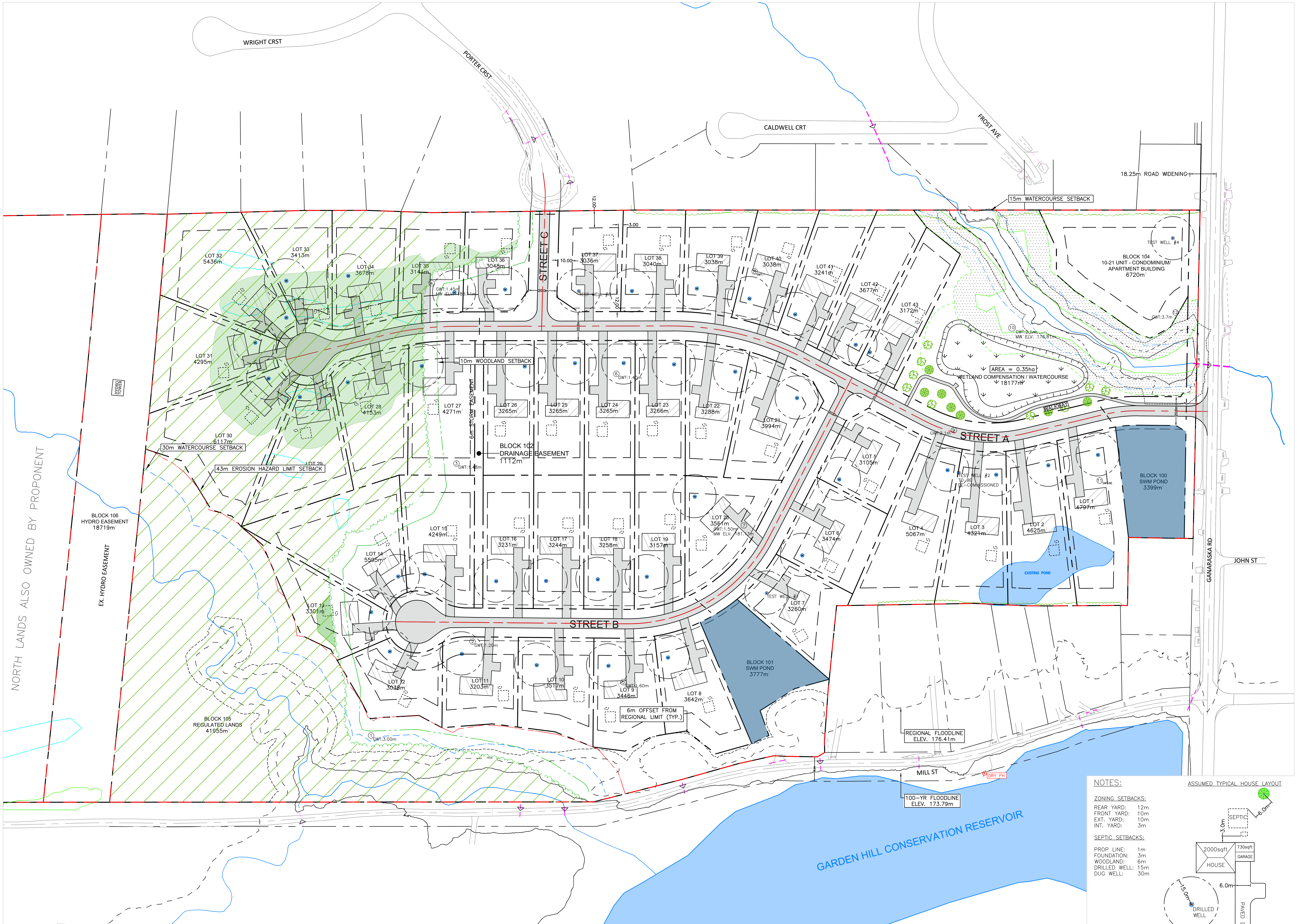
APPENDIX D – CONCEPTUAL SERVICING PLAN AND TYPICAL CROSS SECTION



NOTES:

ZONING SETBACKS:

REAR YARD:	12m
FRONT YARD:	10m
EXT. YARD:	10m
INT. YARD:	3m

SEPTIC SETBACKS:

PROP LINE:	1m
FOUNDATION:	3m
WOODLAND:	6m
DRILLED WELL:	15m
DUG WELL:	30m

LEGEND

- PROPERTY BOUNDARY
- SIGNIFICANT WOODLAND AREA
- WOODLAND REMOVAL
- SWM BLOCK
- WETLAND COMPENSATION
- WETLAND VEGETATION PLANTING AREAS
- EXISTING WETLAND TO BE REMOVED
- WATERCOURSE
- WATERCOURSE SETBACK
- 100-YR FLOOD LIMIT
- REGIONAL FLOOD LIMIT
- BUILDING ENVELOPE
- 6m SETBACK FROM REGIONAL FLOOD LIMIT
- WETLAND BOUNDARY
- 15m WETLAND SETBACK
- 10m WOODLAND SETBACK
- EROSION HAZARD LIMIT
- EXISTING TEST WELL
- PROPOSED WELL
- VERNAL POOLS

NO.	DATE (D/M/Y)	REVISION	BY
-	-	-	-

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SCALES:	HORZ. 1:1250
SCALES:	VERT.

MISTRAL LAND DEVELOPMENTS

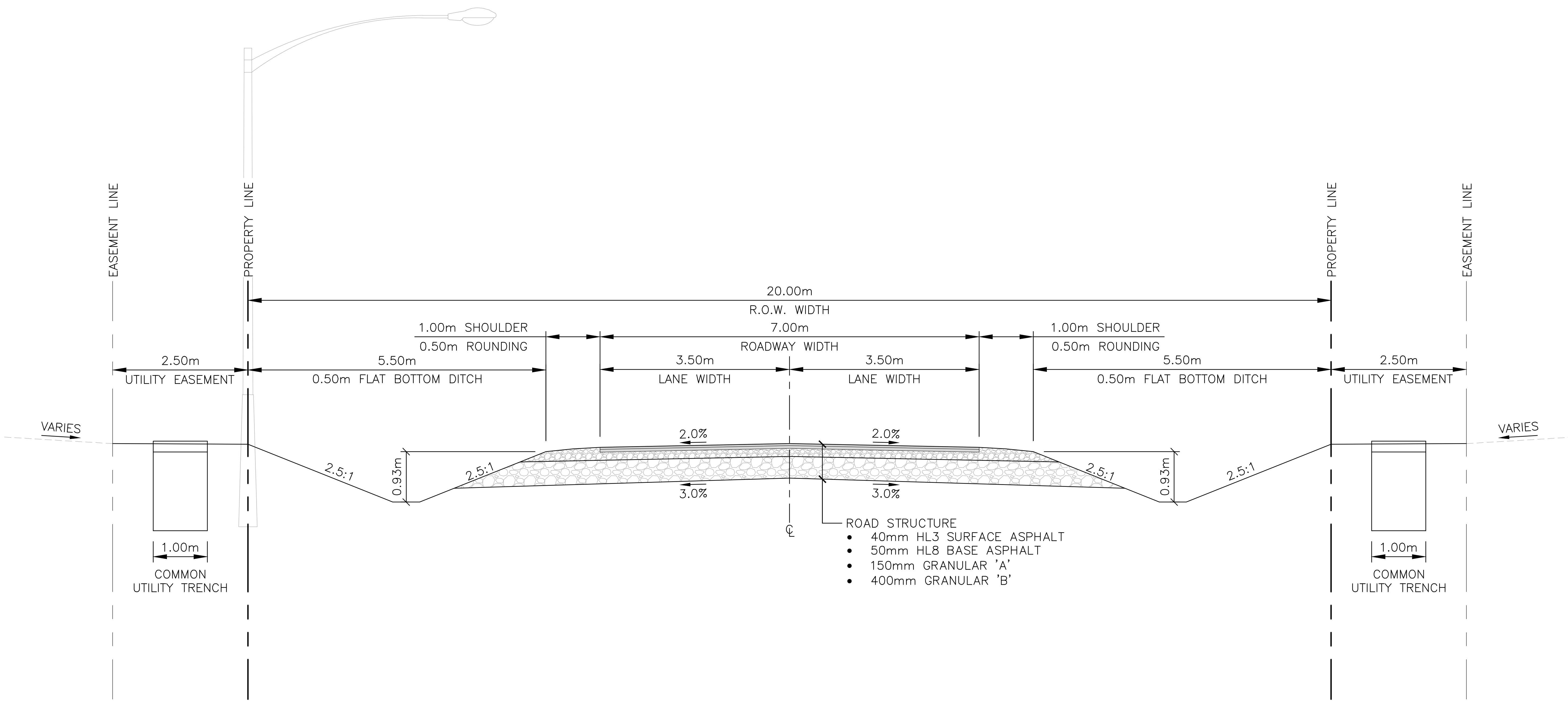
GARDEN HILL ESTATES
MUNICIPALITY OF PORT HOPE

GENERAL SERVICING PLAN

DRAWING No.
GS-01

0 30 60 Meters

 GENERAL NOTES: ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK.ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. ALL DIMENSIONS SHOWN ARE IN METERS OR MILLIMETERS, UNLESS OTHERWISE NOTED.



TITLE.

GARDEN HILL ESTATES

TYPICAL ROAD CROSS SECTION

COUNTY OF NORTHUMBERLAND

PROJECT No.:	21–0135
DRAWN:	T.M.
CHECKED:	P.Q.
DATE:	01/04/22
SCALE:	1:75
FIGURE No.	XX

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APPENDIX E – PRECIPITATION DATA

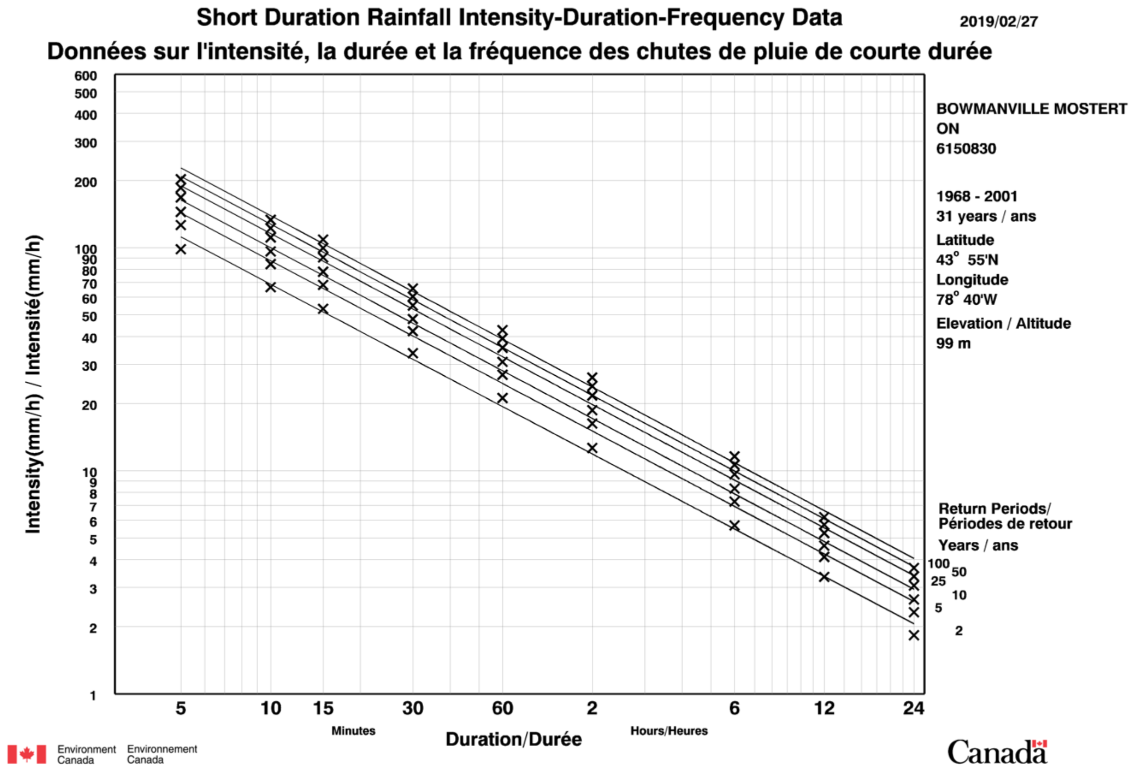


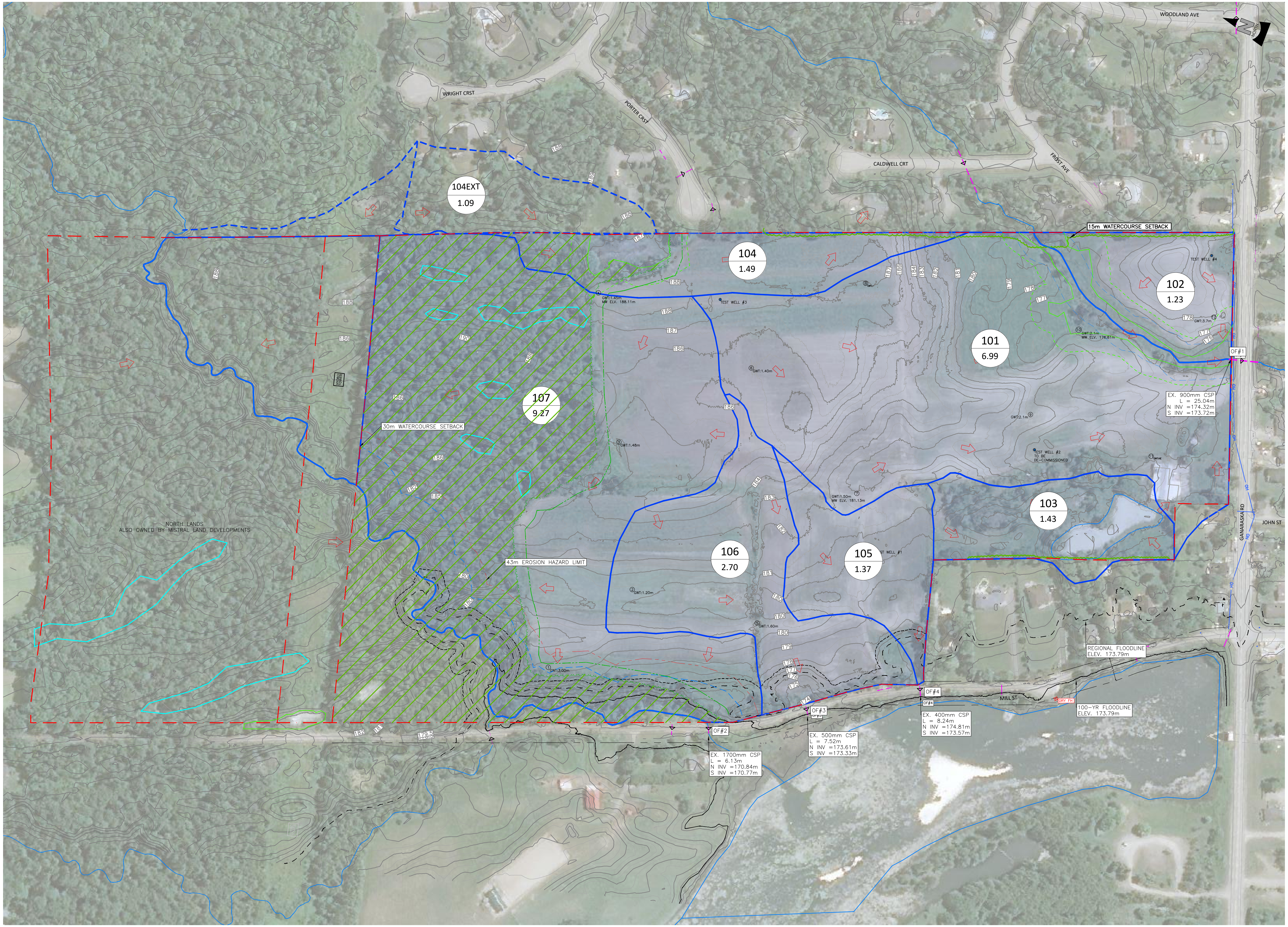
Table 2a : Return Period Rainfall Amounts (mm)
Quantité de pluie (mm) par période de retour

Duration/Durée	2 yr/ans	5 yr/ans	10 yr/ans	25 yr/ans	50 yr/ans	100 yr/ans	#Years Années
5 min	8.2	10.5	12.1	14.0	15.5	16.9	32
10 min	11.1	14.1	16.1	18.6	20.4	22.2	32
15 min	13.3	17.0	19.5	22.6	24.9	27.2	32
30 min	16.9	21.1	24.0	27.6	30.2	32.9	32
1 h	21.2	27.0	30.8	35.6	39.2	42.8	32
2 h	25.3	32.6	37.4	43.5	48.0	52.5	32
6 h	34.2	43.7	50.0	57.9	63.8	69.7	31
12 h	40.2	49.4	55.5	63.2	68.9	74.5	31
24 h	44.0	55.8	63.6	73.5	80.8	88.0	32

Design Chart 1.03: Hurricane Hazel

	Depth		Percent of 12 hour
	(mm)	(inches)	
First 36 hours	73	2.90	
37th hour	6	.25	3
38th hour	4	.17	2
39th hour	6	.25	3
40th hour	13	.50	6
41st hour	17	.66	8
42nd hour	13	.50	6
43rd hour	23	.91	11
44th hour	13	.50	6
45th hour	13	.50	6
46th hour	53	2.08	25
47th hour	38	1.49	18
48th hour	13	.50	6
	285	11.21	100

APPENDIX F – PRE-DEVELOPMENT CATCHMENT AREA DRAWING



LEGEND

- PROPERTY BOUNDARY
- SIGNIFICANT WOODLAND AREA
- WATERCOURSE
- WATERCOURSE SETBACK
- 100-YR FLOOD LIMIT
- REGIONAL FLOOD LIMIT
- 6m SETBACK FROM REGIONAL FLOOD LIMIT
- WETLAND BOUNDARY
- 15m WETLAND SETBACK
- 10m WOODLAND SETBACK
- EROSION HAZARD LIMIT
- VERNAL POOLS
- 101 — CATCHMENT ID; 2.09 — AREA (ha)
- CATCHMENT BOUNDARY
- EXTERNAL CATCHMENT
- OVERLAND FLOW ARROW
- BOREHOLE
- GWT:3.7m — DEPTH TO WATER TABLE

NOTES:
CONTOUR DATA WAS DERIVED FROM MONUMENT GPS SURVEY AND LIDAR SURVEY SUPPLEMENTED WITH DERIVED CONTOURS FROM ONTARIO'S SURFACE ELEVATION MODEL.

NO.	DATE (D/M/Y)	REVISION	BY
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DATE:	30/09/2021
SCALES:	HORZ. 1:1500
SCALES:	VERT.

MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES

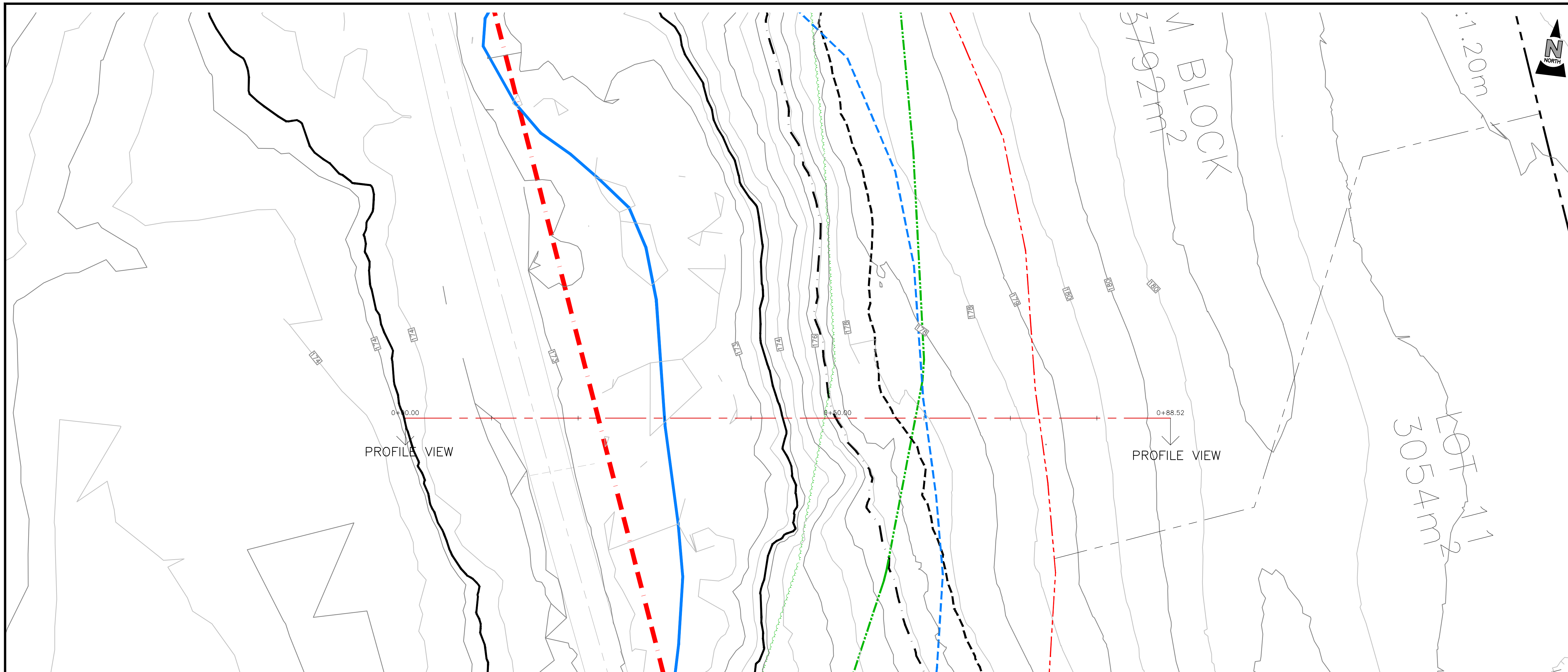
MUNICIPALITY OF PORT HOPE

PRE-CATCHMENT AREA DRAWING

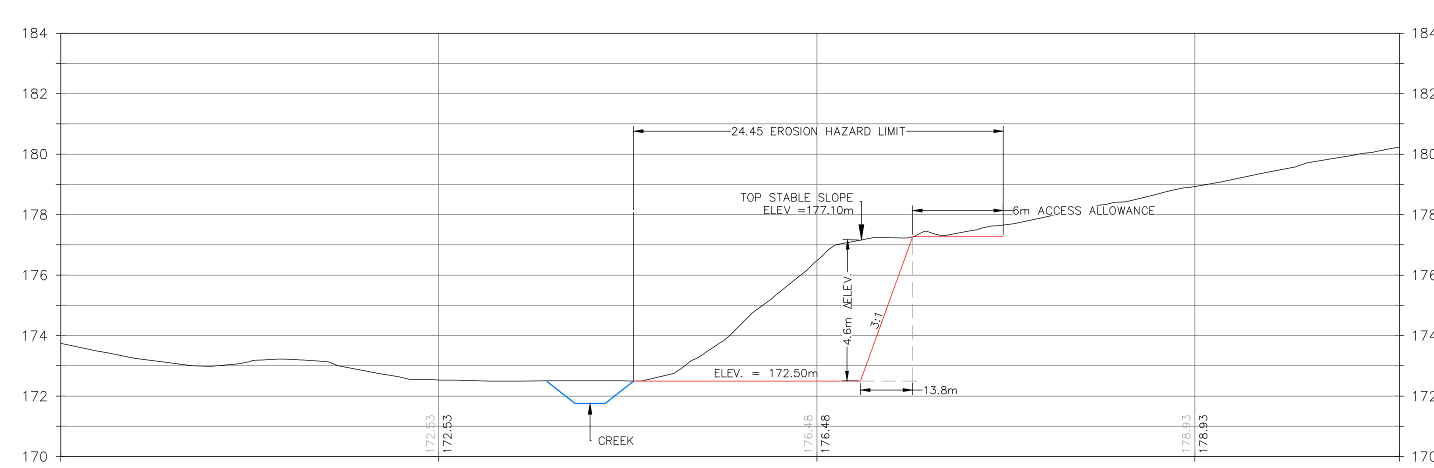
DRAWING No.
SWM-01

 GENERAL NOTES: ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK.ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. ALL DIMENSIONS SHOWN ARE IN METERS OR MILLIMETERS, UNLESS OTHERWISE NOTED.

APPENDIX G – EROSION HAZARD LIMIT



PROFILE - ALN - HazardLimit1



LEGEND

PROPERTY BOUNDARY
SIGNIFICANT WOODLAND AREA
WOODLAND REMOVAL
SWM BLOCK
WETLAND COMPENSATION
WETLAND VEGETATION PLANTING AREAS
EXISTING WETLAND TO BE REMOVED
WATERCOURSE
WATERCOURSE SETBACK
100-YR FLOOD LIMIT
REGIONAL FLOOD LIMIT
6m SETBACK FROM REGIONAL FLOOD LIMIT
WETLAND BOUNDARY
15m WETLAND SETBACK
10m WOODLAND SETBACK
EROSION HAZARD LIMIT
VERNAL POOLS

NOTES:

NO. OF LOTS: 44
TOTAL LOT AREA: 169,326m² (16.93ha)
WOODLAND TO BE REMOVED: 1.50ha
WETLAND AREA TO BE REMOVED: 0.18ha
WETLAND COMPENSATION AREA: 0.35ha

NO.	DATE (D/M/Y)	REVISION	BY
-	-	-	-

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GEOMATICS
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MONUMENT PROJECT No.:	-
DRAWN:	P.Q.
CHECKED:	-
ENGINEER:	-
DATE:	31/01/2022
SCALES:	HORZ. 1:250
SCALES:	VERT. 1:2

MISTRAL LAND DEVELOPMENTS

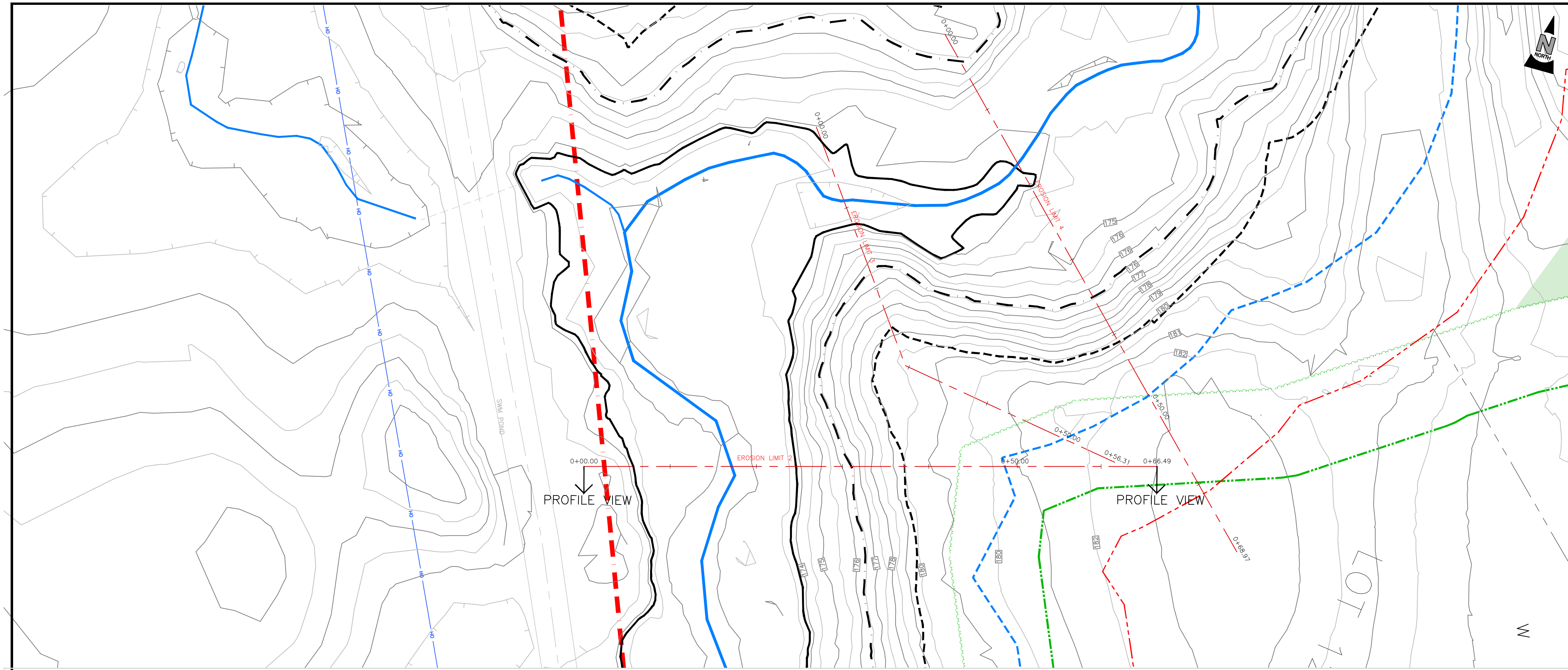
GARDEN HILL ESTATES
EROSION HAZARD LIMIT
LIMIT 1

DRAWING No.
EHL-01

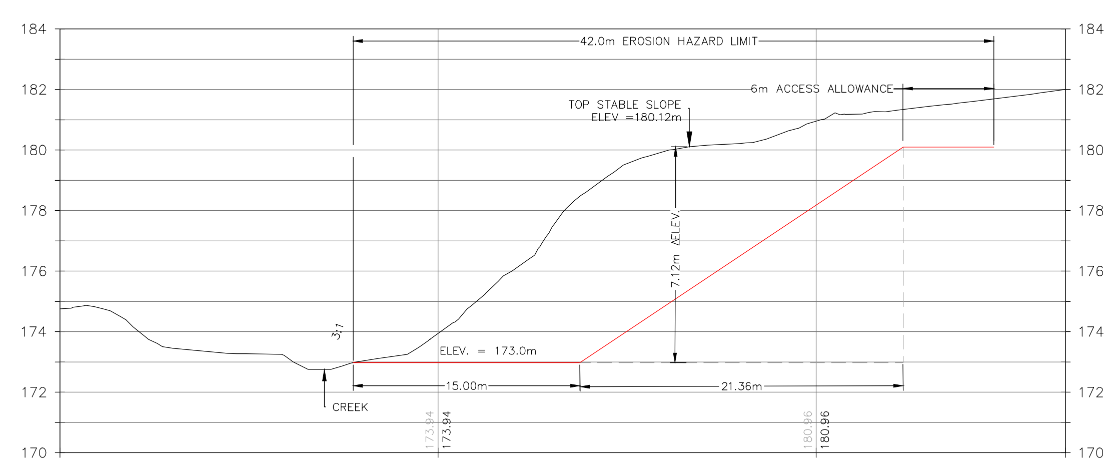
0 7.5 15 Meters

C:\Users\Patrick Quinn\Monument Geomatics\E&D-Garden Hill - General\4-Design\1-C3D\C2022-21-0135BASE Concept 3.dwg

 GENERAL NOTES: ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK.ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. ALL DIMENSIONS SHOWN ARE IN METERS OR MILLIMETERS, UNLESS OTHERWISE NOTED.



PROFILE - ALN - Hazard Limit 2



LEGEND

- PROPERTY BOUNDARY
- SIGNIFICANT WOODLAND AREA
- WOODLAND REMOVAL
- SWM BLOCK
- WETLAND COMPENSATION
- WETLAND VEGETATION PLANTING AREAS
- EXISTING WETLAND TO BE REMOVED
- WATERCOURSE
- WATERCOURSE SETBACK
- 100-YR FLOOD LIMIT
- REGIONAL FLOOD LIMIT
- 6m SETBACK FROM REGIONAL FLOOD LIMIT
- WETLAND BOUNDARY
- 15m WETLAND SETBACK
- 10m WOODLAND SETBACK
- EROSION HAZARD LIMIT
- VERNAL POOLS

NOTES:

NO. OF LOTS: 44

TOTAL LOT AREA: 169,326m² (16.93ha)

WOODLAND TO BE REMOVED: 1.50ha

WETLAND AREA TO BE REMOVED: 0.18ha

WETLAND COMPENSATION AREA: 0.35ha

NO.	DATE (D/M/Y)	REVISION	BY

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MONUMENT PROJECT No.:	-
DRAWN:	P.Q.
CHECKED:	-
ENGINEER:	-
DATE:	31/01/2022
SCALES:	HORZ. 1:250
SCALES:	VERT. 1:2

MISTRAL LAND DEVELOPMENT

GARDEN HILL ESTATES
EROSION HAZARD LIMIT
LIMIT 2

DRAWING No.
EHL-02

0 7.5 15 Meters

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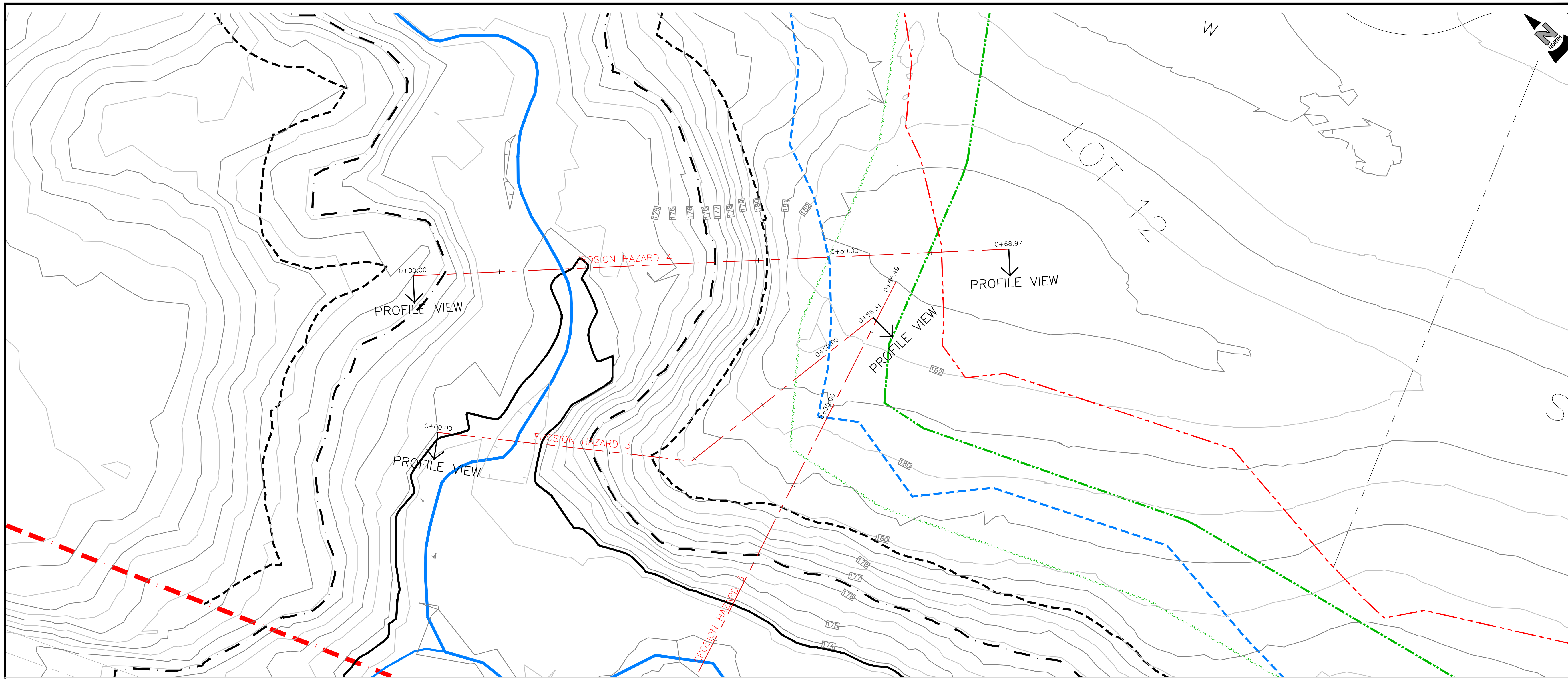

LEGEND

- PROPERTY BOUNDARY
- SIGNIFICANT WOODLAND AREA
- WOODLAND REMOVAL
- SWM BLOCK
- WETLAND COMPENSATION
- WETLAND VEGETATION PLANTING AREAS
- EXISTING WETLAND TO BE REMOVED
- WATERCOURSE
- WATERCOURSE SETBACK
- 100-YR FLOOD LIMIT
- REGIONAL FLOOD LIMIT
- 6m SETBACK FROM REGIONAL FLOOD LIMIT
- WETLAND BOUNDARY
- 15m WETLAND SETBACK
- 10m WOODLAND SETBACK
- EROSION HAZARD LIMIT
- VERNAL POOLS

NOTES:

NO. OF LOTS: 44

TOTAL LOT AREA: 169,326m² (16.93ha)

WOODLAND TO BE REMOVED: 1.50ha

WETLAND AREA TO BE REMOVED: 0.18ha

WETLAND COMPENSATION AREA: 0.35ha

NO.	DATE (D/M/Y)	REVISION	BY
-	-	-	-

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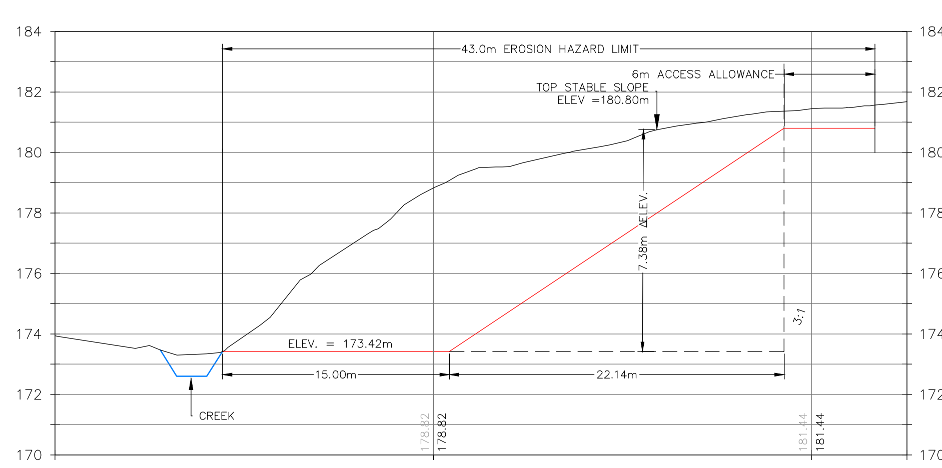
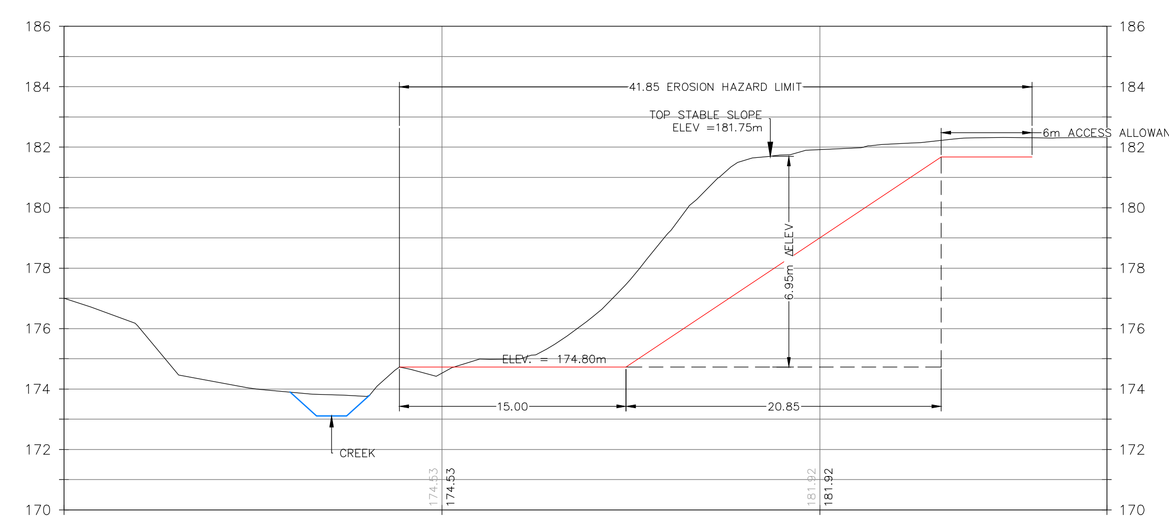
PROFILE - ALN - Hazard Limit 3(1)



PROFILE - ALN - Hazard Limit 4



MONUMENT PROJECT No.:	-
DRAWN:	P.Q.
CHECKED:	-
ENGINEER:	-
DATE:	31/01/2022
SCALES:	HORZ. 1:250
SCALES:	VERT. 1:2

MISTRAL LAND DEVELOPMENT

GARDEN HILL ESTATES
EROSION HAZARD LIMIT
LIMIT 3 AND 4

DRAWING No.
EHL-03

0 7.5 15 Meters

APPENDIX H – FLOODLINE DRAWINGS

1. **GRCA Fill and Flood Plain Mapping of Ganaraska River (TSH 1990) Sheet 94 of 117**
2. **GRCA Fill and Flood Plain Mapping of Ganaraska River (TSH 1990) Sheet 82 of 117**
3. **Flood Study Catchment Area Drawing**
4. **Existing Condition Floodline**
5. **Post-Conditions with New Culvert Replacement**
6. **Post-conditions with New Culvert Replacement and Access**

LEGEND

FLOODLINES CENTRED OVER WATERSHED
REGIONAL STORM
100 YEAR STORM

FILL AND CONSTRUCTION LIMIT

CROSS-SECTION LOCATION
FLOODLINE ELEVATION IN FEET REGIONAL / 100yr. X-SEC. NO.

ADJOINING SHEETS

	106	
93	94	95
	82	

GANARASKA REGION CONSERVATION AUTHORITY
FILL AND FLOOD PLAIN MAPPING
OF THE GANARASKA RIVER

ONTARIO REGULATION NO. ____________ SCHEDULE NO. ____________

TOPOGRAPHIC MAPPING COMPILED BY MARSHALL MACKLIN, MONAGHAN LTD.
FROM 1976 AERIAL PHOTOGRAPHY

...ubicki associates
...CTS AND PLANNERS

SCALE 1:2400
100 0 100 200 300 400 500 600 700
FEET

CONTOUR INTERVAL — 5 FEET, 2 1/2 FEET INTERPOLATION
GEODETIC DATUM — MEAN SEA LEVEL AS ESTABLISHED BY GEODETIC SURVEY OF CANADA.
PROJECTION SYSTEM — 3° TRANSVERSE MERCATOR

DATE JAN. 1990

SHEET 94 of 117

REGISTERED PROFESSIONAL ENGINEER
A. E. MARIN
PROVINCE OF ONTARIO
MARSHALL MACKLIN MONAGHAN LTD.

REGISTERED PROFESSIONAL ENGINEER
L. SANCHEZ
PROVINCE OF ONTARIO
TOTTEN SIMS HUBICKI ASSOCIATES

LEGEND

FLOODLINES CENTRED OVER WATERSHED
REGIONAL STORM
100 YEAR STORM
FILL AND CONSTRUCTION LIMIT
CROSS-SECTION LOCATION
FLOODLINE ELEVATION IN FEET REGIONAL / 100 yr. X-SEC. NO.

ADJOINING SHEETS

	94	
81	82	83
	70	

GANARASKA REGION CONSERVATION AUTHORITY
FILL AND FLOOD PLAIN MAPPING
OF THE GANARASKA RIVER

ONTARIO REGULATION NO. ________ SCHEDULE NO. ________

TOPOGRAPHIC MAPPING COMPILED BY MARSHALL, MACKLIN, MONAGHAN LTD.
FROM 1976 AERIAL PHOTOGRAPHY

totten sims hubicki associates
ENGINEERS ARCHITECTS AND PLANNERS

SCALE 1:2400
100 0 100 200 300 400 500 600 700
FEET

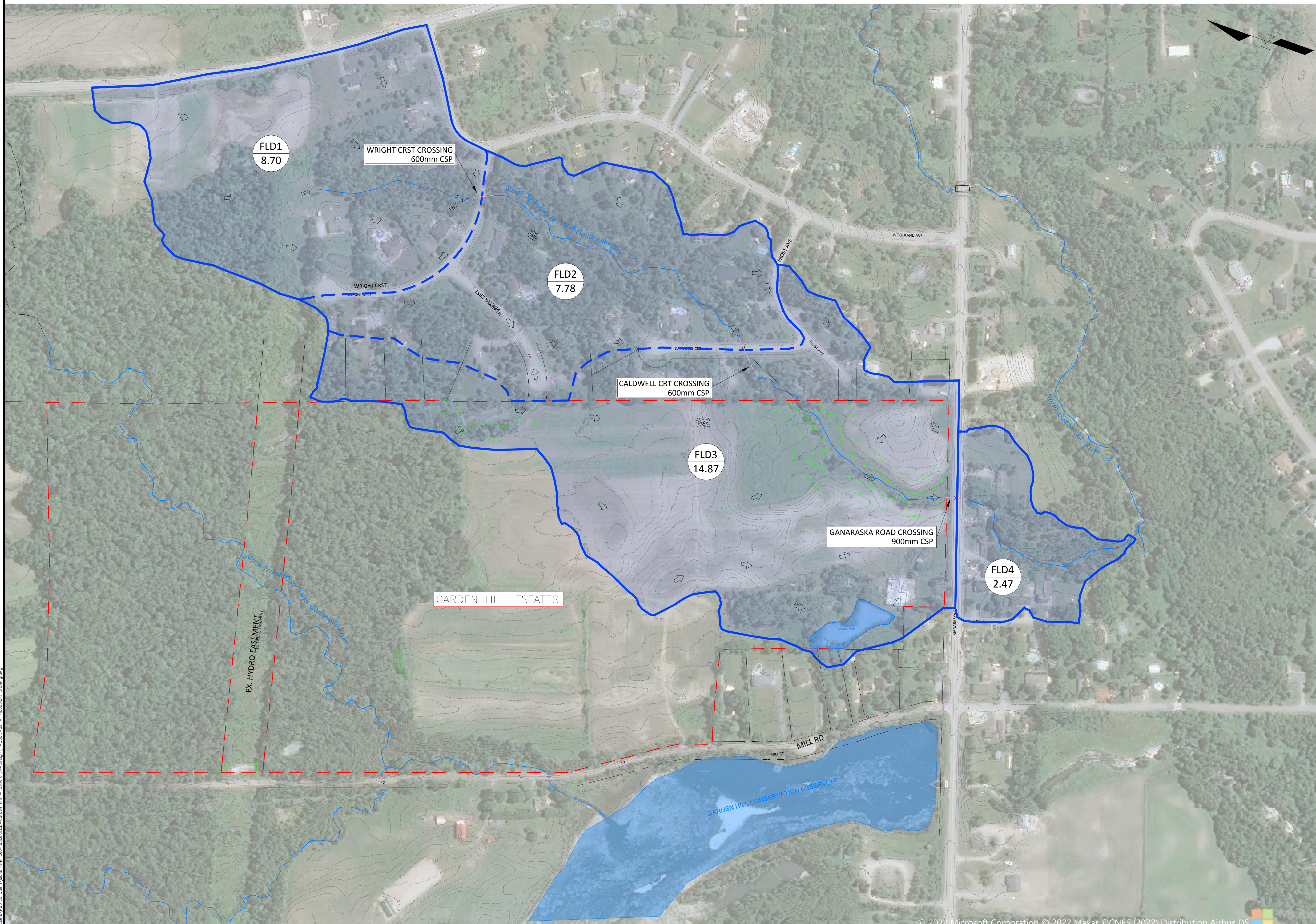
CONTOUR INTERVAL — 5 FEET, 2 1/2 FEET INTERPOLATION
GEODETIC DATUM — MEAN SEA LEVEL AS ESTABLISHED BY GEODETIC SURVEY OF CANADA.
PROJECTION SYSTEM — 3° TRANSVERSE MERCATOR

DATE JAN. 1990

SHEET 82 of 117

REGISTERED PROFESSIONAL ENGINEER A. E. MARIN PROVINCE OF ONTARIO
MARSHALL, MACKLIN, MONAGHAN LTD.

REGISTERED PROFESSIONAL ENGINEER L. SANCHEZ PROVINCE OF ONTARIO
TOTTEN SIMS HUBICKI ASSOCIATES



LEGEND

- PROPERTY BOUNDARY
- WATERCOURSE
- WETLAND BOUNDARY
- WETLAND BOUNDARY
- CATCHMENT BOUNDARY
- 101 — CATCHMENT ID
- 10.1 — AREA (ha)

NO.	DATE (D/M/Y)	REVISION	BY
–	–	–	–

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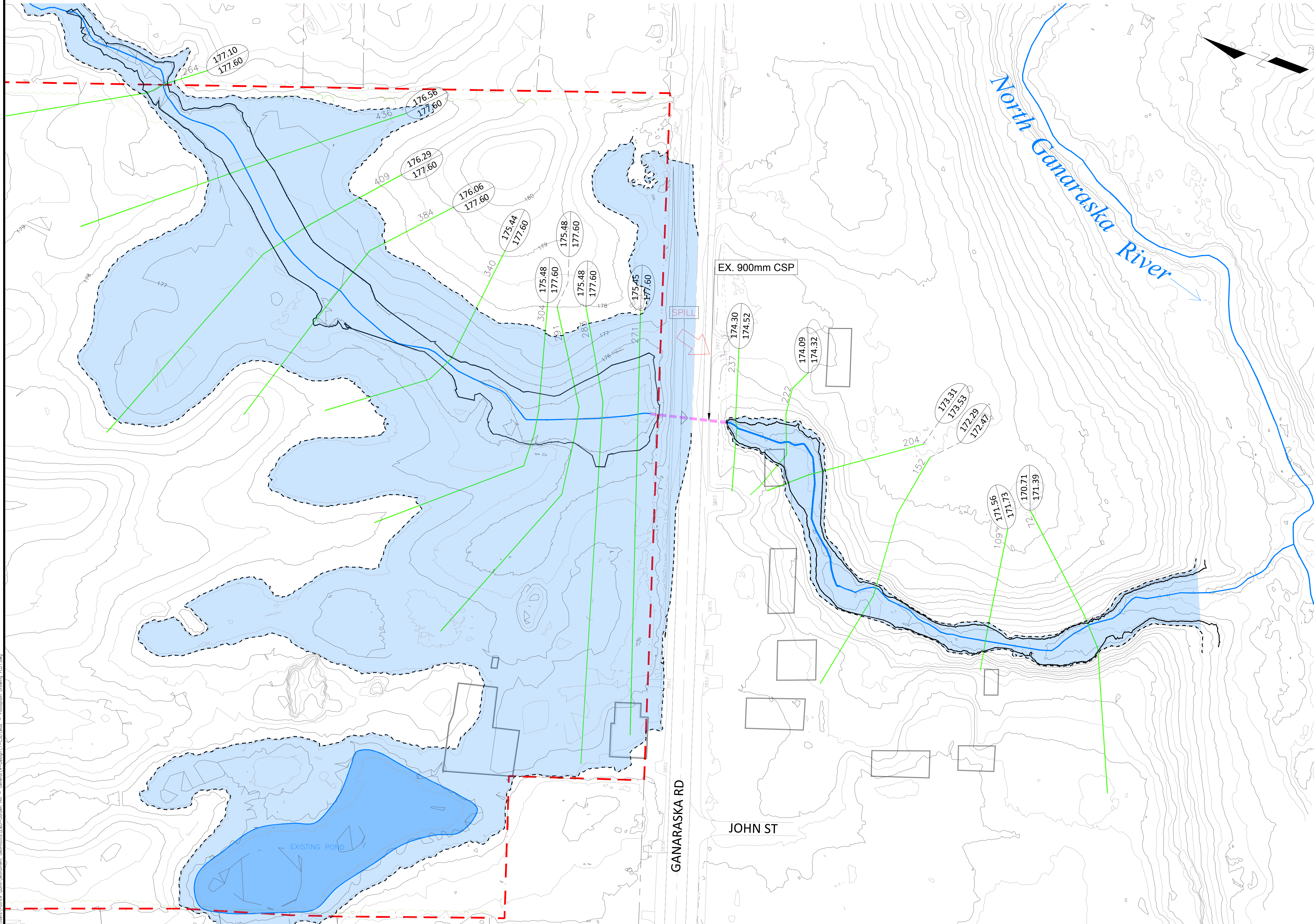
SCALES:	VERT.

MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES

MUNICIPALITY OF PORT HOPE
SOUTH TRIBUTARY FLOODLINE
CATCHMENT AREA DRAWING

DRAWING No.
FLD-1

0 50 100 Meters



LEGEND

- PROPERTY BOUNDARY
- WATERCOURSE
- 100-YR FLOOD LIMIT
- REGIONAL FLOOD LIMIT
- REGIONAL FLOOD AREA
- 173.71 – 100-yr WSEL
- 173.92 – REGIONAL WSEL
- 20 – STATION I.D.
- RIVER STATION

NO.	DATE (D/M/Y)	REVISION	BY
–	–	–	–

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ENGINEER:	–
DATE:	2022-03-18
SCALES:	HORZ. 1:600
SCALES:	VERT.

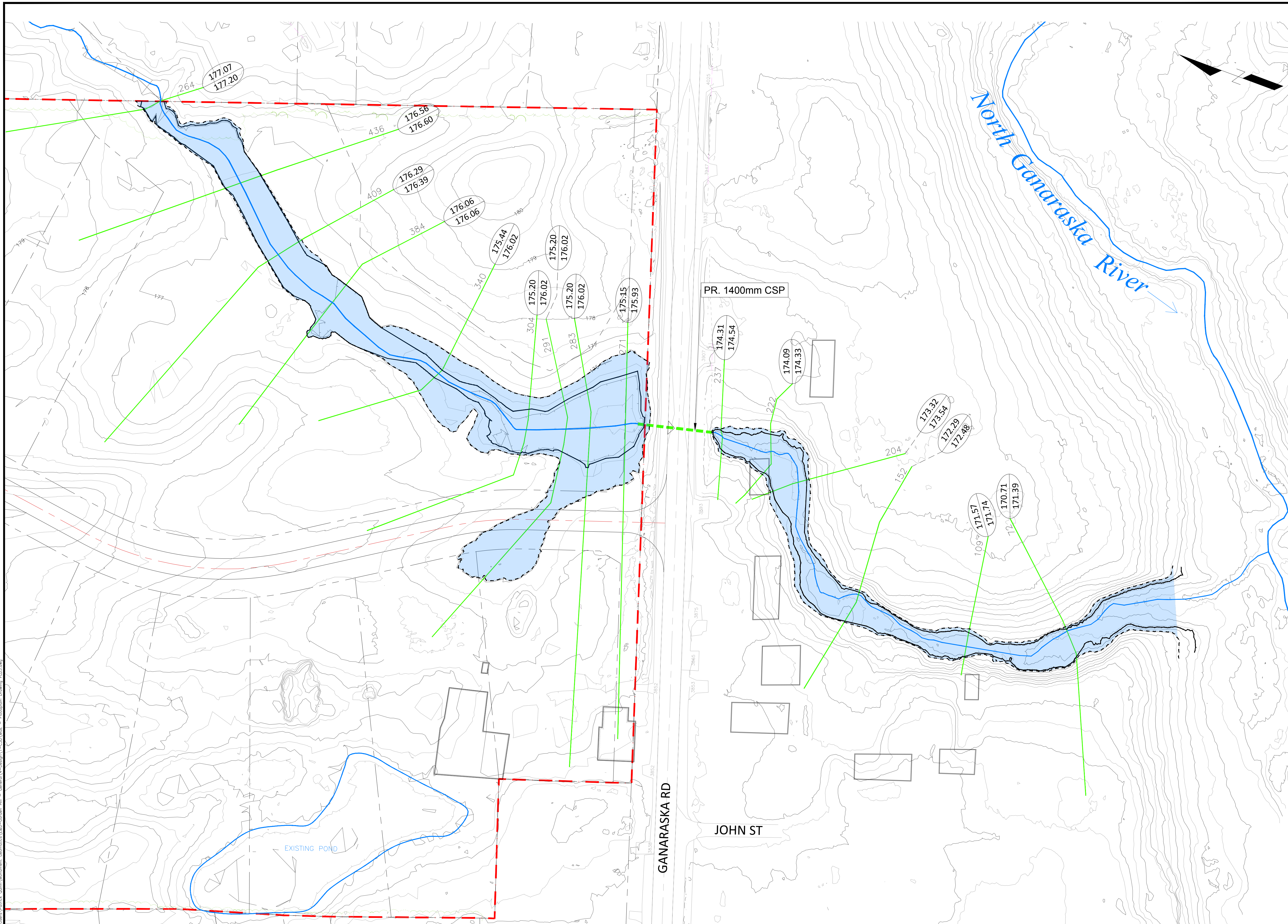
MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES
MUNICIPALITY OF PORT HOPE

SOUTH TRIBUTARY FLOODLINE
EXISTING CONDITIONS

DRAWING No.
FLD-2

0 15 30 Meters

 GENERAL NOTES: ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK.ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. ALL DIMENSIONS SHOWN ARE IN METERS OR MILLIMETERS, UNLESS OTHERWISE NOTED.



LEGEND

- PROPERTY BOUNDARY
- WATERCOURSE
- 100–YR FLOOD LIMIT
- REGIONAL FLOOD LIMIT
- REGIONAL FLOOD AREA
- 173.71 – 100–yr WSEL
- 173.92 – REGIONAL WSEL
- 20 – STATION I.D.
- RIVER STATION

NO.	DATE (D/M/Y)	REVISION	BY
–	–	–	–

NOT FOR CONSTRUCTION

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MONUMENT PROJECT No.:	21–0135
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CHECKED:	
ENGINEER:	–
DATE:	2022–03–18
SCALES:	HORZ. 1:600
SCALES:	VERT.

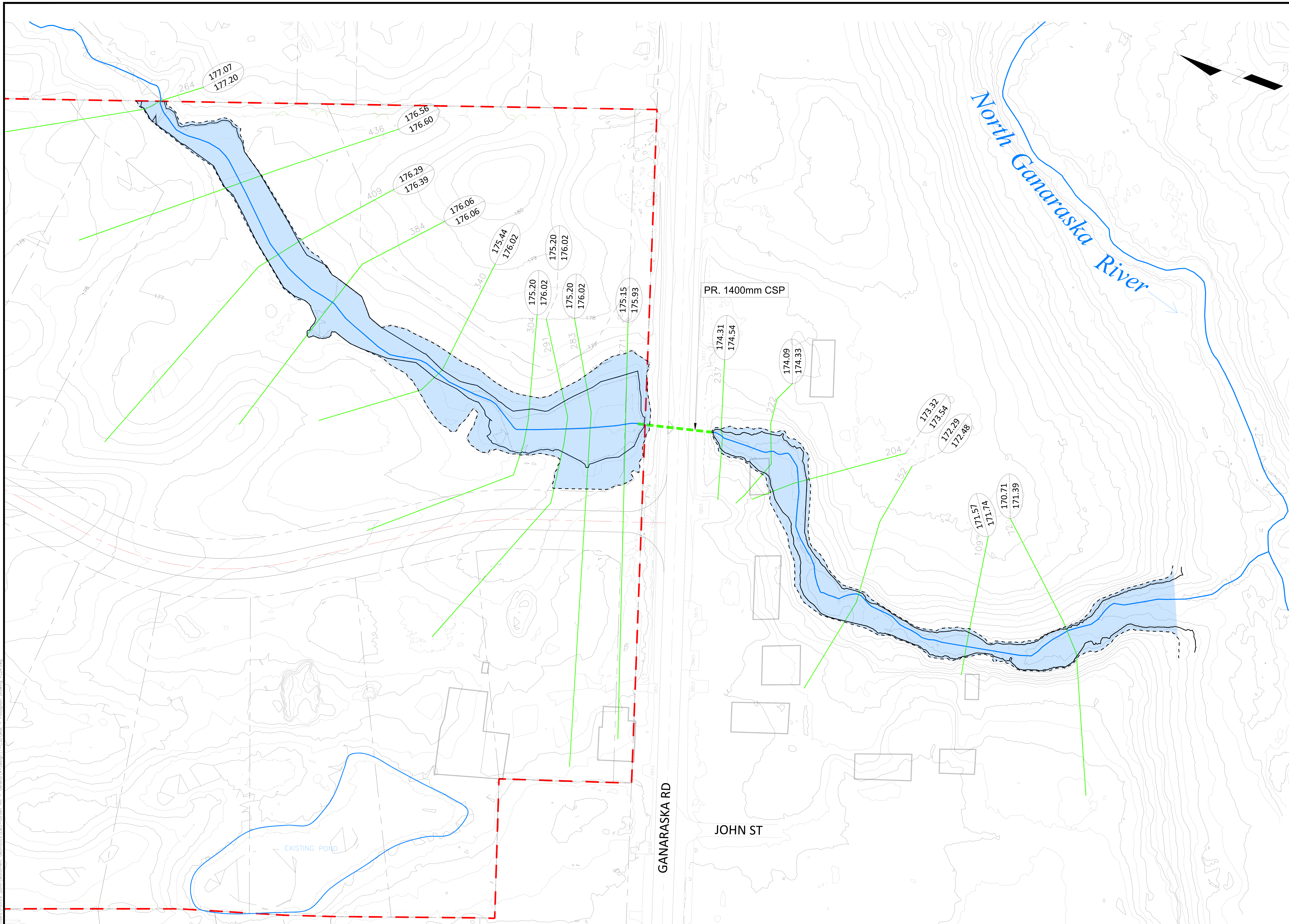
MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES
MUNICIPALITY OF PORT HOPE

SOUTH TRIBUTARY FLOODLINE
POST–CONDITIONS

DRAWING No.
FLD-3

0 15 30 Meters

 GENERAL NOTES: ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK.ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. ALL DIMENSIONS SHOWN ARE IN METERS OR MILLIMETERS, UNLESS OTHERWISE NOTED.



LEGEND

PROPERTY BOUNDARY	
WATERCOURSE	
100-YR FLOOD LIMIT	
REGIONAL FLOOD LIMIT	
REGIONAL FLOOD AREA	
173.71	100-yr WSEL
173.92	REGIONAL WSEL
20	STATION I.D.
	RIVER STATION

NO.	DATE (D/M/Y)	REVISION	BY

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MONUMENT PROJECT No.:	21-0135
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CHECKED:	
ENGINEER:	–
DATE:	2022-03-18
SCALES:	HORZ. 1:600
SCALES:	VERT.

MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES
MUNICIPALITY OF PORT HOPE

SOUTH TRIBUTARY FLOODLINE
POST-CONDITIONS WITH ACCESS

DRAWING No.
FLD-4

0 15 30 Meters

 GENERAL NOTES: ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK.ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. ALL DIMENSIONS SHOWN ARE IN METERS OR MILLIMETERS, UNLESS OTHERWISE NOTED.

APPENDIX I – METHODOLOGY

This section describes the methodology applied to determine hydrologic parameters for each catchment area.

1. **Floodline Assessment**
2. **Pre-development Onsite**
3. **Post-development Onsite**

South Tributary Floodline Analysis

Garden Hill Estates

Catchment	Area (ha)
FLD1	8.70
FLD2	7.76
FLD3	14.87
FLD4	2.47

Soil Groups Found
Soil A Classification: Ponty Pool Sandyloam
Soil B Classification: Bondhand Loam

HSG breakdown & Curve Number

Land Type	*CN	FLD1	FLD2	FLD3	FLD4
Impervious Area		1.10	0.94	0.67	1.00
Lakes & Wetland	50			0.86	
Hydrologic Soil Group A					
Crop (Improved land)	77	1.80		10.23	
Pasture (Unimproved Land)	40			0.88	
Woodland & Forests	30	4.69	4.07	0.87	0.80
Residntial 1 Acre Lot	51	1.11	2.75	0.90	0.13
Hydrologic Soil Group B					
Woodland & Forests	50			0.23	
Residntial 1 Acre Lot	68			0.23	0.54

*Curve Numbers represent AMC II conditions

	FLD1	FLD2	FLD3	FLD4
% Impervious:	13%	12%	5%	41%
Area (ha) :	8.70	7.76	14.87	2.47
Weighted Curve No.	**44.2**	**38**	**68**	**46**
AMC III Curve No.	**57**	**53**	**80**	**59**

Drainage Area Slope using 85/10 Method

Catchment	Elev 85%	Elev 10%	$L_{watershed}$	Slope
FLD1	195.5	187.5	442	2.4%
FLD2	186.5	181.5	206	3.2%
FLD3	186.0	176.0	540	2.5%
FLD4	176.5	173.5	95	4.2%

PCSWMM Input Summary

PCSWMM Input Summary				
Name	**FLD1**	**FLD2**	**FLD3**	**FLD4**
Outlet Point	OF1	OF1	OF1	OF1
Area	8.70	7.76	14.87	2.47
Flow Length	442	206	540	95
Slope	2.4%	3.2%	2.5%	4.2%
Percent Impervious (%)	13%	12%	5%	41%
N Impervious	0.013	0.013	0.013	0.013
N Pervious	0.24	0.24	0.13	0.13
Curve Number (AMC II)	44	38	68	46
Curve Number (AMC III)	57	53	80	59

Pre-development

Garden Hill Estates

Catchment	Area (ha)
101	6.99
102	1.23
103	1.43
104	1.49
105	1.37
106	2.70
107	9.27

Soil Groups Found
Soil A Classification: Ponty Pool Sandyloam
Soil B Classification: Bondhand Loam

Curve Number

Land Type	*CN	101	102	103	104	105	106	107
Impervious Area	-							
Lakes & Wetland	50	0.18	0.10	0.23	0.18			
Hydrologic Soil Group A								
Crop (Improved land)	77	5.93	0.78		1.11	0.63	1.08	1.95
Pasture (Unimproved Land)	40	0.88	0.35					
Woodland & Forests	30			0.57	0.20			3.60
Residential (1acre lot)	51							
Hydrologic Soil Group B								
Crop (Improved land)	86					0.74	1.62	1.56
Pasture (Unimproved Land)	50							
Woodland & Forests	60			0.63				2.42

*Curve Numbers represent AMC II conditions

	101	102	103	104	105	106	107
% Impervious:	0%	0%	0%	0%	0%	0%	0%
Area (ha) :	6.99	1.23	1.43	1.49	1.37	2.70	9.52
Weighted Curve No.	**72**	**64**	**46**	**67**	**82**	**82**	**56**

Drainage Area Slope using 85/10 Method

Catchment	Elev 85%	Elev 10%	$L_{watershed}$	Slope
101	186.0	177.0	336	3.6%
102	179.0	177.5	100	2.0%
103	185.0	179.0	255	3.1%
104	181.0	177.0	131	4.1%
104EXT	189.5	187.0	157	2.1%
105	183.5	176.0	232	4.3%
106	185.5	176.0	267	4.7%
107	186.0	178.5	393	2.5%

PCSWMM Input Summary

Name	101	102	103	104	105	106	107
Outlet Point	OF1	OF1	OF1	OF1	OF4	OF3	OF2
Area	6.99	1.23	1.43	1.49	1.37	2.70	9.52
Flow Length	540	100	255	131	232	267	393
Slope	2.5%	2.0%	3.1%	4.1%	4.3%	4.7%	2.5%
Percent Impervious (%)	0	0	0	0	0	0	0
N Impervious	0.013	0.013	0.013	0.013	0.013	0.013	0.013
N Pervious	0.06	0.06	0.13	0.13	0.06	0.06	0.13
Curve Number	72	64	46	67	82	82	55

Post-development

Garden Hill Estates

Catchment	Area (ha)
200	1.31
201	3.90
202A	2.25
202B	1.62
203	1.23
300A	5.60
300B	2.43
400	0.78
500	0.65
600	4.70

Soil Groups Found

Soil A Classification: Ponty Pool Sandyloam
Soil B Classification: Bondhand Loam
Soil C Classification: -
Soil D Classification: -

Drainage Area Slope using 85/10 Method

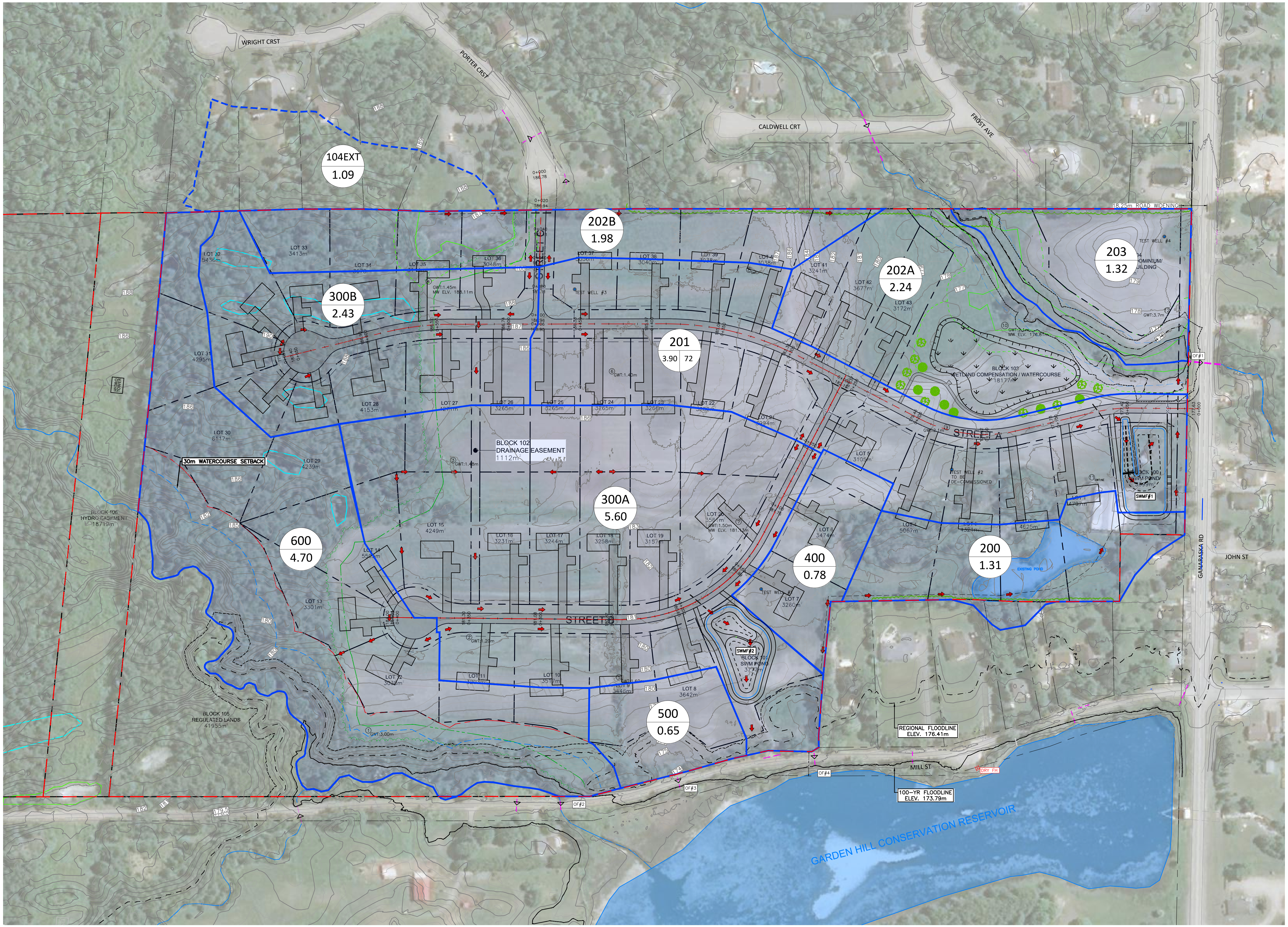
Catchment	$L_{watershed}$	Slope
200	100	1.0%
201	150	3.5%
202A	120	4.0%
202B	255	1.0%
203	100	2.0%
300A	140	2.5%
300B	66	2.0%
400	50	2.0%
500	75	3.0%
600	239	2.3%

PCSWMM Input Summary

Name	200	201	202A	202B	203
Outlet Point	OF1	OF1	OF1	OF1	OF1
Area	1.31	3.90	2.25	1.62	1.23
Flow Length	100	150	120	255	58
Slope	1%	3.5%	4%	1%	2%
Percent Impervious (%)	4.4%	27.3%	7.7%	7.2%	25.0%
N Impervious	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.25	0.06
Subarea Routing (%)	80%	30%	80%	80%	-
Curve Number (AMCII)	60	73	72	72	72

Name	300A	300B	400	500	600
Outlet Point	OF3	OF3	OF4	OF3	OF2
Area (ha)	5.60	2.43	0.78	0.65	4.70
Flow Length (m)	140	66	50	75	239
Slope	2.5%	2.0%	2.0%	3.0%	2.3%
Percent Impervious (%)	18.7%	30.6%	15.6%	4.4%	4.9%
N Impervious	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.25	0.25
Subarea Routing	30%	30%	80%	80%	80%
Curve Number (AMCII)	75	75	78	78	56

APPENDIX J – POST-DEVELOPMENT CATCHMENT AREA DRAWING



LEGEND

- PROPERTY BOUNDARY
- SIGNIFICANT WOODLAND AREA
- WETLAND
- WATERCOURSE
- WATERCOURSE SETBACK
- 100-YR FLOOD LIMIT
- REGIONAL FLOOD LIMIT
- 6m SETBACK FROM REGIONAL FLOOD LIMIT
- WETLAND BOUNDARY
- 15m WETLAND SETBACK
- 10m WOODLAND SETBACK
- EROSION HAZARD LIMIT
- VERNAL POOLS
- 101 — CATCHMENT ID
- 2.09 | 60 — CURVE NUMBER
- AREA (ha)
- CATCHMENT BOUNDARY
- EXTERNAL CATCHMENT
- OVERLAND FLOW ARROW
- 12 BOREHOLE
- GWT:3.7m — DEPTH TO WATER TABLE

NOTES:
CONTOUR DATA WAS DERIVED FROM MONUMENT GPS TOPOGRAPHIC SURVEY SUPPLEMENTED WITH IMAGE-DERIVED FROM ONTARIO'S DIGITAL ELEVATION MODEL.

NO.	DATE (D/M/Y)	REVISION	BY
-	-	-	-

NOT FOR CONSTRUCTION

MONUMENT
GEOMATICS
MONUMENTGE.COM

MONUMENT PROJECT No.:	-
DRAWN:	P.Q.
CHECKED:	-
ENGINEER:	-
DATE:	12-04-2022
SCALES:	HORZ. 1:1250
SCALES:	VERT.

MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES

MUNICIPALITY OF PORT HOPE
POST-DEVELOPMENT
DRAINAGE PLAN

DRAWING No.
SWM-02

 GENERAL NOTES: ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK.ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. ALL DIMENSIONS SHOWN ARE IN METERS OR MILLIMETERS, UNLESS OTHERWISE NOTED.

APPENDIX K – STAGE-STORAGE-DISCHARGE RELATIONSHIP

MONUMENT GEOMATICS

Monument Geomatics Estimating Inc.
93 Bellevue Drive, Belleville, ON K8N 4Z5
info@monumentge.com

Stage Storage Calculations

Stormwater Management Facility #1 (South)

Pond Details	
Bottom of Pond =	**175.40**
Permanent Pool Elevation (P.P) =	**0**
Extended Detention (E.D.) =	**175.75**
Active Storage (A.S.) =	**176.30**
Top of Berm Elevation (T.B.) =	**176.60**
Depth of Active Storage =	0.9
Depth of Permanent Pool =	0.0
Depth of Pond =	1.2
Pond Interval =	0.1

Outlet 1		Outlet 2		Outlet 3	
Orifice		Sharp-Crested Weir		Emergency Spillway	
Invert =	**175.4**	**Invert =**	**176.05**	**Invert =**	**176.3**
Diameter =	**0.075**	**Weir Length =**	**0.65**	**Weir Length =**	**3**
C =	**0.60**				
Obvert =	175.48				
Radius =	0.04				
Area =	0.00				

Equations

Orifice = $Q = CA_o(2gh)^{0.5}$

V-Notch Weir = $Q = 1.38H^{2.5}$

Broad-Crested Weir = $Q = 1.67LH^{\frac{3}{2}}$

Sharp-Crested Weir = $Q = 1.84LH^{\frac{3}{2}}$

C = Orifice Coefficient
A_o = Pipe Area, m2
h = Orifice Head, m
H = Head, m
L = Length of Weir

						Outlet 1 Orifice		Outlet 2 Sharp-Crested Weir		Outlet 3 Emergency Spillway				
	Elevation (m)	**Area (m^2)**	**Incremental Volume (m^3)**	**Cumulative Volume (m^3)**	**Active Storage Depth, (m)**	**Orifice Head, h, (m)**	**$Q_{orifice}$ (m^3/s)**	**Head, H (m)**	**$Q_{outlet\,2}$ (m^3/s)**	**Head, h (m)**	**$Q_{outlet3}$**	**PP Storage (m^3)**	**Active Storage (m^3)**	**Discharge (m^3/s)**
	175.40	0.9	0.0	0	0.00	-0.038	0.000	0.00	0.000	0.00	0.000	0.0	0.0	0.000
	175.50	398.3	20.0	20.0	0.10	0.062	0.003	0.00	0.000	0.00	0.000	0.0	20.0	0.003
	175.60	498.5	44.8	64.8	0.20	0.162	0.005	0.00	0.000	0.00	0.000	0.0	64.8	0.005
	175.70	594.7	54.7	119.5	0.30	0.262	0.006	0.00	0.000	0.00	0.000	0.0	119.5	0.006
	175.80	695.1	64.5	183.9	0.40	0.362	0.007	0.00	0.000	0.00	0.000	0.0	183.9	0.007
	175.90	798.2	74.7	258.6	0.50	0.462	0.008	0.00	0.000	0.00	0.000	0.0	258.6	0.008
	176.00	924.6	86.1	344.8	0.60	0.562	0.009	0.00	0.000	0.00	0.000	0.0	344.8	0.009
	176.10	1084.0	100.4	445.2	0.70	0.662	0.010	0.05	0.013	0.00	0.000	0.0	445.2	0.023
	176.20	1252.5	116.8	562.0	0.80	0.762	0.010	0.15	0.069	0.00	0.000	0.0	562.0	0.080
A.S.	176.30	1431.7	134.2	696.2	0.90	0.862	0.011	0.25	0.149	0.00	0.000	0.0	696.2	0.160
	176.40	1583.6	150.8	847.0	1.00	0.962	0.012	0.35	0.248	0.10	0.158	0.0	847.0	0.418
	176.50	1743.8	166.4	1013.4	1.10	1.062	0.012	0.45	0.361	0.20	0.448	0.0	1013.4	0.821
T.P.	176.60	1912.0	182.8	1196.1	1.20	1.162	0.013	0.55	0.488	0.30	0.823	0.0	1196.1	1.324

**Note: All units are in metric, unless otherwise stated*

Monument Geomatics Estimating Inc.
93 Bellevue Drive, Belleville, ON K8N 4Z5
info@monumentge.com

Stage Storage Calculations

Stormwater Management Facility #2 (North)

Pond Details	
Bottom of Pond =	**177.80**
Permanent Pool Elevation (P.P) =	**178.8**
Extended Detention (E.D.) =	**0.00**
Active Storage (A.S.) =	**179.50**
Top of Berm Elevation (T.B.) =	**179.80**
Depth of Active Storage =	0.7
Depth of Permanent Pool =	1.0
Depth of Pond =	2.0
Pond Interval =	0.1

Outlet 1		Outlet 2		Outlet 3	
Orifice		Sharp-Crested Weir		Emergency Spillway	
Invert =	**178.8**	**Invert =**	**179.10**	**Invert =**	**179.50**
Diameter =	**0.1**	**Weir Length =**	**0.85**	**Weir Length =**	**3**
C =	**0.60**				
Obvert =	178.90				
Radius =	0.05				
Area =	0.01				

Equations

Orifice = $Q = CA_o(2gh)^{0.5}$

V-Notch Weir = $Q = 1.38H^{2.5}$

Broad-Crested Weir = $Q = 1.67LH^{\frac{3}{2}}$

Sharp-Crested Weir = $Q = 1.84LH^{\frac{3}{2}}$

where,

C = Orifice Coefficient

A_o = Pipe Area, m2

h = Orifice Head, m

H = Head, m

L = Length of Weir

						Outlet 1		Outlet 2		Outlet 3				
						Orifice		Sharp-Crested Weir		Emergency Spillway				
	Elevation (m)	Area (m^2)	Incremental Volume (m^3)	Cumulative Volume (m^3)	Active Storage Depth, (m)	Orifice Head, h, (m)	$Q_{orifice}$ (m^3/s)	Head, H (m)	$Q_{outlet\ 2}$ (m^3/s)	Head, h (m)	$Q_{outlet3}$	PP Storage (m^3)	Active Storage (m^3)	Discharge (m^3/s)
	177.80	513	0.0	0	0.00	0.000	0.000	0.00	0.000	0.00	0.000	0.0	0.0	0.000
	177.90	557	53.5	53.5	0.00	0.000	0.000	0.00	0.000	0.00	0.000	53.5	0.0	0.000
	178.00	603	58.0	111.6	0.00	0.000	0.000	0.00	0.000	0.00	0.000	111.6	0.0	0.000
	178.10	650	62.7	174.2	0.00	0.000	0.000	0.00	0.000	0.00	0.000	174.2	0.0	0.000
	178.20	699	67.4	241.7	0.00	0.000	0.000	0.00	0.000	0.00	0.000	241.7	0.0	0.000
	178.30	748	72.3	314.0	0.00	0.000	0.000	0.00	0.000	0.00	0.000	314.0	0.0	0.000
	178.40	798	77.3	391.3	0.00	0.000	0.000	0.00	0.000	0.00	0.000	391.3	0.0	0.000
	178.50	850	82.4	473.7	0.00	0.000	0.000	0.00	0.000	0.00	0.000	473.7	0.0	0.000
	178.60	902	87.6	561.3	0.00	0.000	0.000	0.00	0.000	0.00	0.000	561.3	0.0	0.000
	178.70	956	92.9	654.2	0.00	0.000	0.000	0.00	0.000	0.00	0.000	654.2	0.0	0.000
P.P	178.80	1110	103.3	757.5	0.00	-0.050	0.000	0.00	0.000	0.00	0.000	757.5	0.0	0.000
	178.90	1183	114.6	872.2	0.10	0.050	0.005	0.00	0.000	0.00	0.000	0.0	114.6	0.005
	179.00	1257	122.0	994.2	0.20	0.150	0.008	0.00	0.000	0.00	0.000	0.0	236.7	0.008
	179.10	1333	129.5	1123.7	0.30	0.250	0.010	0.00	0.000	0.00	0.000	0.0	366.2	0.010
	179.20	1410	137.2	1260.9	0.40	0.350	0.012	0.10	0.049	0.00	0.000	0.0	503.4	0.062
	179.30	1489	145.0	1405.9	0.50	0.450	0.014	0.20	0.140	0.00	0.000	0.0	648.3	0.154
	179.40	1570	153.0	1558.8	0.60	0.550	0.015	0.30	0.257	0.00	0.000	0.0	801.3	0.272
A.S.	179.50	1652	161.1	1719.9	0.70	0.650	0.017	0.40	0.396	0.00	0.000	0.0	962.4	0.412
	179.60	1735	169.3	1889.3	0.80	0.750	0.018	0.50	0.553	0.10	0.158	0.0	1131.8	0.729
	179.70	1820	177.7	2067.0	0.90	0.850	0.019	0.60	0.727	0.20	0.448	0.0	1309.5	1.194
T.B.	179.80	1907	186.3	2253.4	1.00	0.950	0.020	0.70	0.916	0.30	0.823	0.0	1495.9	1.760

**Note: All units are in metric, unless otherwise stated*

APPENDIX L – MANNING'S OPEN CHANNEL FLOW EQUATION

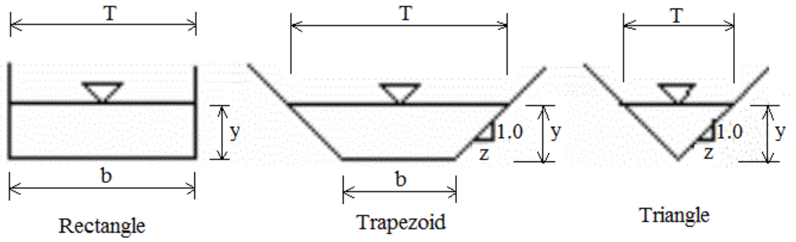
Manning's Open Channel Flow Sheet

$$Q = \frac{1}{n} \, x \, A \, x \, R^{\frac{2}{3}} \, x \, S^{\frac{1}{2}}$$

Where,

n = Manning's coefficient
A = Cross-sectional Area (m2)
R = Hydraulic Radius = Area / Wetted Perimeter (m)
S = Longitudinal Slope (m/m)

Select Channel Type: Trapezoid



Input Values		
Bottom width, b =	0.5	m
Channel depth, y =	0.45	m
Side Slope, z =	3	:1
Longitudinal Slope, S =	0.03	m/m
Roughness, n =	0.035	Grassed

Calculated Values	
Top Width, T =	3.200 m
Area, A =	0.833 m^2
Wetted Perimeter, P =	3.346 m
Hydraulic Radius, R =	0.249 m

Channel Capacity

Channel Velocity = **1.958 m/s**

Channel Flow = **1.630 m^3/s**

Note: Channel flow is calculated to depth of channel assumed to be full.

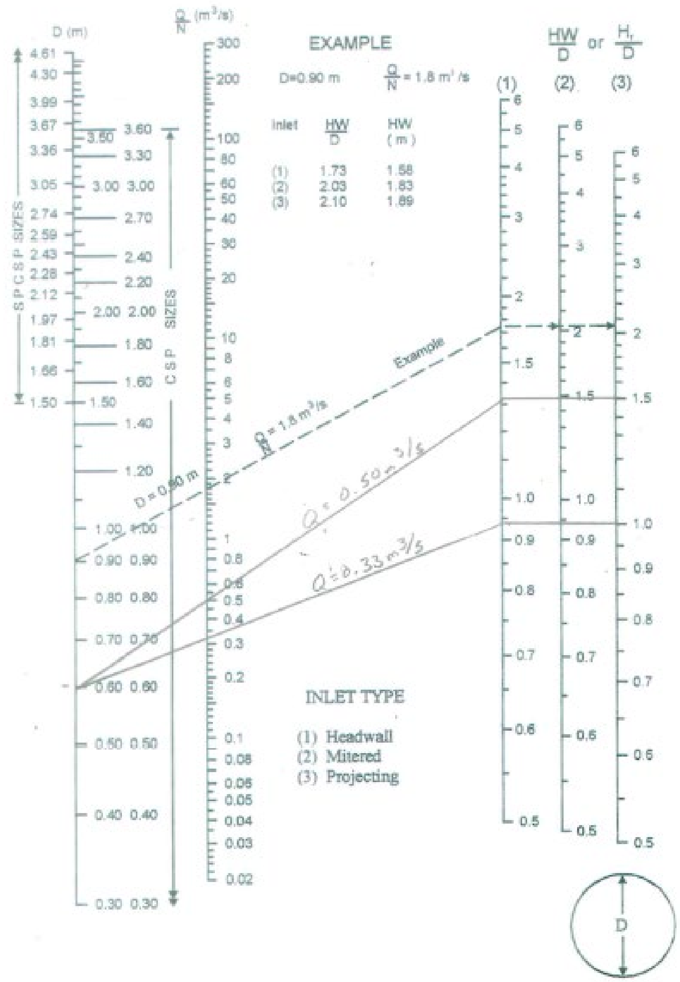
APPENDIX M – DESIGN CHART 2.32: INLET CONTROL CIRCULAR CSP AND SPCSP

Garden Hill Estates

Maximum Circular Culvert Size.
600mm CSP

MTO Drainage Management Manual

Design Chart 2.32: Inlet Control: Circular CSP and SPCSP Culverts



Source: Herr (1977)

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APPENDIX N – PCSWMM OUTPUT FILES

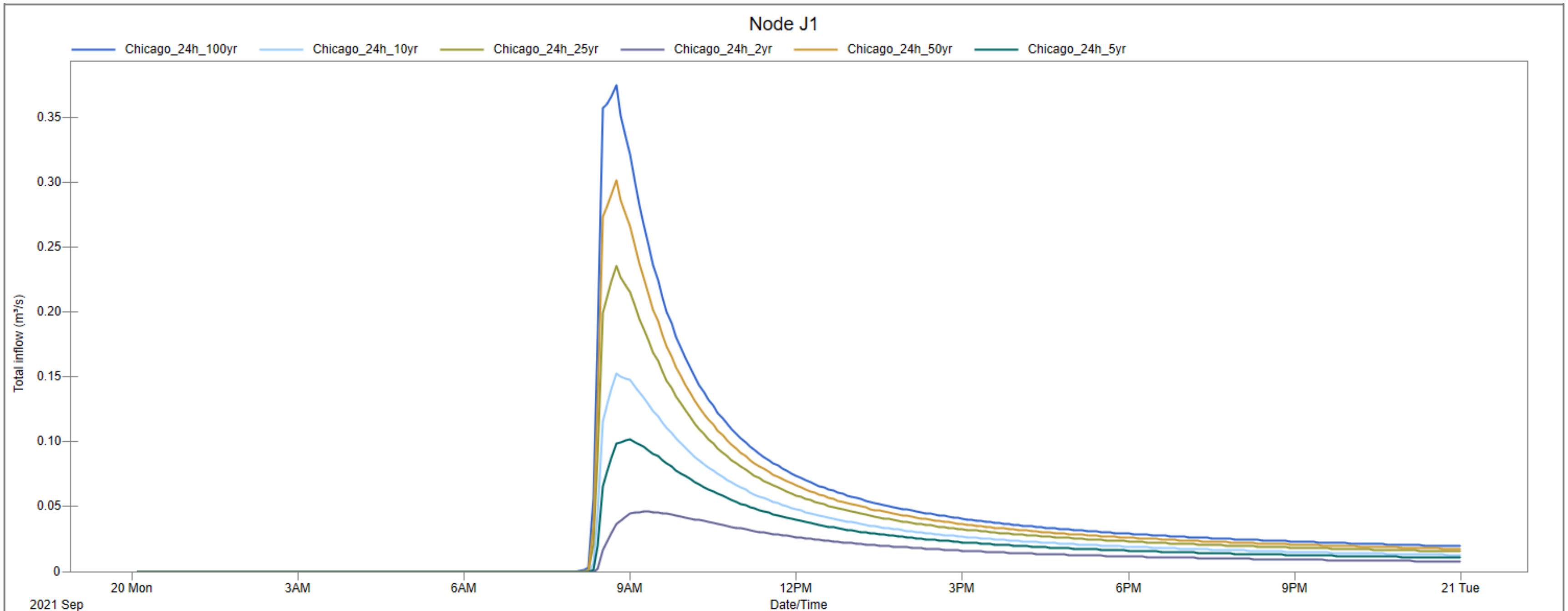
Table 1: Existing Conditions - Catchment Summary

Name	Curve Number	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	N Imperv	N Perv	Dstore Imperv (mm)	Dstore Perv (mm)	Infiltration Method
101	72	6.9987	129.606	539.998	2.5	0.013	0.06	1	2.5	CURVE_NUMBER
102	64	1.2332	123.32	100	2	0.013	0.06	1	2.5	CURVE_NUMBER
103	46	1.4344	56.251	255	3.1	0.013	0.13	1	2.5	CURVE_NUMBER
104	67	1.4917	113.87	131	4.1	0.013	0.13	1	2.5	CURVE_NUMBER
105	82	1.3682	62.475	219	4.7	0.013	0.06	1	2.5	CURVE_NUMBER
106	82	2.6961	100.978	266.999	4.5	0.013	0.06	1	2.5	CURVE_NUMBER
107	56	9.2668	308.893	300	2.3	0.013	0.13	1	5	CURVE_NUMBER

Table 1: Post-condition Subcatchments

Name	Area (ha)	Width (m)	Flow Length (m)	Slope (%)	Imperv. (%)	N Imperv	N Perv	Dstore Imperv (mm)	Dstore Perv (mm)	Zero Imperv (%)	Subarea Routing	Percent Routed (%)	Curve Number
200	1.3105	131.05	100	1	2.6	0.013	0.25	1	5	25	PERVIOUS	80	60
201	3.9009	260.06	150	3.5	28	0.013	0.25	1	5	25	PERVIOUS	30	72
202A	2.2455	187.125	120	4	7.7	0.013	0.25	1	5	25	PERVIOUS	80	72
202B	1.6157	323.14	50	2	7.2	0.013	0.25	1	5	25	PERVIOUS	80	72
203	1.2332	123.32	100	2	0	0.013	0.06	1	5	25	OUTLET	100	72
300A	5.5983	399.879	140	2.5	18.7	0.013	0.25	1	5	25	PERVIOUS	30	75
300b	2.4396	369.636	66	2	30.6	0.013	0.25	1	5	25	PERVIOUS	30	72
400	0.7821	78.21	100	2	15.6	0.013	0.25	1	5	25	PERVIOUS	80	78
500	0.6578	87.707	75	3	4.4	0.013	0.25	1	5	25	PERVIOUS	80	78
600	4.7079	196.163	239.999	2.3	4.9	0.013	0.25	1	5	25	PERVIOUS	80	56

Existing Peak Flows to Outfall#1 – 2yr to 100-yr – Chicago 24hr



Data | Objectives | Error | Storage | Patterns | Edit | Derive | Audit | Events | Scatter | Duration | IDF

Objective functions for: Total inflow (m³/s)

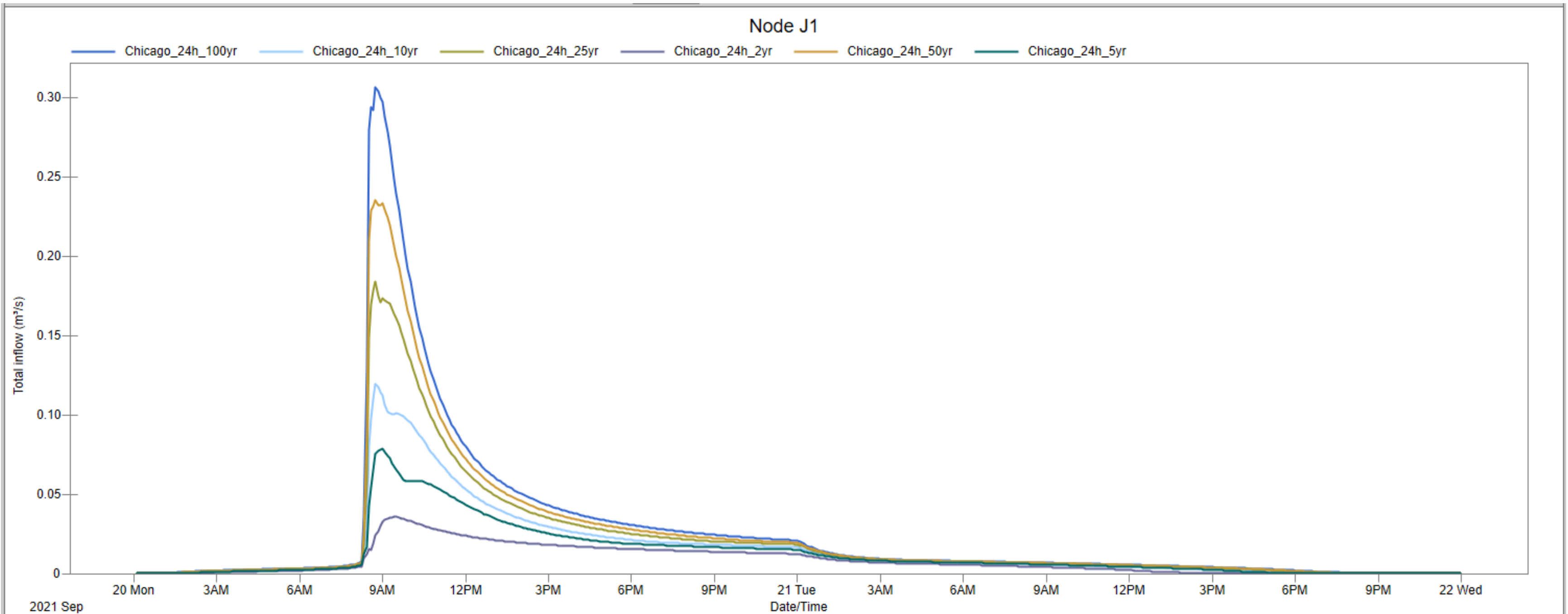
From Sep 19, 2021 10:53 PM to Sep 21, 2021 1:11 AM (26.31 hours)

	J1 Chicago_24h_100yr	J1 Chicago_24h_10yr	J1 Chicago_24h_25yr	J1 Chicago_24h_2yr	J1 Chicago_24h_50yr	J1 Chicago_24h_5yr
Maximum Total inflow (m³/s)	0.3756	0.1528	0.2359	0.04644	0.3021	0.1024
Minimum Total inflow (m³/s)	0	0	0	0	0	0
Mean Total inflow (m³/s)	0.04454	0.02511	0.03282	0.01203	0.03849	0.0196
Duration of Exceedances (h)	23.92	23.92	23.92	23.92	23.92	23.92
Duration of Deficits (h)	7.917	8.167	8.167	8.25	8	8.167
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	1	1	1	1	1	1
Volume of Exceedances (m³)	3832	2160	2823	1035	3311	1686
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total inflow (m³)	3835	2162	2826	1036	3314	1688

Total inflow (m³/s) Exceedance: 0

Total inflow (m³/s) Deficit: 0

Post-controlled Peak Flows to Outfall#1 – 2yr to 100-yr – Chicago 24hr



Data | Objectives | Error | Storage | Patterns | Edit | Derive | Audit | Events | Scatter | Duration | IDF

Objective functions for Total inflow (m³/s)

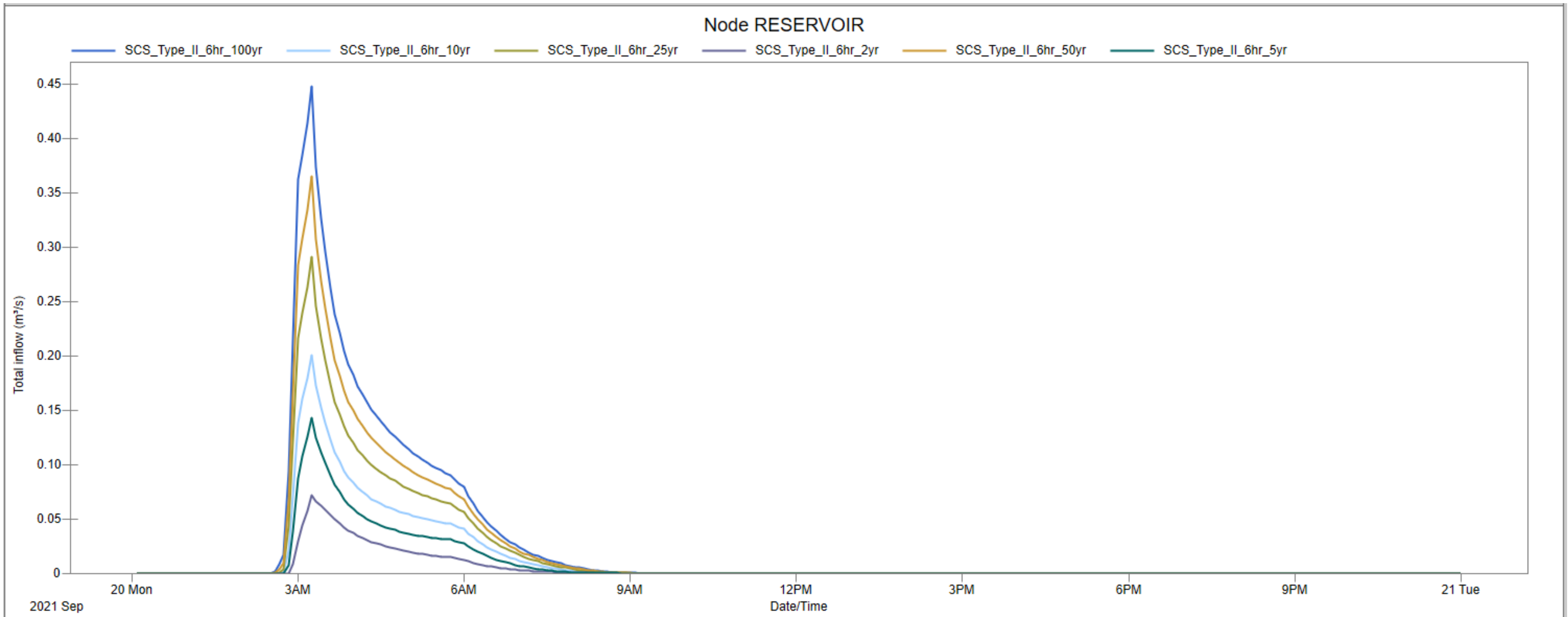
From Sep 19, 2021 9:41 PM to Sep 22, 2021 2:23 AM (52.71 hours)

	J1 Chicago_24h_100yr	J1 Chicago_24h_10yr	J1 Chicago_24h_25yr	J1 Chicago_24h_2yr	J1 Chicago_24h_50yr	J1 Chicago_24h_5yr
Maximum Total inflow (m³/s)	0.3066	0.1195	0.1841	0.0356	0.2354	0.07859
Minimum Total inflow (m³/s)	2.016E-06	5.154E-07	9.557E-07	1.226E-07	1.42E-06	3.272E-07
Mean Total inflow (m³/s)	0.02568	0.01528	0.01945	0.007958	0.02248	0.01224
Duration of Exceedances (h)	47.92	47.92	47.92	47.92	47.92	47.92
Duration of Deficits (h)	0	0	0	0	0	0
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	0	0	0	0	0	0
Volume of Exceedances (m³)	4430	2635	3355	1373	3878	2112
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total inflow (m³)	4430	2635	3355	1373	3878	2112

Total inflow (m³/s) Exceedance: 0

Total inflow (m³/s) Deficit: 0

Existing Peak Flows to Reservoir – 2yr to 100-yr – SCS Type II 6hr



Data | Objectives | Error | Storage | Patterns | Edit | Derive | Audit | Events | Scatter | Duration | IDF

Objective functions for: Total inflow (m³/s)

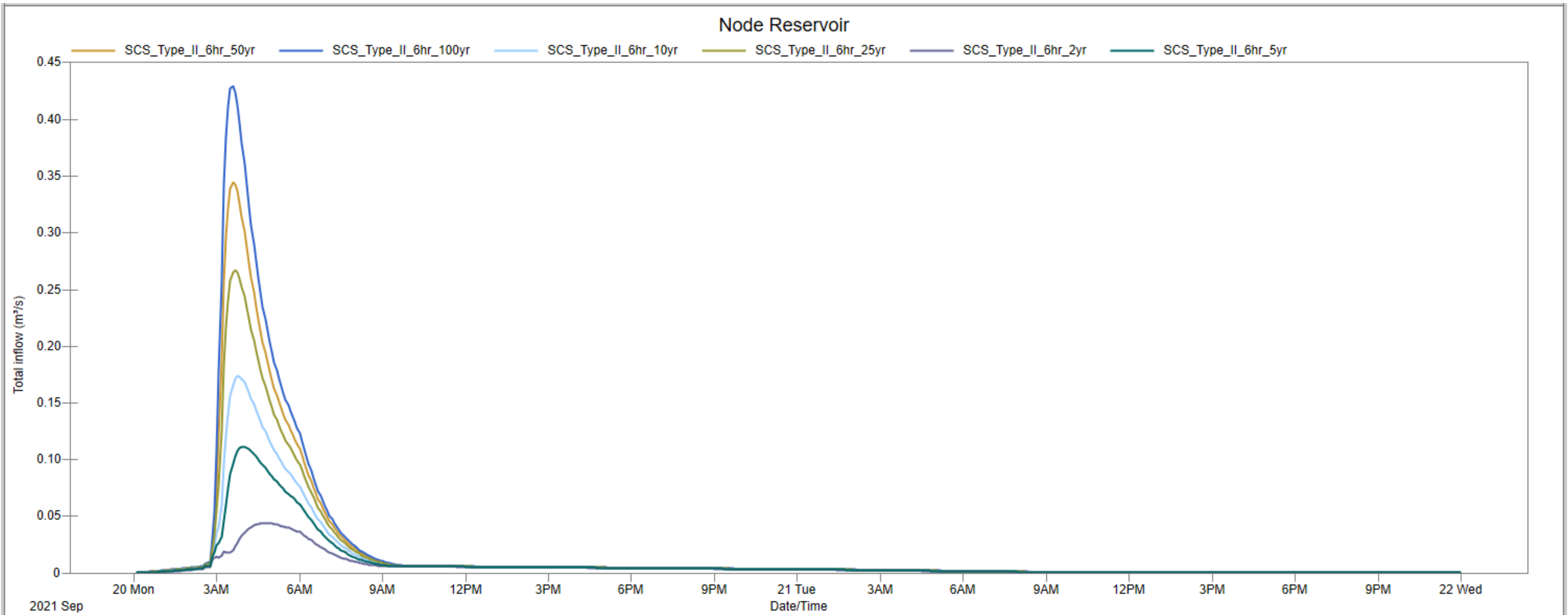
From Sep 19, 2021 10:53 PM to Sep 21, 2021 1:11 AM (26.31 hours)

	RESERVOIR SCS_Type_II_6hr_100yr	RESERVOIR SCS_Type_II_6hr_10yr	RESERVOIR SCS_Type_II_6hr_25yr	RESERVOIR SCS_Type_II_6hr_2yr	RESERVOIR SCS_Type_II_6hr_50yr	RESERVOIR SCS_Type_II_6hr_5yr
Maximum Total inflow (m³/s)	0.4485	0.2012	0.2909	0.07148	0.3654	0.1427
Minimum Total inflow (m³/s)	0	0	0	0	0	0
Mean Total inflow (m³/s)	0.027	0.01218	0.01775	0.004361	0.0222	0.008337
Duration of Exceedances (h)	23.92	23.92	23.92	23.92	23.92	23.92
Duration of Deficits (h)	17.25	17.67	17.5	18	17.33	17.83
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	2	2	2	2	2	2
Volume of Exceedances (m³)	2325	1048	1528	375.5	1911	717.8
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total inflow (m³)	2325	1048	1528	375.5	1911	717.8

Total inflow (m³/s) Exceedance: 0

Total inflow (m³/s) Deficit: 0

Post-controlled Peak Flows to Reservoir – 2yr to 100-yr – SCS Type II 6hr



Data | Objectives | Error | Storage | Patterns | Edit | Derive | Audit | Events | Scatter | Duration | IDF

Objective functions for: Total inflow (m³/s)

From Sep 19, 2021 9:41 PM to Sep 22, 2021 2:23 AM (52.71 hours)

	Reservoir SCS_Type_II_6hr_100yr	Reservoir SCS_Type_II_6hr_10yr	Reservoir SCS_Type_II_6hr_25yr	Reservoir SCS_Type_II_6hr_2yr	Reservoir SCS_Type_II_6hr_50yr	Reservoir SCS_Type_II_6hr_5yr
Maximum Total inflow (m³/s)	0.4298	0.1737	0.2675	0.04405	0.3449	0.1113
Minimum Total inflow (m³/s)	1.3E-07	1.171E-07	8.117E-08	8.576E-08	1.216E-07	9.673E-08
Mean Total inflow (m³/s)	0.02149	0.0116	0.01542	0.005146	0.01837	0.00881
Duration of Exceedances (h)	47.92	47.92	47.92	47.92	47.92	47.92
Duration of Deficits (h)	0	0	0	0	0	0
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	0	0	0	0	0	0
Volume of Exceedances (m³)	3707	2001	2660	887.7	3168	1520
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total inflow (m³)	3707	2001	2660	887.7	3168	1520

Total inflow (m³/s) Exceedance: 0

Total inflow (m³/s) Deficit: 0