Servicing and Stormwater Management Report
Garden Hill Estates

Mistral Land Development Inc.

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Revised December 20, 2022

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TABLE OF CONTENTS

Table of Tables

Table of Figures

List of Appendices

Revision Summary

This Servicing and Stormwater Management Report is a revised report in response to comments provided from all stakeholders after the second draft plan submission on September 9, 2022. A summary of the key updates to this report are listed below.

<u>Section 2 – Proposed Development</u>

Phase 2 was removed from the Draft Plan and all other supporting plans. These areas are now referred to as "Other Lands Owned by the Developer" illustrated on the draft plan of subdivision. References to Phase 2 in this report have been updated accordingly. The proposed development now consists of 31 lots instead of 43, however, the SWM design concept has not been revised in order to accommodate future development lands if the additional lots are pursued.

<u>Section 3 – Water Supply and Sewage Disposal</u>

Further details provided on the water supply for fire protection as per comments from the Municipality of Port Hope Fire and Emergency Services.

<u>Section 8 – Peak Flows</u>

Peak flows have been updated from the Post-development Uncontrolled Model to match the Post-controlled Model results.

<u>Section 10 – Water Balance</u>

A concluding paragraph has been added to the end of this Section, describing the recommended steps forward for water balance and recharge analysis onsite which will be conducted at the time of detailed design. The further water balance analysis was recommended in Greer Galloway Group's response letter titled *GRCA Hydrogeology Comment Response* dated December 2022. Further details into this recommended water balance can be reviewed under this separate cover.

1 Background

Monument Geomatics and Estimating ("Monument") was retained by Mistral Land Development Inc. 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 31 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 draft plan was developed to reduce the building footprint within the forested areas and future development may be considered through future phases of the development. 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 (also referred to as the "Garden

Hill Pond") 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 below provides an overview of the subject property.

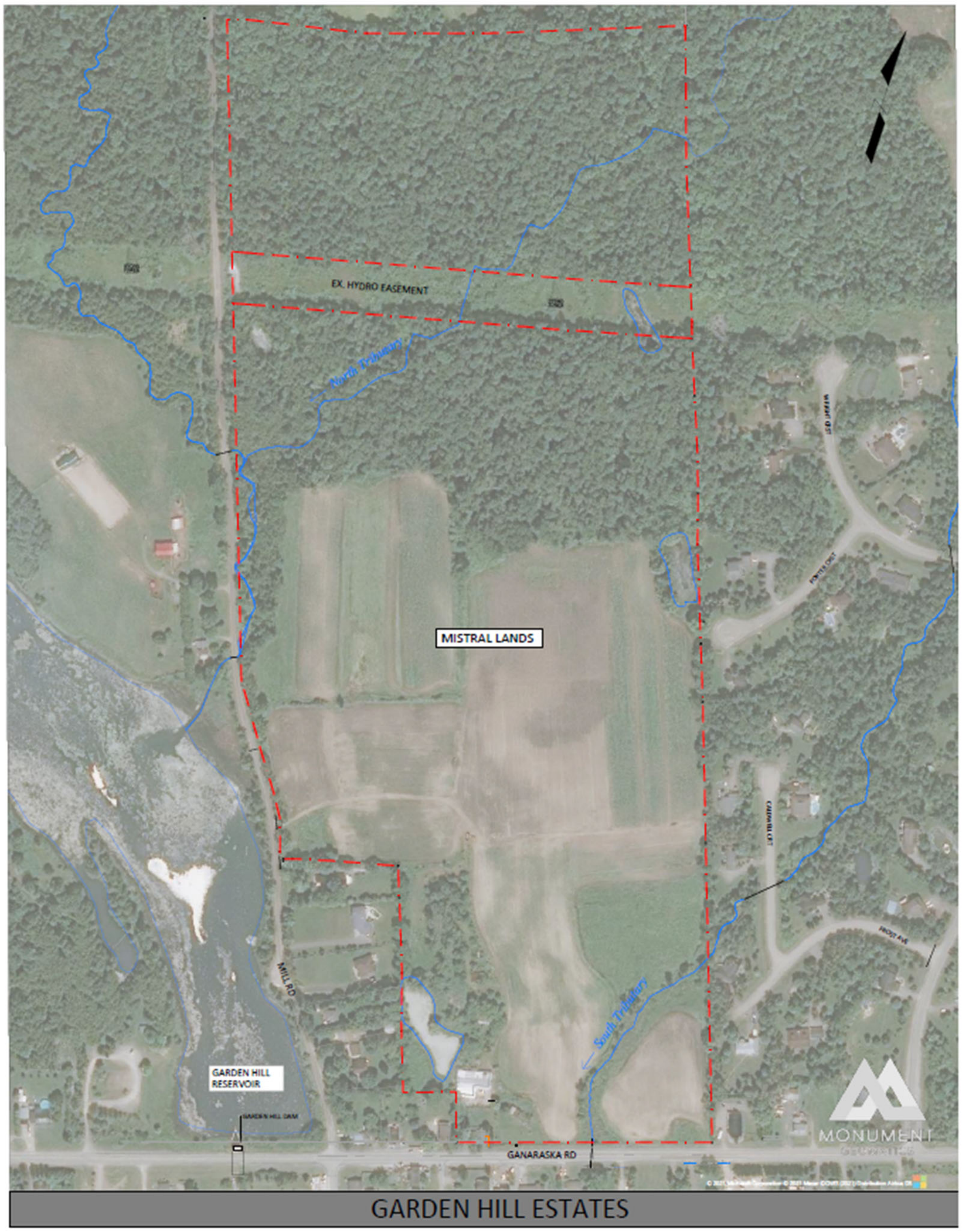


Figure: Site Overview

2 Proposed Development

The development will involve the construction of two rural roadways to allow frontage of 31 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 and be designed accordingly at the time through a Site Plan Control Application.

A draft plan of the development is provided in Appendix C illustrating the proposed lot fabric. Outside the lot fabric, Blocks 106 and 108 are designated as “Other Lands Owned by the Developer” and may be used for future development. These blocks include the lands just north of the development area and south of the hydro easement (Block 106) as well as the lands in the southwest corner of the development where the old farmer’s pond is located (Block 108). Regulated lands are sensitive areas including watercourses and wetlands onsite and their applicable setbacks. There are two stormwater management facilities proposed. One at the southeast (SWMF#1) corner and northwest (SWMF#2) portion of the site. At this stage it is unclear if the other lands owned by the development will be developed in the future, **therefore, the stormwater management plan will account for all Phase 1 and future development within Block 106 and 108.**

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. A Site Access Letter prepared by Monument dated April 22, 2022, provides additional details to support the site’s access locations.

Monument prepared a conceptual servicing plan (see Appendix D) to provide 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 cover. 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 from top of pond banks.

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 for the development will be provided for the 31 single-detached homes from an existing dry fire hydrant located on Mill Street within the Mill Pond. The location of this hydrant is approximately 450m from the Street ‘A’ access off Ganaraska Road. To provide a secondary emergency access, a 3.0m wide gated pathway is proposed within the north SWM block, 150m north of the existing dry hydrant. The location of the dry fire hydrant and proposed emergency access path is illustrated on the General Servicing Plan in Appendix D.

Fire suppression for the multi-unit residential block will also benefit from the dry hydrant with access off Frost Avenue. This block is subject to Site Plan Control which will require a servicing study to identify the fire supply requirements once the size of the building has been determined. The servicing study will review the need for internal sprinklered requirements and possibly an on-site cistern underground which could be feed from an additional well within the block.

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

The site's soil composition consists of a mix of HSG A Ponty Pool Sandyloam and HSG B Bondhead 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.

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 $A_{1,2,...}$
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 Garden Hill Pond. The contributing catchment areas onsite are 107 and 107B which both sheet flow to the North Tributary.

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 Pond 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 Mistral Lands, 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 Garden Hill Pond 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 resulting in an 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 from Wright Crescent to Caldwell Court. 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. All culvert crossings were modelled as a “Dummy conduit” to ensure the crossing is not assessed as a flood control structure. The steady flow routing method was also selected within the model.

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 7-2.

Table 7-2: Pre-development Peak Flows

Pre-development Peak Flows (m^3/s)		
Crossing	100-yr	Regional
Wright CRST	0.352	0.681
Caldwell CRT	0.642	1.339
U.S. Ganaraska RD	1.126	2.970
D.S. Ganaraska RD	1.126	2.970
N. Ganaraska River	1.453	3.268

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 of Monuments LIDAR survey and inputted into HEC-RAS. The remaining cross sections downstream were extracted from Ontario's Digital Elevation Model LIDAR data.

Figure 1: 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 7-2, were simulated at river stations 470, 271, 232 and 72, respectively. From the simulation the resulting WSELs at each river station is illustrated in Table 7-3 below.

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	177.11	177.60
436	176.55	177.60
409	176.30	177.60
384	176.05	177.60
340	175.47	177.60
304	175.50	177.60
291	175.50	177.60
283	175.50	177.60
271	175.47	177.60
254	Ganaraska Crossing	
237	174.35	174.55
222	174.14	174.33
204	173.36	173.54
152	172.33	172.49
109	171.60	171.74
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 upstream of Ganaraska Road crossing creating ponding within the development upstream to the neighboring property.

7.2.2 Post-development

Monument prepared a “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 7-2: 100-yr and Regional Post-development Peak Flows

Post-development Peak Flows (m^3/s)		
Crossing	100-yr	Regional
Wright CRST	0.352	0.681
Caldwell CRT	0.642	1.339
U.S. Ganaraska RD	1.126	3.097
D.S. Ganaraska RD	1.126	3.097
N. Ganaraska River	1.453	3.395

Note that the 100-yr peak flows were not adjusted since quantity control measures will maintain pre-development 100-yr flow levels.

Like the pre-development WSELs, Hurricane Hazel creates a large amount of ponding upstream of the crossing overtopping Ganaraska Road.

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

	Post-development	
River Station (m)	100-yr WSEL (m)	Regional WSEL (m)
477	177.11	177.60
436	176.55	177.60
409	176.30	177.60
384	176.05	177.60
340	175.47	177.60
304	175.50	177.60
291	175.50	177.60
283	175.50	177.60
271	175.47	177.60
254	Ganaraska Crossing	
237	174.35	174.57
222	174.14	174.34
204	173.36	173.55
152	172.33	172.49
109	171.60	171.75
72	170.71	171.39

Table 7-4: Comparison of Pre and Post Water Surface Elevations downstream of the Ganaraska Road Crossing River Stations.

River Station (m)	Regional WSELs (m)		
	Pre	Post	Increase
237	174.55	174.57	0.02
222	174.33	174.34	0.01
204	173.54	173.55	0.01
152	172.49	172.49	0
109	171.74	171.75	0.01
72	171.39	171.39	0

The results shown in Table 7-4, illustrate the increase in WSELs from Pre to Post-development. The Post-development conditions will result in a minimal increase in WSELs of 2cm. These WSELs have been plotted on the Post-development Floodline Drawing in Appendix H. Which 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, each event is held within the banks of the creek.

7.2.3 Proposed Culvert Replacement

Monument recommends that the culvert crossing at Ganaraska Road be replaced to reduce flooding upstream within the development under the Regional Storm.

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.

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 second submission, Monument proposed three culvert replacement options with design objective of having maximum flooding elevation of 176.50m in the Regional storm event as this is the maximum elevation to support development outside of the wetland setback. The proposed culvert options are as follows:

Option 1 **- New 1400mm diameter corrugated circular pipe. (300mm of river stone installed at the bottom of the culvert)**

Option 2 **- New 1200mm diameter corrugated steel pipe (150mm of river stone installed to line the bottom of culvert)**

Option 3 **- Existing 900mm diameter corrugated steel pipe + new 800mm corrugated steel pipe (Invert set 700mm higher than existing culvert)**

Note: Options 1 and 2 are modelled to ensure the top of the river stone matches the existing invert elevation.

Table 7-5: Comparison of WSELs of Proposed Options with Existing Culvert Crossing at R.S. 271

	Upstream WSEL (m)			
Return Period	**Ex. 900mm CSP**	**Option 1**	**Option 2**	**Option 3**
2-yr	174.87	174.82	174.79	174.87
5-yr	174.98	174.90	174.88	174.98
10-yr	175.17	174.95	174.95	175.06
25-yr	175.28	175.03	175.04	175.15
50-yr	175.38	175.09	175.11	175.22
100-yr	175.47	175.16	175.18	175.30
Regional	177.60	175.95	176.45	176.42

Based on these results Option 1 and 2 prove to be a more hydraulically efficient culvert crossing for the 2-100yr return period events. Therefore, an assessment of single culvert crossing configuration is provided in Table 7-6 which will be reviewed with the twin crossing configuration in Table 7-7.

Table 7-6: Assessment Matrix of a Single Culvert Replacement of Ganaraska Road Crossing

Assessment Single Culvert Crossing – Option 1 & 2		
No.	**Pros**	**Cons**
1	Provides sufficient capacity to reduce Regional Flooding to less than 176.50m.	Potential for nuisance flooding in normal storm events.

Option 3 was selected based on the recommendation from stakeholders. To meet the Regional WSEL below 176.50m, an 800mm diameter CSP culvert is required. The invert was set 700mm higher than the existing culvert invert to maintain normal flow through the crossing in the 2-5yr storm period events. The new culvert was modelled on the east side of the existing culvert offset with 0.45m spacing between the outside walls of the two culverts (1/2 span of existing culvert). An assessment matrix of a twin crossing configuration is provided in Table 7-7 below.

Table 7-7: Assessment Matrix of Twin Culvert Replacement Option of Ganaraska Road Crossing

Assessment Twin Culvert Crossing – Option 3		
No.	**Pros**	
1	Provides sufficient capacity to reduce Regional Flooding to less than 176.50m.	Widen overall span of the crossing. This will most-likely require widening of receiving banks on private property downstream.
2	The minor storms (2-5yr) will only be conveyed in the existing culvert. The second culvert would be engaged in the 10-yr storm and greater.	Twin culvert crossings create a weak point between the pipes and have a greater potential to be susceptible to erosion and culvert washout in major storm events.

Velocities through each culvert Option was also compared with the existing culvert configuration provided in the Table below. The results illustrate that all three options reduce velocities downstream in major storms which will help mitigate localized erosion. Option 3 will maintain the hydraulic condition of the existing culvert for the 2-yr and 5-yr flows as design. Option 2 will also maintain similar outlet velocities, while Option 1 will increase outlet velocities slightly (2-yr to 5-yr only).

Table 7-8: Comparison of Outlet Velocities between the Existing Configuration and Options 1 and 2

	Velocity at Downstream Node (m/s)			
Return Period	**Ex. 900mm CSP**	**Option 1**	**Option 2**	**Option 3**
2-yr	1.27	1.45	1.30	1.27
5-yr	1.51	1.59	1.49	1.51
10-yr	2.31	1.68	1.62	1.65
25-yr	2.44	1.80	1.81	2.17
50-yr	2.53	1.89	1.97	2.28
100-yr	2.61	1.99	2.10	2.58
Regional	3.84	2.97	3.28	3.06

Considering the results displayed above, the preferred option for the replacement is **Option 2.** Monument believes that maintaining one culvert configuration will have less impact to private property downstream and help to mitigate channel erosion at the inlet side under major storms. With the lined culvert bottom and smaller culvert size, this option also helps to maintain similar flows for the minor events which should minimize any potential for nuisance flooding downstream.

Further consideration on constructability and implementation will be completed at detailed design. Based on Option 2, the revised 100-yr and Regional Storm WSELs determined in HEC-RAS are provided in the Table below. These WSELs have also been plotted on the revised Floodplain drawing FL-3 in Appendix H.

Table 7-9: HEC-RAS Water Surface Elevations in Post-Development Condition with Culvert Replacement

	Post-Development (Option 2)	
River Station (m)	100-yr WSEL (m)	Regional WSEL (m)
477	177.1	177.23
436	176.56	176.56
409	176.29	176.50
384	176.06	176.50
340	175.44	176.50
304	175.23	176.50
291	175.23	176.50
283	175.23	176.50
271	175.18	176.45
254	Ganaraska Crossing	
237	174.35	174.57
222	174.14	174.34
204.0	173.36	173.55
152.0	172.33	172.49
109.0	171.6	171.75
72.0	170.71	171.39

As illustrated in the results above, the replacement option drastically lowers WSELs upstream and eliminates the extent of backwater effect to only 140m within the reach onsite. The max flood elevation at the culvert crossing is 176.50m which is now 1.06m below Ganaraska Road sag, eliminating water from overtopping the road.

Monument believes that the recommendation of upsizing the culvert would benefit the development, the County Road 9, and eliminate potential risk of flooding of the upstream neighbors. Therefore, the proposed conditions discussed herewithin will be based on the 100-yr and Regional flood elevations generated with the culvert replacement.

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 excerpt of the HEC-RAS cross section with the obstruction in place.

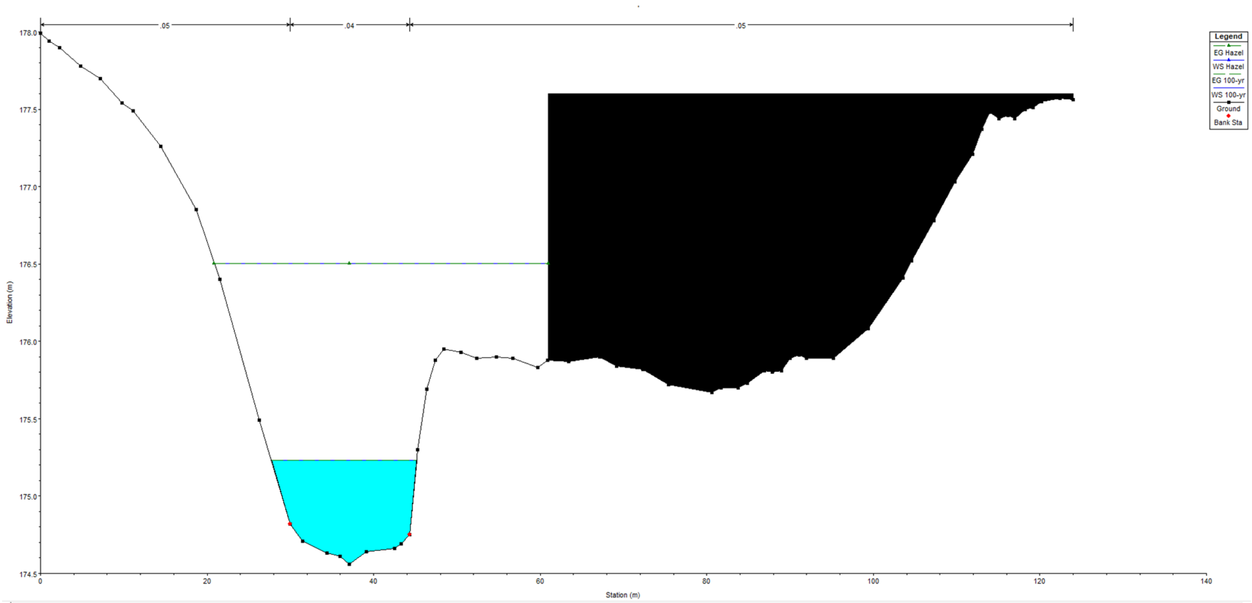


Figure 2: HEC-RAS Rivers Station 291 with User Defined Obstruction

The results showed no change in WSELs and therefore is only occupying ineffective flow areas or potential storage areas. Monument completed a cut and fill analysis in Civil3D to determine the storage lost in the Regional event. The analysis determined a loss of approximately 485m^3 of storage could be loss with the advancement of the roadway. Therefore, is proposed to be reinstated alongside the stormwater management facility between the wetland boundaries. Further details will be provided at the design stage of the SWM facility.

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	105
Outlet Point	OF1	OF1	OF1	OF1	OF4
Area	7.00	1.23	1.43	1.49	1.37
Flow Length	540	100	255	131	219
Slope	2.5%	2.0%	3.1%	4.1%	4.7%
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	0.06
Curve Number	72	64	46	67	82

Name	106	107	107B
Outlet Point	OF3	OF2	OF2
Area	2.70	9.27	2.51
Flow Length	267	300	177
Slope	4.5%	2.3%	1.1%
Percent Impervious (%)	0	0	0
N Impervious	0.013	0.013	0.013
N Pervious	0.06	0.13	0.13
Curve Number	82	56	30

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 (m^3/s)				
	OF1	OF2	OF3	OF4	Garden Hill Pond
SCS Type II					
24hr	**0.602**	**0.122**	**0.382**	**0.212**	0.706
12hr	0.450	0.084	0.323	0.180	0.566
6hr	0.407	0.086	0.246	0.130	0.449
Chicago					
24hr	0.377	0.093	0.294	0.168	0.497
12hr	0.291	0.072	0.231	0.135	0.379
6hr	0.259	0.058	0.213	0.125	0.346
Note: Garden Hill Pond peak flows are addition of OF2 + OF3 + OF4					

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. **Note: The Post-development Catchment Area Drawing illustrates future development layout from previous draft plan submissions within Block 106 and 108. These blocks will be modelled according to this layout in order to account for future development.** As illustrated on this plan, Monument determined a total of 9 different post-development catchment areas draining to the four different Outfall locations. A preliminary grading plan has also been provided in Appendix J to support each catchment.

Catchment areas 200, 201, 202 and 203 will drain to Outfall #1.

Catchment 200

Delineates the rear yards of lots 30 and 31 as well as a portion of the "Other Lands Owned by the Developer". It also contains the small external area draining into the old farmers pond shown on the Existing Catchment Area Drawing. Lot 30 will be constructed with a rear yard swale draining around to Street A. If the existing pond is to remain, this rear yard swale will be constructed behind as a spill point from the pond to maintain the existing drainage pattern. If the existing pond is removed, a rear yard drainage swale will be extended from Lot 30 up to Lot 31. This are will be left uncontrolled in the post-development conditions.

Catchment 201

Contains majority of Street 'A' and the front yards draining towards the right-of-way. This area will drain within the roadside ditches directly to the first stormwater management facility (SWMF#1) at the entrance to the south.

Catchment 202

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 1 to 8. Runoff from the rear of these lots will also contribute and be released uncontrolled.

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. Quality control devices will need to be reviewed at the time of site plan control.

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

Catchment 300A

Catchment 300A contains majority of Street 'B' with a high point to be constructed at the south end to direct drainage northwest to SWMF#2 to maintain existing drainage patters. Rear yard swales will split behind lots 15 and 19 to direct drainage into the roadside ditch and drainage easement within Block 107.

Catchment 300B

This catchment includes the "Other Lands Owned by the Developer" where the original proposal was to include 9 additional lots within the environmental sensitive lands fronting a cul-de-sac south of the hydro easement. Based on stakeholder feedback, lots are not permitted within these lands until further environmental surveys are complete to support development. However, to account for the future development, this catchment has been modelled using the full-build out conditions as noted on the Post-development Catchment Area Drawing.

To maintain existing drainage patterns 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 within Block 107 easement. This swale will tie into the ditches along Street 'B' and then into the second SWMF ("SWMF#2").

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 OF #3 and OF #4 have been reduced and runoff will drain uncontrolled. A rear yard swale starting behind Lot 29 and continue along the perimeter of the property to lot 27 will control runoff directly to the existing 400mm CSP at OF#4 at Mill Street.

Catchment 600

Catchment 600 contains the north portion of the site that will contain the rear roof tops of Lots 21 and 22 and future rear yards of Lot 35 to 38. Quantity control will not be provided for the minor impervious rooftop area draining to Outfall #2.

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).

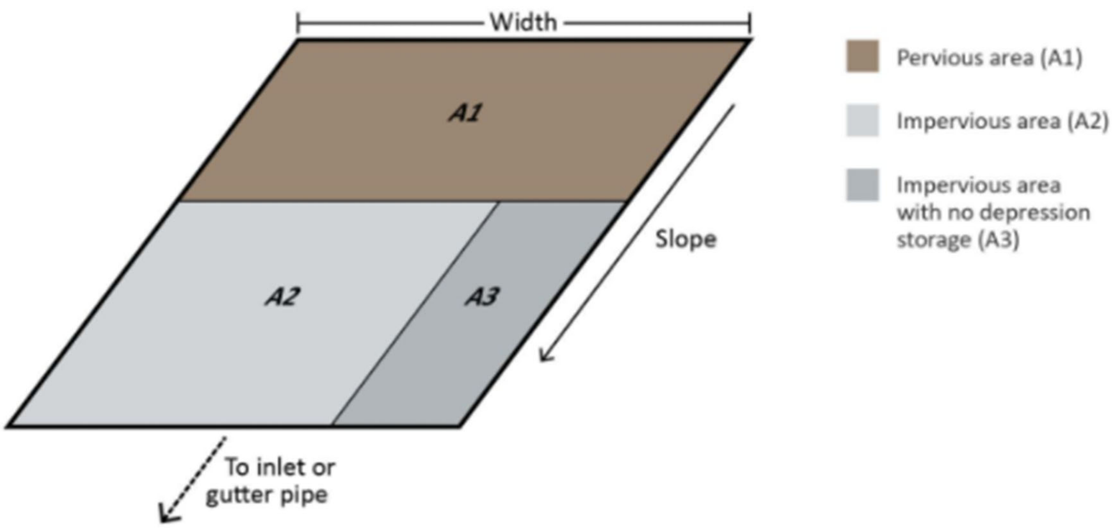


Figure 3: PCSWMM Support Excerpt describing Subarea Routing

(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.

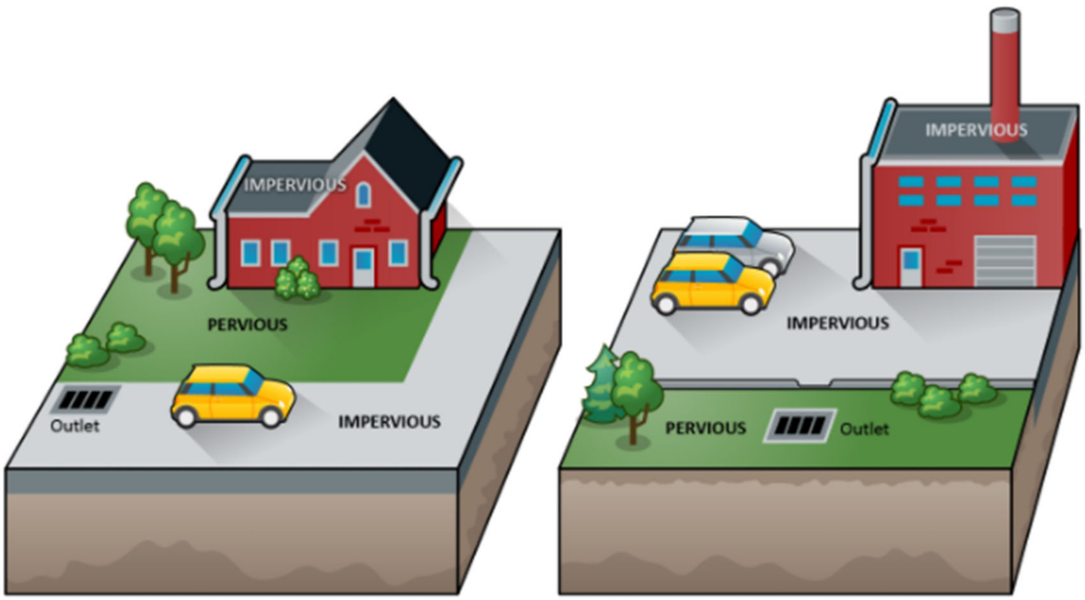


Figure 4: Excerpt from PCSWMM Support website illustrating Subarea Routing

(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 8-3 provides the selected parameters used for these catchments. More details are provided in the Methodology Section of the Appendix.

Table 8-3: Post-development PCSWMM Input Summary

Name	200	201	202	203
Outlet Point	OF1	OF1	OF1	OF1
Area	1.82	4.45	2.95	1.23
Flow Length	100	150	120	100
Slope	2%	3.5%	4%	2%
Percent Impervious (%)	6.5%	31.0%	3.9%	0.0%
N Impervious	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.06
Subarea Routing (%)	80%	30%	80%	-
Curve Number (AMCII)	60	73	72	72

Name	300A	300B	400	500	600	107B
Outlet Point	OF3	OF3	OF4	OF3	OF2	OF2
Area (ha)	5.44	2.48	0.78	0.66	4.67	2.51
Flow Length (m)	140	66	100	75	240	177
Slope	2.5%	2.0%	2.0%	3.0%	2.3%	1.1%
Percent Impervious (%)	16.8%	29.4%	15.6%	4.4%	1.7%	0
N Impervious	0.013	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.25	0.3	0.13
Subarea Routing	30%	30%	80%	80%	80%	-
Curve Number (AMCII)	75	75	78	78	56	30

Site Imperviousness

Based on the anticipated lot grading, Monument used the proposed drainage plan to determine an impervious area for each catchment. Catchments that include "other lands owned by developer" are considered as developed to account for potential future development. Imperviousness for these areas is calculated based on the site layout from previous draft plan submissions. As illustrated in Table 8-4, it is anticipated that the overall site will have a percent impervious of 16%.

Table 8-4: Estimated Site Imperviousness within each Catchment Area

Catchment	Area (ha)	Impervious Area	Impervious
200	1.83	0.12	6.5%
201	4.46	1.38	31.0%
202	2.95	0.11	3.9%
203	1.23	0.31	25.0%
300A	5.44	0.91	16.8%
300B	2.48	0.73	29.4%
400	0.78	0.12	15.6%
500	0.66	0.03	4.4%
600	4.67	0.08	1.7%
Total:	**24.51**	**3.79**	**15.5%**

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 8-5.

Table 8-5: Post-Uncontrolled Peak Flows to each Outfall

Duration	Uncontrolled Peak Flows (m^3/s)				
	OF1	OF2	OF3	OF4	Garden Hill Pond
SCS Type II					
24hr	0.898	0.046	0.915	0.075	1.031
12hr	0.708	0.032	0.762	0.061	0.848
6hr	0.570	0.033	0.564	0.055	0.645
Chicago					
24hr	0.630	0.037	0.653	0.053	0.719
12hr	0.513	0.029	0.500	0.042	0.551
6hr	0.485	0.024	0.467	0.038	0.512
Note: Garden Hill Pond peak flows are addition of OF2 + OF3 + OF4					

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 8-6 and 8-7). 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 Garden Hill Pond instead of matching pre-dev rates for each outlet. Outlet conveyance capacity is reviewed in Section 12 to confirm the balanced flow rates across the 3 outlets can safely be conveyed to the Garden Hill Pond.

Table 8-6: Pre-development Peak Flow Comparison to Outfall #1 and #2

Duration	OF1		OF2	
	Pre	Post	Pre	Post
SCS Type II				
24hr	**0.602**	0.898	0.122	0.046
12hr	**0.450**	0.708	0.084	0.032
6hr	**0.407**	0.570	0.086	0.033
Chicago				
24hr	**0.377**	0.630	0.093	0.037
12hr	**0.291**	0.513	0.072	0.029
6hr	**0.259**	0.485	0.058	0.024

Table 8-7: Pre-development Peak Flow Comparison to Outfall #3 and #4

Duration	OF3		OF4		Reservoir	
	Pre	Post	Pre	Post	Pre	Post
SCS Type II						
24hr	0.382	0.915	0.212	0.075	0.706	1.031
12hr	0.323	0.762	0.180	0.061	0.566	0.848
6hr	0.246	0.564	0.130	0.055	0.449	0.645
Chicago						
24hr	0.294	0.653	0.168	0.053	0.497	0.719
12hr	0.231	0.500	0.135	0.042	0.379	0.551
6hr	0.213	0.467	0.125	0.038	0.346	0.512
Note: "Garden Hill Pond" peak flows is a simple addition of OF2 + OF3 + OF4						

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 8-8: Storage Requirement Estimate to each Outfall

Duration	Storage Volume (m^3)	
	OF1	Garden Hill Pond
SCS Type II		
24hr	443	427
12hr	401	372
6hr	486	**560**
Chicago		
24hr	**569**	444
12hr	564	444
6hr	514	378
Note: Garden Hill Pond peak flows are addition of OF2 + OF3 + OF4		

The greatest storage requirements for Outfall #1 were estimated in the 24hr Chicago storm requiring 569m^3 of active storage. At the Garden Hill Pond, the greatest storage will be generated in the 6hr SCS Type II storm with 560m^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 8-9: Pre-development Release Rates

Duration	Target Flows (m^3/s)	
	OF1	Garden Hill Pond
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:
- Garden Hill Pond peak flows are simple addition of OF2+OF3+OF4
- 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 SWM facility #1 be an extended detention wet pond. The roadside ditches will inlet in two locations. The major inlet is to the north where the culvert crossing converges the flow in both ditches to outlet by gravity into the facility. The bottom of pond elevation will be set at 175.20m with a 1.0m permanent pool depth up to an elevation of 176.20m. The bottom of pond will require a clay liner in order to hold the permanent pool. Above the permanent pool, the quality storage and quantity storage will be provided within an active storage depth of 0.7m to an elevation of 176.90m. A minimum freeboard depth of 0.30m will be provided above that to meet the top of berm elevation of 177.20m. 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 from the conceptual layout illustrated on the Post-development Catchment Area drawing. 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 1100m^3 with an additional storage volume of 629m^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 the wet pond facility is to meet a TSS removal efficiency of 80% (Enhanced - Level 1) as per Table 3.2 of the MECP 2003 SWM Guideline. Table 3.2 provides a quick calculation to determine a required permanent pool and extended detention storage based on the impervious area and a 24hr drawdown time. The total contributing area to the pond is 4.44ha (Catchment 201) with a total impervious area of 1.38ha. This results in the contributing area to have a total %imperviousness of 31%. However, runoff from rooftops will not be credited in this calculation and deducted from the impervious treatment area. The remaining area is 1.05ha with an adjusted %impervious of 24% to be used in Table 3.2.

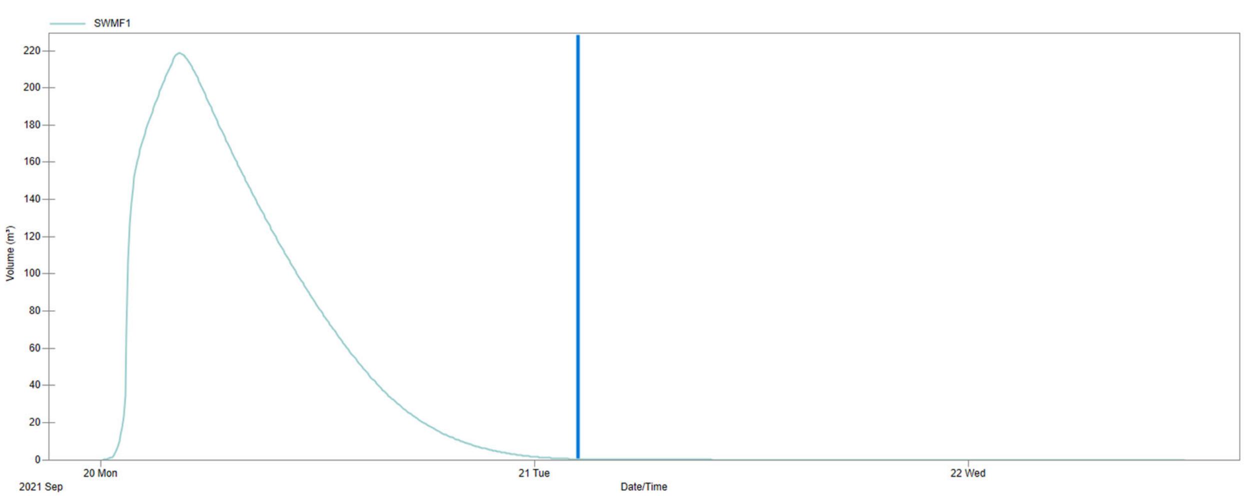
Table 3.2 does not have a pre-determined multiplication factor for impervious levels less than 35%. Therefore, Monument extrapolated a value of 112.5m^3/ha of storage from the 35% and 55% assigned factors. Multiplied by the contributing area (4.44ha,) the required permanent pool volume is 322m^3 while the remaining 177m^3 is to be extended detention.

As illustrated in the Stage-Storage-Discharge Relationship in Appendix K, the facility will have a permanent pool volume over 907m^3 and extended detention volume of 411m^3. Therefore, these storage volumes are greater than the required volumes for enhanced level 1 efficiency. A forebay is also required at the inlet of the wet pond to reduce velocity of inflow and capture suspending solids from reaching the outlet. The forebay design will be provided at the time of detailed design.

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. This is typically achieved by creating a two-stage outlet structure from the pond using an orifice as the quality control structure. This was modelled in PCSWMM using the weir and orifice outlets sized corresponding to the stage-storage-discharge sheet. To meet the drawdown time requirement the minimum orifice size of 75mm was selected. This results in an approximate 26.5hr drawdown time, 0.23m max depth and storage volume of 218.7m^3 in the event of quality storm. Therefore, the drawdown time and storage volume targets are satisfied. An excerpt of the quality storm routed through the pond storage volume from PCSWMM graph tab is provided below.

Figure 5: PCSWMM Excerpt of Quality Storm Routed through SWMF#1



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 176.60m 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: Function of SWMF#1 under each return period event

Duration	Inflow to Pond (m^3/s)	Storage (m^3)	Max Depth (m)	WSEL (m)
2-yr	0.152	396	0.35	176.55
5-yr	0.218	480	0.40	176.60
10-yr	0.269	538	0.44	176.64
25-yr	0.344	618	0.49	176.69
50-yr	0.404	677	0.53	176.73
100-yr	0.468	740	0.57	176.77

Table 9-2: SWMF#1 Pond Release Rates and Quantity Control Check

Duration	Pond Outflow (m^3/s)	Combined Controlled OF1 Controlled Flow (m^3/s)	OF1 Target Flows (m^3/s)	Achieved Quantity Control
2-yr	0.018	0.025	0.046	✓
5-yr	0.042	0.062	0.102	✓
10-yr	0.061	0.097	0.153	✓
25-yr	0.090	0.154	0.236	✓
50-yr	0.113	0.203	0.302	✓
100-yr	0.139	0.260	0.376	✓

An overall review of the ponds function under each event is summarized in Table 9-2. As illustrated from the controlled OF#1 column, which is less than the pre-development target flow, quantity control is achieved. **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 (176.90m). 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 $0.621m^3/s$. 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 0.13m and overflow over weir of $0.27m^3/s$.

9.2 Stormwater Management Facility #2

The second SWMF is proposed to be an extended detention wet pond as well. This allows 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 7.92ha with a percent impervious of 21% 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 permanent pool be set at an elevation of 178.70m. The top of active storage will be set to 179.40m which is 0.20m lower than the top of berm elevation of 179.70m. 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 180.30m. 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 758m^3 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 as previously described above. The total contributing area to the pond is 7.92ha (Catchment 300A + 300B) with a total assumed impervious area of 1.64ha. This results in the contributing area to have a total %imperviousness of 21%. However, runoff from rooftops will not be credited in this calculation and deducted from the impervious area. The remaining impervious area is 1.08ha with an adjusted %impervious of 14% to be used in Table 3.2.

Monument extrapolated a value of 87.5m^3/ha of storage from the 35% and 55% assigned factors from MECP Table 3.2. Multiplied by the contributing area (7.92ha) the 317m^3 shall be extended detention (40m3/ha) while the remaining 376m3 is permanent pool.

As illustrated in the Stage-Storage-Discharge Relationship in Appendix K, the facility will have a permanent pool volume over 757m^3 and extended detention volume of 366m^3. Therefore, these storage volumes are greater than the required volumes for enhanced level 1 efficiency. A forebay is also required at the inlet of the wet pond to reduce velocity of inflow and capture suspending solids from reaching the outlet. The forebay design will be provided at the time of detailed design.

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 28hr was achieved as displayed in the PCSWMM excerpt in Figure 8.

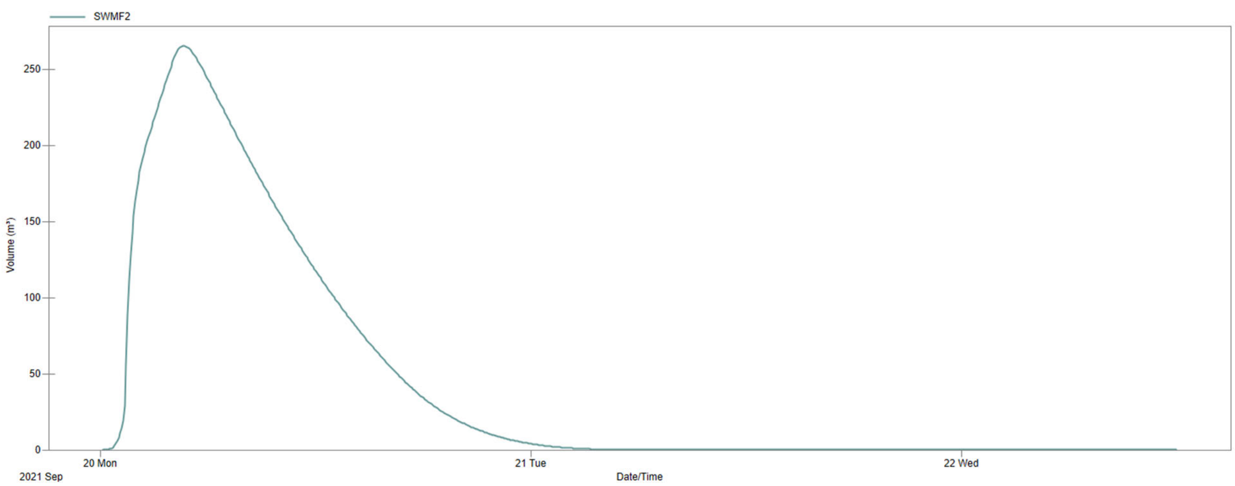


Figure 6: PCSWMM Excerpt of Quality Event through SWMF#2

The results showed that the total storage volume used in this event was 280m^3 with a maximum depth of 0.30m. Therefore, SWMF#2 provides the 24hr drawdown time and meets the required quality storage volumes.

Monument proposed that the second outlet be designed using a 0.85m long sharp-crested weir set to the invert of 179.0m. 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 Garden Hill Pond.

Table 9-3: Function of SWMF#2 under each return period event using the SCS Type II 6-hr Storm.

Duration	Inflow to Pond(m^3/s)	Storage (m^3)	Max Depth (m)	WSEL (m)
2-yr	0.118	402	0.37	179.17
5-yr	0.172	503	0.44	179.24
10-yr	0.238	580	0.49	179.29
25-yr	0.396	687	0.56	179.36
50-yr	0.436	769	0.61	179.41
100-yr	0.550	857	0.67	179.47

Table 9-4: SWMF#2 Pond Release Rates and Quantity Control Check

Duration	Pond Outflow (m^3/s)	Combined Controlled Flow to Garden Hill Pond (m^3/s)	Target Flow (m^3/s)	Achieved Quantity Control
2-yr	0.033	0.040	0.071	✓
5-yr	0.080	0.099	0.143	✓
10-yr	0.124	0.159	0.201	✓
25-yr	0.190	0.244	0.291	✓
50-yr	0.244	0.317	0.365	✓
100-yr	0.303	0.399	0.449	✓

The results in Table 9-4 illustrate the success of controlling peak flows from the development which results in a combined controlled peak flow (OF#2 Un-controlled + OF#3 Controlled + OF#4 Un-controlled) to the Garden Hill Pond. **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 1.09m^3/s. The emergency spillway will be set to the top of active storage (179.40m) and have a 3.0m weir length. The model resulted in the weir being engaged in this event with a total depth of 0.16m over the invert and discharge of 0.55m^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 Garden Hill Pond 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 (m^3/s)	HW Depth (m)	WSEL (m)	Inflow (m^3/s)	Depth (m)	WSEL (m)
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	0.14	0.39	174.00
25-yr	0.17	0.45	174.06	0.21	0.59	174.20
50-yr	0.21	0.58	174.19	0.27	Overtop	-
100-yr	0.25	Overtop	-	0.33	Overtop	-

The results illustrate that controlled flow directed from the SWMF#2 to OF#3 culvert crossing will increase peak flows slightly compared to the pre-development flows. However, will not significantly impact the crossing and therefore will not need replacement.

10 Water Balance

Developments typically result in increased surface hardening that may lead to greater surface runoff and reduced infiltration rates. The GRCA raised concerns about potential impacts to the groundwater recharge and discharge rates of the site. A water balance was conducted to estimate the current infiltration rates to the subsurface and determine how much this rate is anticipated to change as a result of the proposed development.

Monument used the Thornthwaite-Mather monthly water balance method for determining the amount of moisture surplus available for infiltration. Inputs to the model include precipitation and outputs to the model include potential evapotranspiration (PET), actual evapotranspiration (AET), runoff, and infiltration. The difference between water entering the system (precipitation) and the water leaving the system (evapotranspiration) is the water surplus which remains in the system as either runoff or infiltration. The components of the monthly water balance are discussed in the Water Balance Methodology and Calculations Section in Appendix 0.

10.1.1 Pre-Development Infiltration

The site was divided into two catchment areas to reflect the different cover types which influence evapotranspiration and infiltration rates. A catchment area drawing is provided in the Appendix. The soil water holding capacities and total infiltration factors estimated using Table 3.1 of the MOE Stormwater Management Guidelines based on topography, soil type, and vegetation cover. The infiltration factors are multiplied by the moisture surplus to obtain the infiltration rate for the site while the remainder is considered to be runoff. Table 10-1 below illustrates the characteristics of each catchment and the assigned infiltration factors and soil moisture storages.

Table 10-1: Water Balance Catchment Area Values

Catchment Area	HSG Type	Vegetation Type	Soil Water Holding Capacity (mm)	Infiltration Factor
100	A	Mature Forest	250	0.8
101	A	Moderately Rooted	75	0.7

An annual water balance was conducted for the pre-development conditions following the method outlined by *Conservation Authority Guidelines for Hydrogeological Assessments* (2013). The annual average precipitation data and AET calculated in the monthly water balance were used as inputs for the annual water balance. The annual precipitation rate for this area is 866.4mm from Environment Canada data. A summary of the predevelopment water balance is provided in Table 10-2 below. The full water balance is provided in Appendix O.

Table 10-2: Pre-development Water Balance Summary

	Volume (m^3/yr)
Precipitation	317,189.0
Actual Evapotranspiration (AET)	199,055.9
Water Surplus	118,133.2
Infiltration	**88,176.2**
Runoff	29,956.9

The infiltration rate for each catchment was multiplied by the area and summed to obtain a total infiltration volume of **88,176.8m^3/yr** for the entire site in the existing conditions.

10.1.2 Post-Development Infiltration

Monument conducted an annual post-development water balance to determine how the infiltration rate would change as a result of the proposed development. The subcatchment areas used to calculate the pre-development infiltration volume were subdivided into pervious and impervious areas to reflect differences in evapotranspiration rates. Transpiration does not occur on impervious surfaces since there is no vegetative cover therefore only an evaporation rate is applied to these areas. The *Conservation Authority Guidelines for Hydrogeological Assessments* suggests an evaporation rate of 10-20% is acceptable for most impervious areas. To be conservative it was assumed there would be a 20% precipitation loss to evaporation on impervious areas.

The infiltration rate and AET rate calculated in the monthly water balance from section 9.1.1 were used for the post-development water balance. No infiltration occurs on impervious areas, so all moisture surplus goes to runoff, therefore the infiltration rate for these areas is 0 and the runoff coefficient is 1. A summary of the post-development water balance is provided in Table 10-3 below. The full water balance is provided in Appendix O.

Table 10-3: Post-development Water Balance Summary

	Volume (m^3/yr)
Precipitation	317,189.0
Actual Evapotranspiration (AET)	185,501.1
Water Surplus	131,687.9
Infiltration	**78,937.0**
Runoff	52,750.9

The infiltration volume in post-development conditions due to the increased imperviousness reduces the total infiltration volume to **78,937m^3/yr** for the entire site. Therefore, the post development conditions with no mitigation measure would decrease the infiltration volume of the site by 9,239m^3/yr. With no mitigation measures recharge to regional groundwater flow system and interflow within the shallow unsaturated zone would be expected to decrease over time.

10.1.3 Post-Development Infiltration with Mitigation

An annual water balance with enhanced infiltration was modelled for the post development conditions. In the enhanced infiltration scenario, the impervious catchment is subdivided into roofs and paved. It is assumed that 100% of runoff from roofs is redirected to impervious surface by redirecting roof leaders to lawns. Runoff from paved surfaces enter the enhanced grass swales. It is assumed that 20% of runoff from paved surfaces is directed to enhanced infiltration as per the CVC LID SWM Guideline which states a conservative runoff reduction estimate for enhanced grass swales in HSG soil group A/B is 20%. In the water balance runoff that is not redirected to enhanced infiltration is considered “adjusted runoff”. A summary of the post-development water balance is provided in Table 10-4 below. The full water balance is provided in Appendix O.

Table 10-4: Post-Development Water Balance with Mitigation Measures

	Volume (m^3/yr)
Precipitation	317,189.0
Actual Evapotranspiration (AET)	185,501.1
Water Surplus	131,687.9
Infiltration	91,064.1
Runoff	40,623.8

The enhanced infiltration will increase the infiltration volume by 12,127m^3/yr which would bring the total infiltration volume of the site to **91,064m^3/yr**. Thus, the total post-development with mitigation infiltration volume will be greater than the pre-development infiltration volume.

After further correspondence with Stakeholders through a second submission of the Draft Plan of Subdivision, further analysis from a hydrogeological standpoint was conducted to look at recharge over the site on a cell-by-cell basis to determine areas where development has decreased groundwater recharge. This analysis can further be reviewed in the document to titled *GRCA Hydrogeology Comment Response* prepared in December of 2022.

The response letter recommends that a more detailed water balance be completed at a representative lot-basis; supplementary to the water balance conducted herewithin, to ensure ground water recharge around ecological sensitive areas (i.e., wetlands) meet pre-development levels within the zone of influence of these features. At this stage, a lot-based analysis is pre-mature for draft plan approval and is better completed at the time of detailed design once the house locations, driveways, and other drainage features are advanced through design and therefore has not been completed within this submission.

11 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.

11.1 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 11-1: 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

11.2 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 11-2: 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.

11.3 Design Matrix

The Table below describes the proposed LIDs for the development.

Table 11-3: 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%	Road & Driveways
3)	End-of-system	Dry Pond & Wet Pond	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 SWMF#1. With combined treatment, the last treatment efficiency is conservatively reduced to account for water that is cleaned in the pre-treatment device. Therefore, the wet pond will have a reduced theoretical efficiency of 40% 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 90%.

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 94% overall.

Figure 11-4 below illustrates the combined removal efficiency of the proposed treatment train approach for each of the treatment areas.

Table 11-4: Quality treatment Design Matrix

Volume Untreated	Volume Treated

<table>
<tr><td colspan="7">201</td></tr>
<tr><td colspan="7">263.1 m^3 Precipitation (10,525 m^2 x 0.025m)</td></tr>
<tr><td colspan="4">Driveway (5,769m^2)</td><td colspan="3">Roadway (4,756m^2)</td></tr>
<tr><td colspan="4">144.3m^3</td><td colspan="3" rowspan="3">118.9m^3</td></tr>
<tr><td colspan="4">Vegetated Buffer Strip 40% TSS Removal</td></tr>
<tr><td colspan="3">86.5m^3</td><td>57.7m^3</td></tr>
<tr><td colspan="7">Enhanced Grass Swale 50% TSS Removal</td></tr>
<tr><td colspan="2">43.3m3</td><td colspan="2">101.0m^3</td><td>59.5m^3</td><td colspan="2">59.5m^3</td></tr>
<tr><td colspan="7">Dry Pond and Wet Pond 80% TSS Removal</td></tr>
<tr><td rowspan="2">8.6m^3</td><td colspan="3">135.6m^3</td><td colspan="2">107.0m^3</td><td rowspan="2">11.9m^3</td></tr>
<tr><td colspan="5">242.6m^3</td></tr>
</table>

<table>
<tr><td colspan="7">300A + 300B</td></tr>
<tr><td colspan="7">266.8 m^3 Precipitation (10,671 m^2 x 0.025m)</td></tr>
<tr><td colspan="4">Driveway (5,565m^2)</td><td colspan="3">Roadway (5,106m^2)</td></tr>
<tr><td colspan="4">139.1m^3</td><td colspan="3" rowspan="3">127.7m^3</td></tr>
<tr><td colspan="4">Vegetated Buffer Strip 40% TSS Removal</td></tr>
<tr><td colspan="3">83.5m^3</td><td>55.7m^3</td></tr>
<tr><td colspan="7">Enhanced Grass Swale 50% TSS Removal</td></tr>
<tr><td colspan="2">41.8m3</td><td colspan="2">97.5m^3</td><td>63.8m^3</td><td colspan="2">63.8m^3</td></tr>
<tr><td colspan="7">Dry Pond and Wet Pond 80% TSS Removal</td></tr>
<tr><td rowspan="2">8.4m^3</td><td colspan="3">130.9m^3</td><td colspan="2">114.9m^3</td><td rowspan="2">12.8m^3</td></tr>
<tr><td colspan="5">245.8m^3</td></tr>
</table>

The proposed treatment train approach provides a total TSS removal of 92% for both treatment areas. This meets the quality control design objective of 80%. TSS removal for enhanced treatment outlined in Section 4.

12 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 = 0.823m^3/s (Catchment 201)
SWMF #2: Regional Inflow = 1.09m^3/s (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 300A to 300B 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

Further review of each culvert crossing will be required at time of detailed design.

13 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.

14 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.

15 Conclusion

Monument Geomatics and Estimating ("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 31 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 31 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 replaced 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 three new replacement options to improve the hydraulic efficiency of the crossing and reduced upstream WSELs. The results from each option selected, drastically reduced the flooding within the development to meet the desired design objective. Within this revised report, Option 2 is stated to be the desired configuration. However, after review with stakeholders, Option 3 is the preferred configuration. At this time, Section 7 has not been updated since it demonstrates the feasibility of each Option to support development and therefore, will be carried out at the time of detailed design. Floodplain Drawings FL-3 and FL-4 will also be updated at this time.

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). Both facilities were selected to be extended detention wetponds. 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 80% 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 as well. 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 12.

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.

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Reviewed by:

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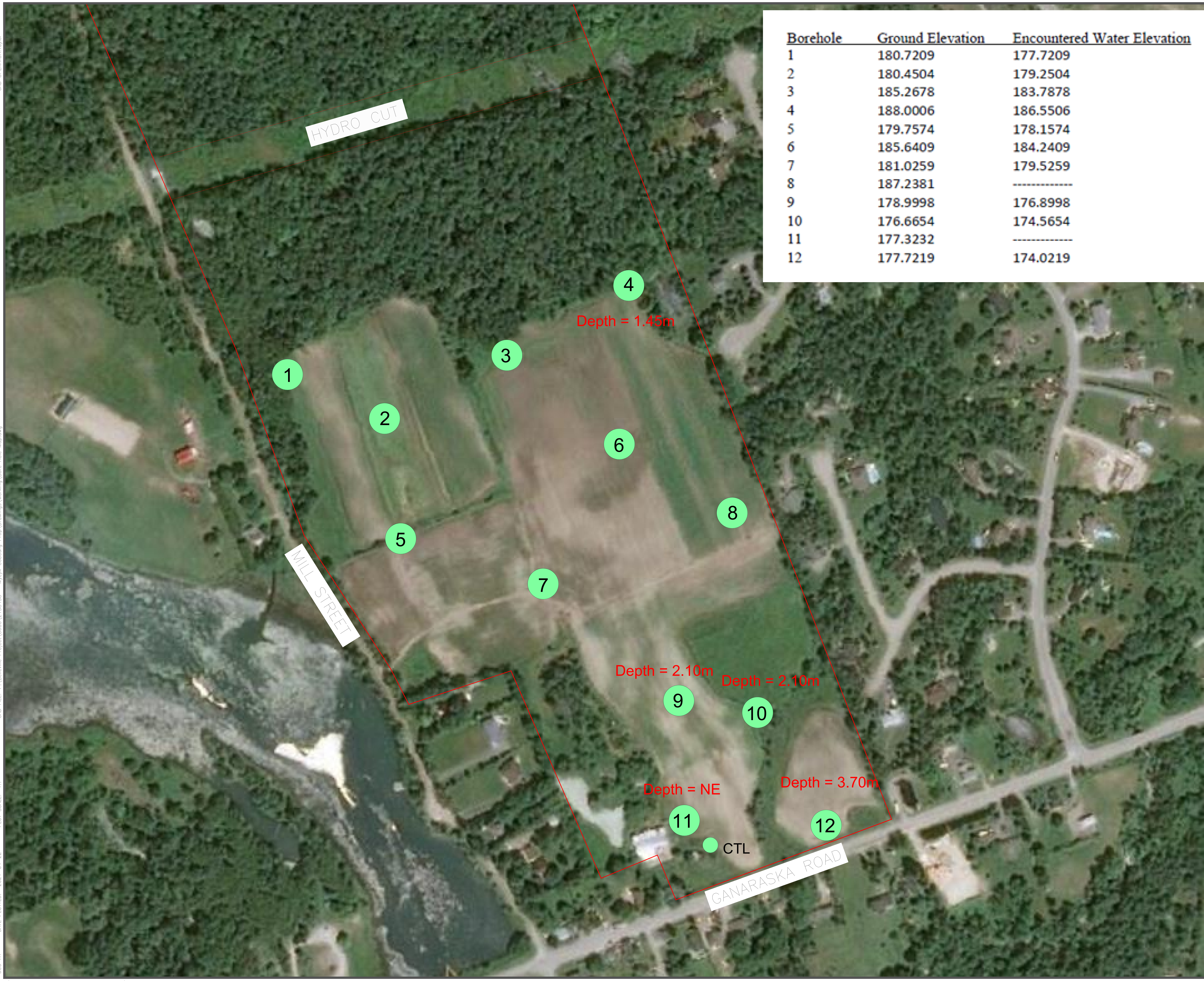
Appendix A – Borehole Log Excerpt



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

THE GREER GALLOWAY GROUP INC.
ENGINEERS & PLANNERS
PETERBOROUGH
BELLEVILLE
KINGSTON
1620 WALLBRIDGE LOYALIST ROAD
BELLEVILLE, ONTARIO, K8N 4Z5
PHONE: 613-966-3068
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 — A DETAIL NO.
B — 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)

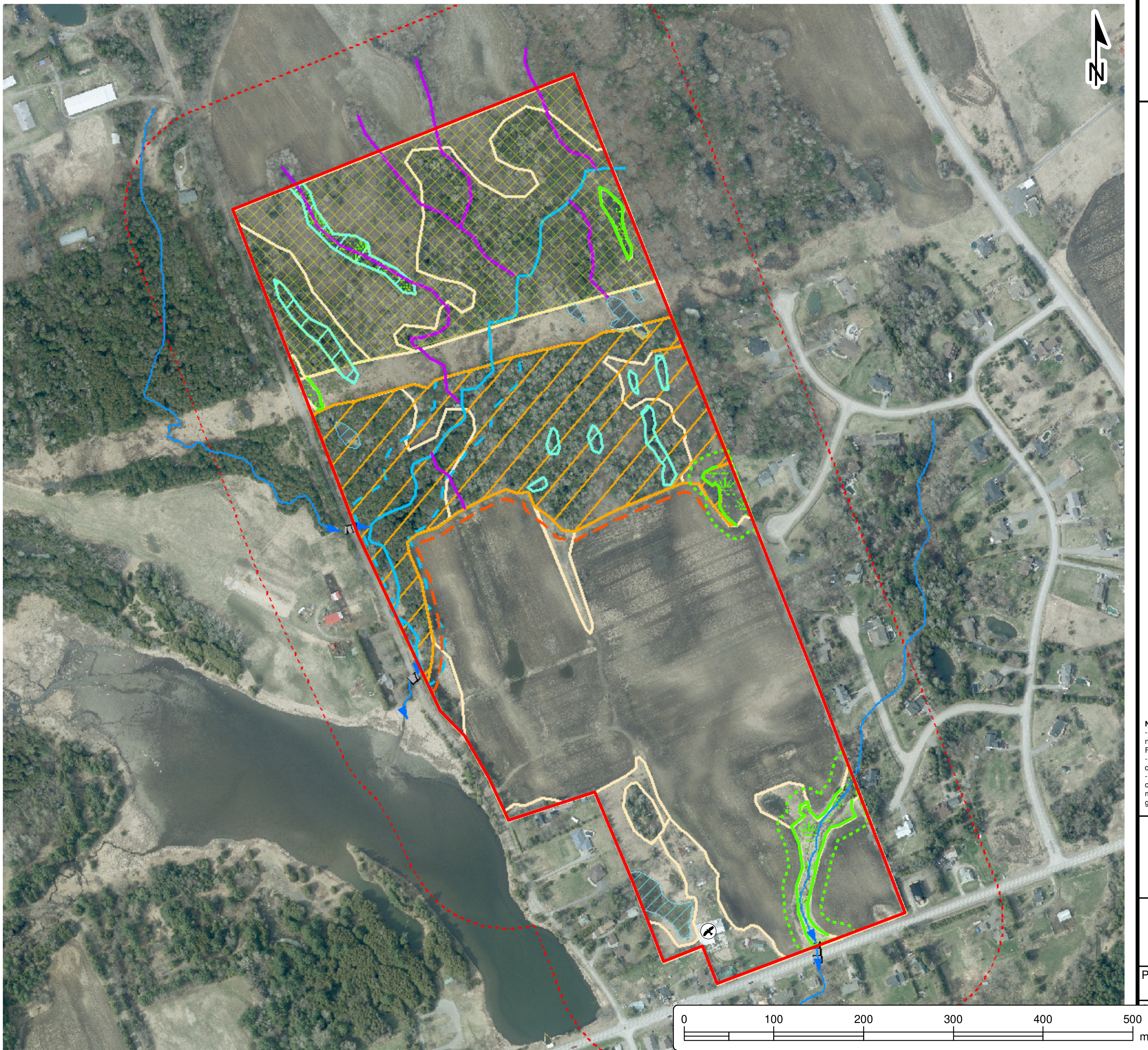
PROJECT #
21-3-8438

SCALE
HOR: 1:200
VER: N/A

DRAWING #
SP-1

(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.

CAMBIUM
194 Sophia Street
Peterborough, Ontario, K9H 1E5
Tel: (705) 742.7900 Fax: (705) 742.7907
www.cambium-inc.com

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 1–31 – 3036m² MIN. RURAL RESIDENTIAL	10.29ha	28.2%
BLOCK 101 – FUTURE ROAD WIDENING	0.15ha	0.4%
BLOCK 102 – 10–21 UNIT CONDOMINIUM/APARTMENT BUILDING	0.67ha	1.8%
BLOCK 103 – EXISTING WETLAND	1.18ha	3.2%
BLOCK 104, 109 – SWM POND	0.71ha	1.9%
BLOCK 105 – MAINTENANCE EASEMENT	0.10ha	0.3%
BLOCK 106, 108, 112 – OTHER LANDS OWNED BY DEVELOPER	15.19ha	41.5%
BLOCK 107 – DRAINAGE EASMENT	0.10ha	0.3%
BLOCK 110 – REGULATED LANDS	4.29ha	11.7%
BLOCK 112 – HYDRO EASEMENT	1.87ha	5.1%
BLOCK 113 – 0.30m RESERVE	0.00ha	0.0%
20.0m MUNICIPAL ROAD ALLOWANCE – STREET A, B, C	2.05ha	5.6%
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 (Y/M/D)	REVISION	BY
4.	22/12/06	REVISED PER MUNICIPALITY COMMENTS	C.O.
3.	22/08/31	REVISED PER MUNICIPALITY COMMENTS	C.O.
2.	22/05/26	REVISED PER MUNICIPALITY COMMENTS	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
f.1) NOT APPLICABLE
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) NON APPLICABLE
l) AS SHOWN ON THE DRAFT PLAN

OWNER'S CERTIFICATE

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

OWNER
MISTRAL LAND DEVELOPMENT INC.

DATE

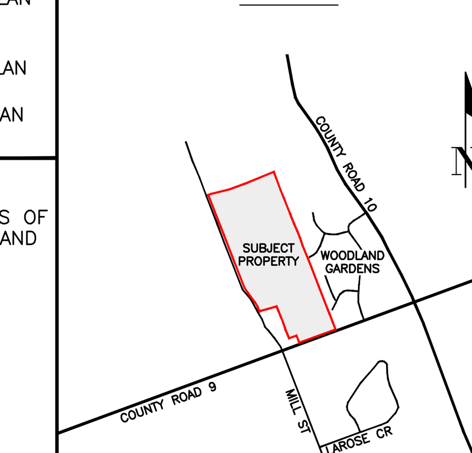
SURVEYOR'S CERTIFICATE

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

ONTARIO LAND SURVEYOR
D.S. URSO.

DATE

KEY PLAN



PROJECT No.: 21–0135
DRAWN: T.M.
CHECKED: C.O.
DATE: 22/12/06

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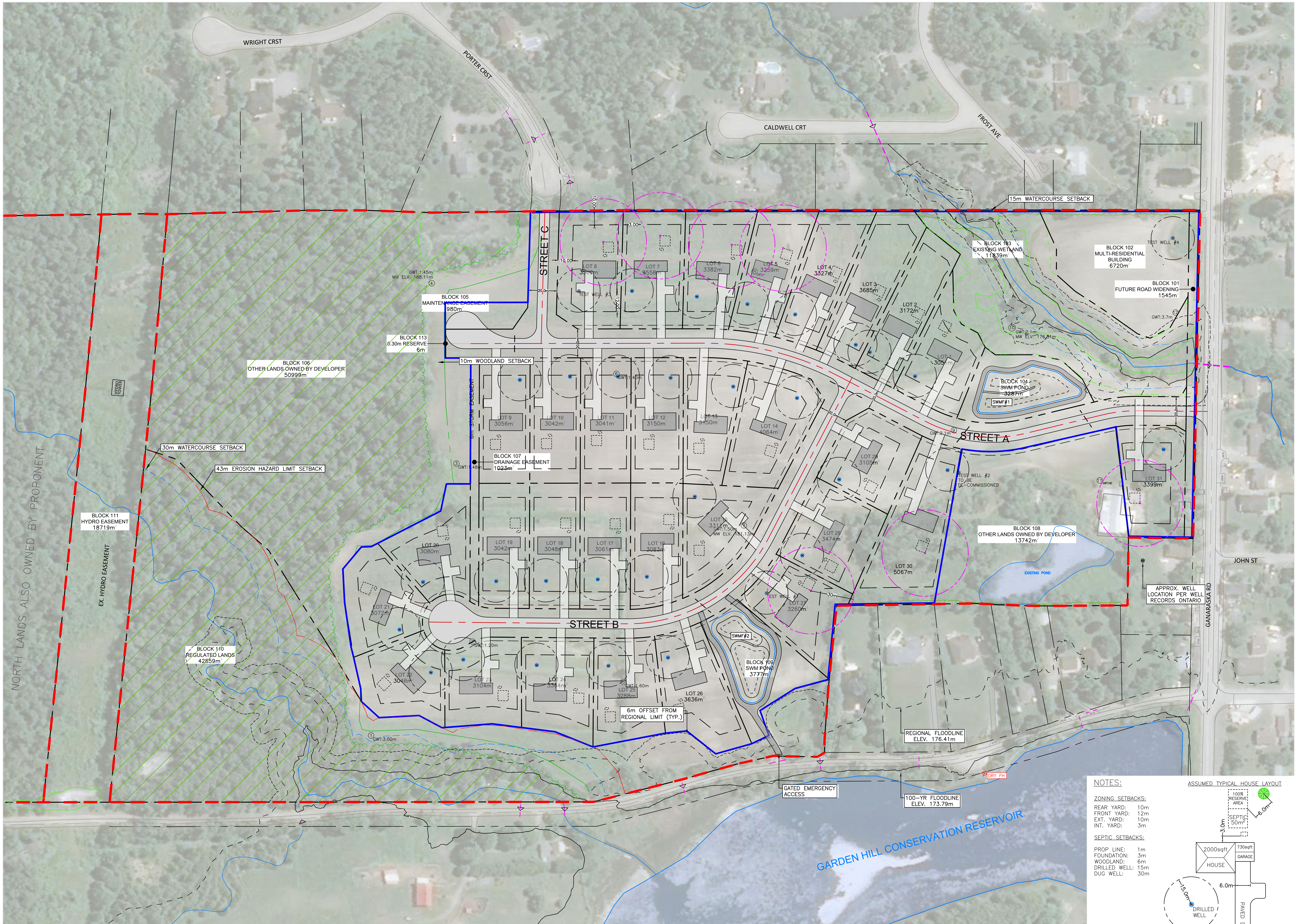
Appendix D – Servicing Plan & Typical Cross Section



LEGEND

- PROPERTY BOUNDARY
- DEVELOPMENT BOUNDARY
- SIGNIFICANT WOODLAND AREA
- SWM BLOCK
- 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

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

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MONUMENT PROJECT No.:	21-0135
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DATE:	2022/04/06
SCALES:	HORZ. 1:1250
SCALES:	VERT.

MISTRAL LAND DEVELOPMENTS

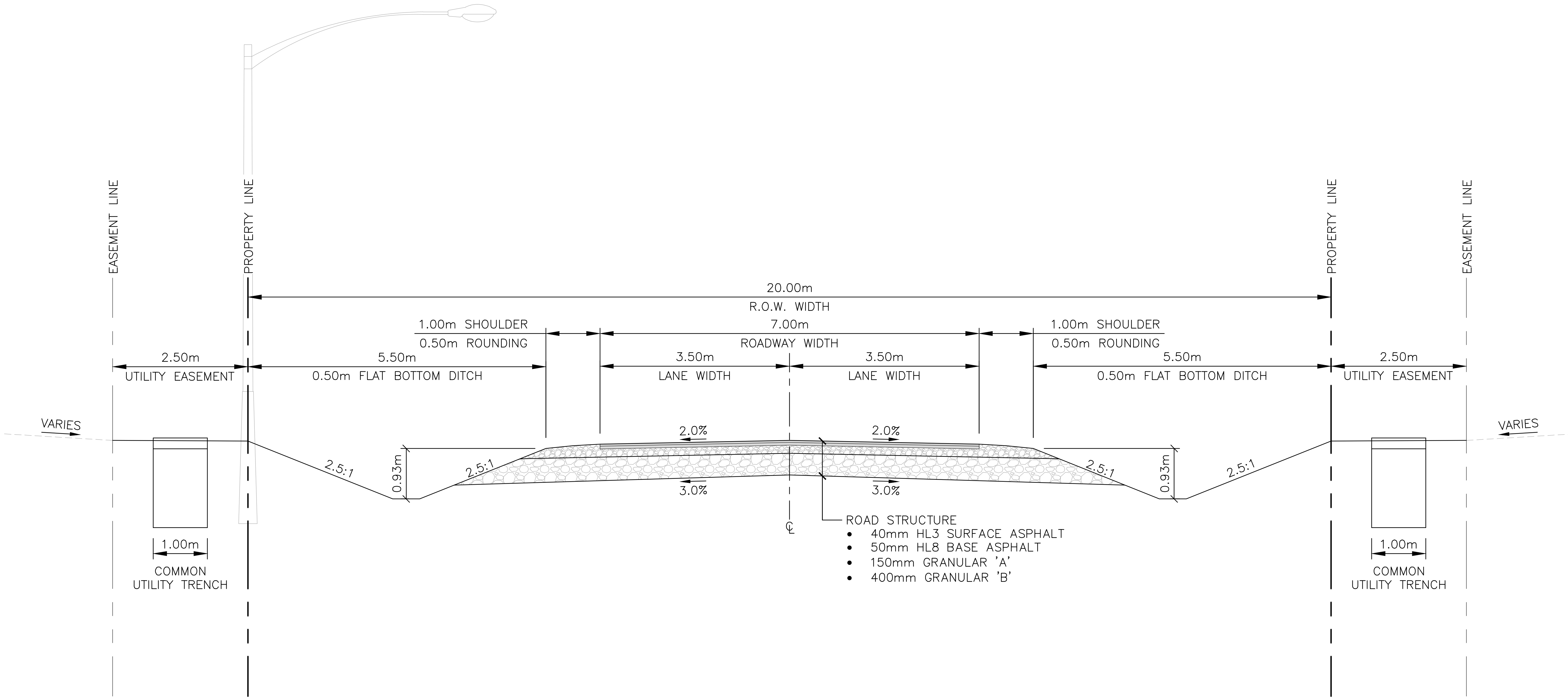
GARDEN HILL ESTATES
MUNICIPALITY OF PORT HOPE

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

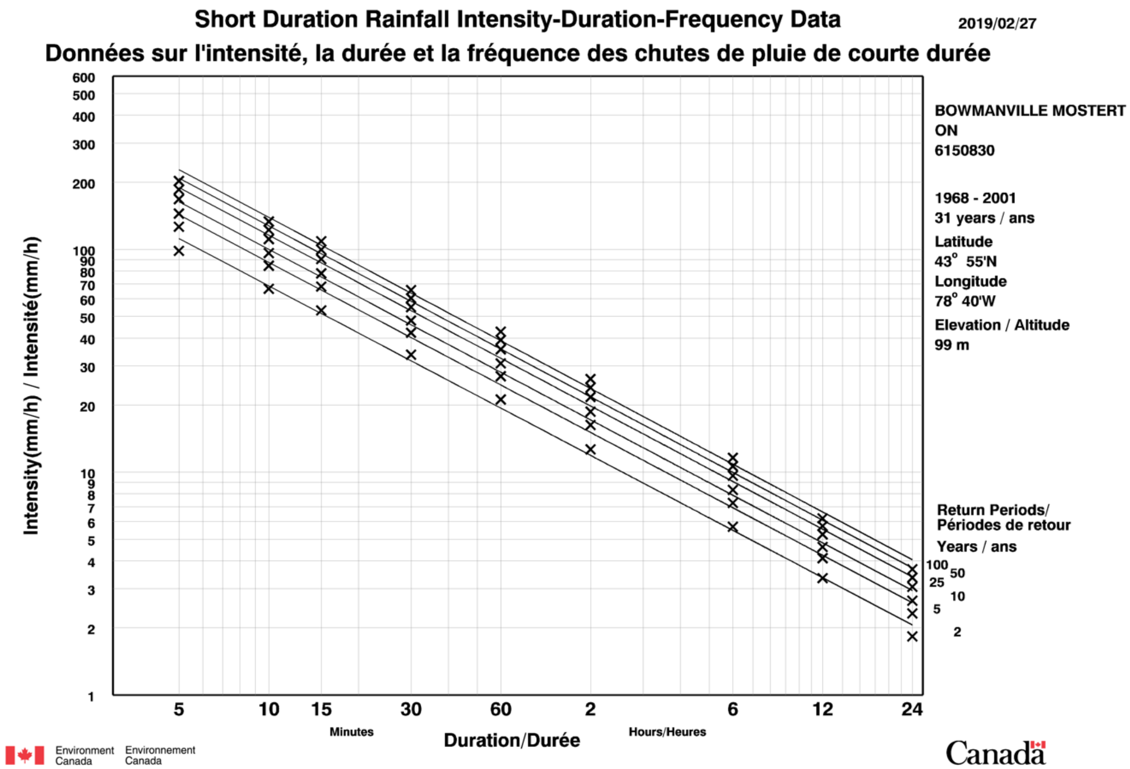


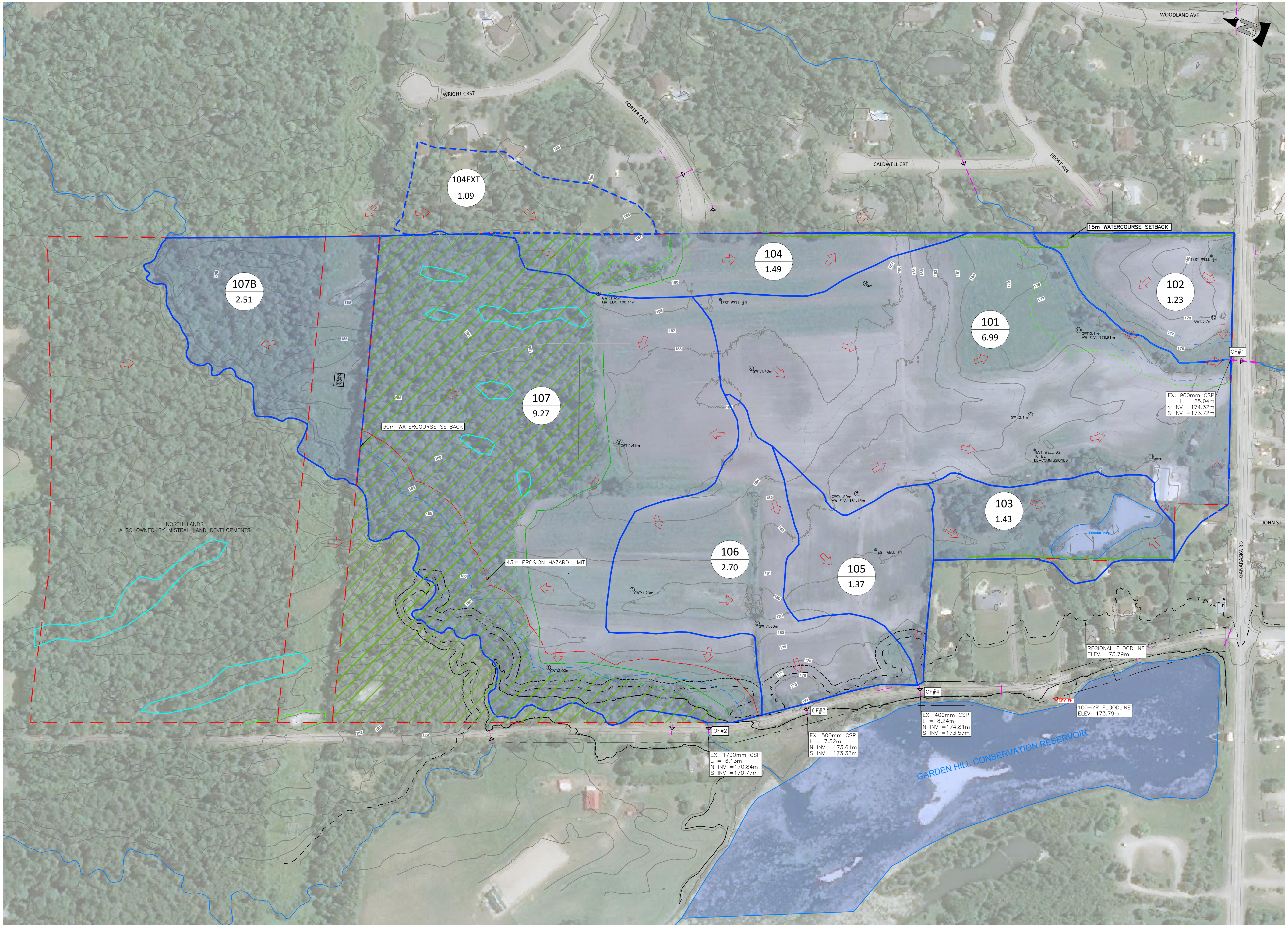
Table 2a : Return Period Rainfall Amounts (mm)
Quantité de pluie (mm) par période de retour

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 / 2.09 — CATCHMENT ID / 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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SCALES:	VERT.

MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES

MUNICIPALITY OF PORT HOPE

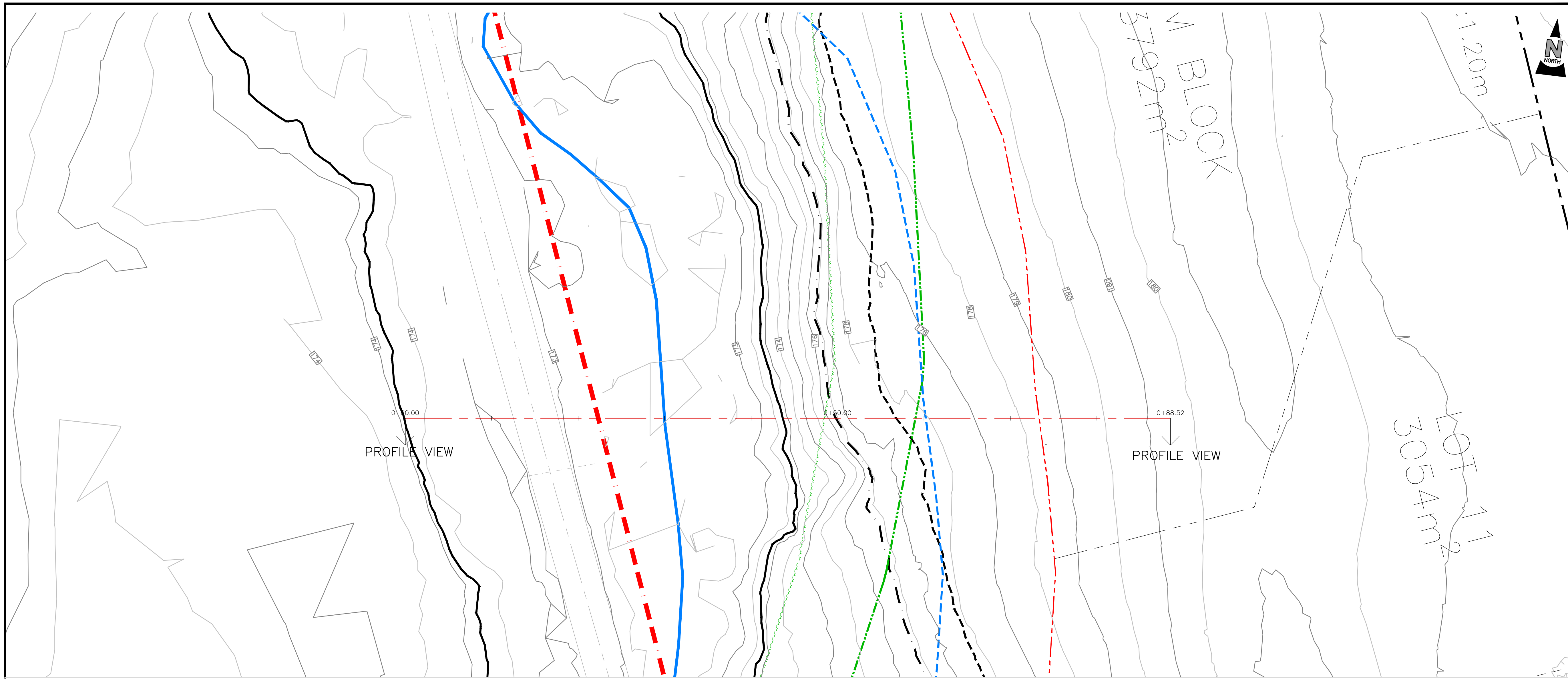
PRE-CATCHMENT AREA DRAWING

DRAWING No.
SWM-01

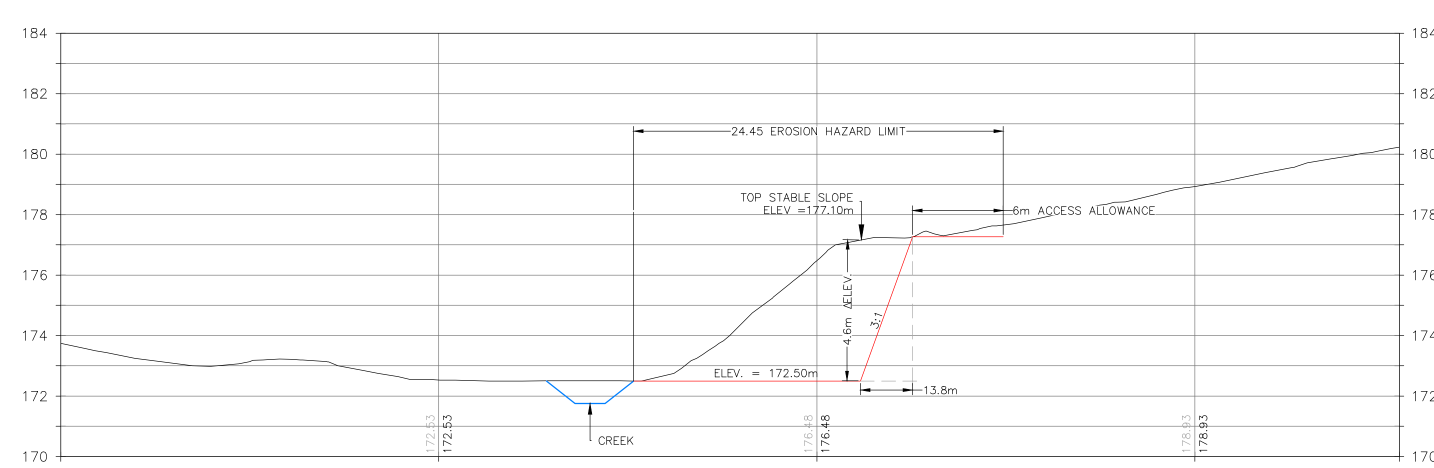
0 30 60 Meters

C:\Users\Patrick Quinn\Monument Geomatics\E&D-Garden Hill - General\4-Design\1-C3D\220830_Garden Hill Estates_Concept Plan.dwg

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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MONUMENT GEOMATICS
MONUMENTGE.COM

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

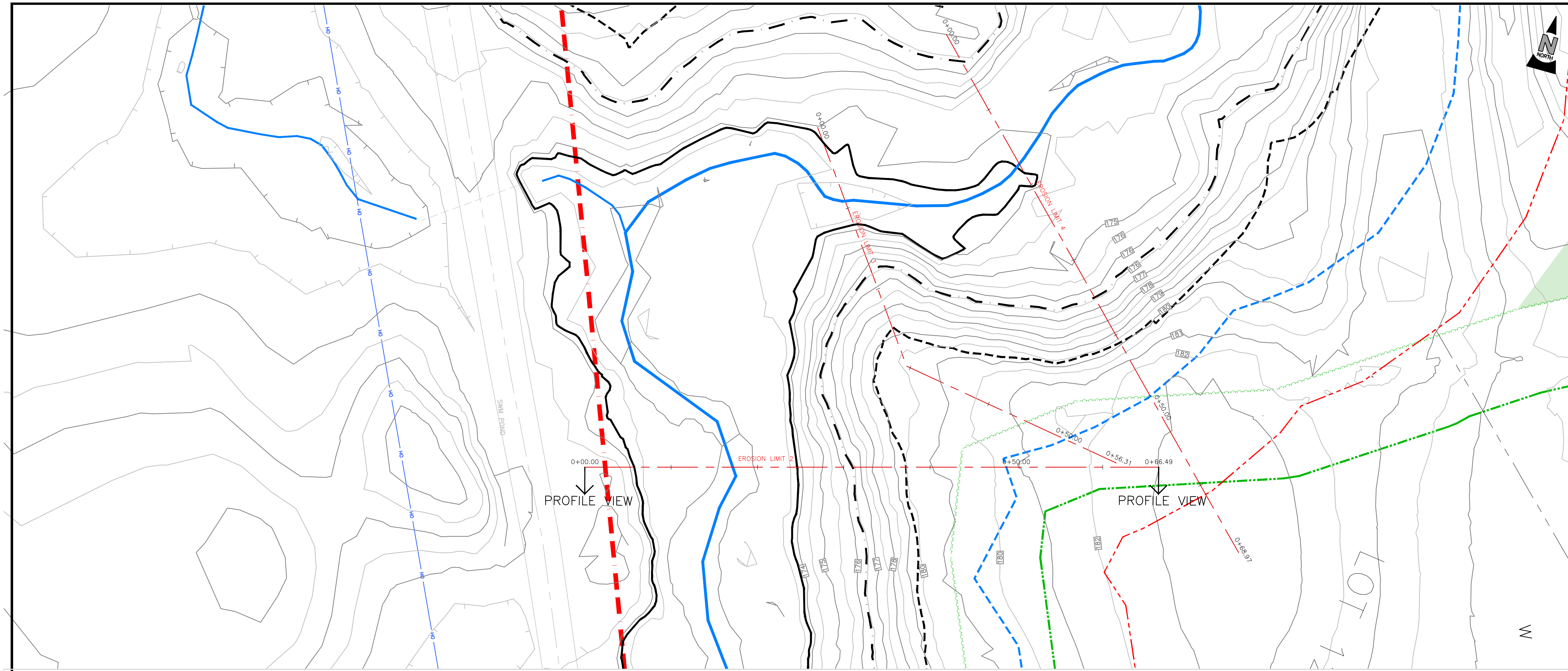
MISTRAL LAND DEVELOPMENTS

GARDEN HILL ESTATES
EROSION HAZARD LIMIT
LIMIT 1

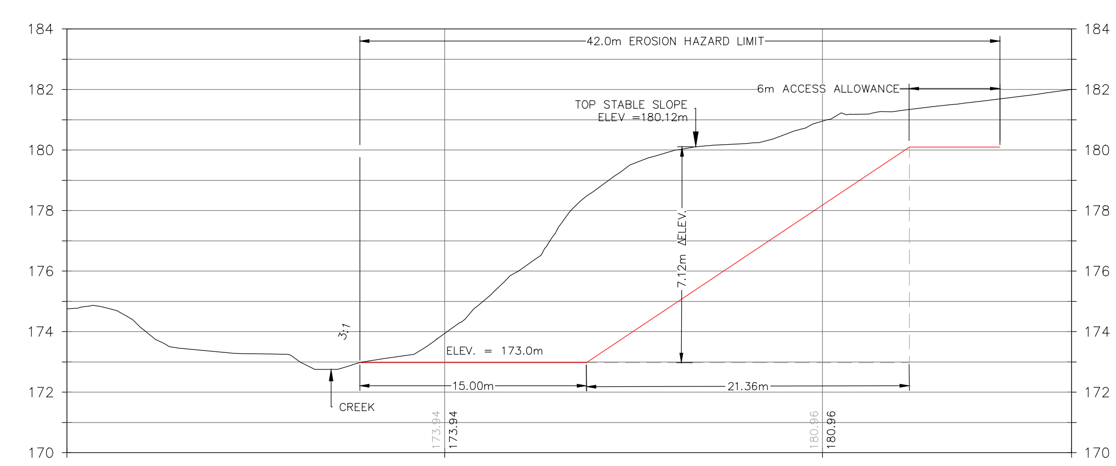
DRAWING No.
EHL-01

0 7.5 15 Meters

 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.:	-
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CHECKED:	-
ENGINEER:	-
DATE:	31/01/2022
SCALES:	HORZ. 1:250
SCALES:	VERT. 1:2

MISTRAL LAND DEVELOPMENT

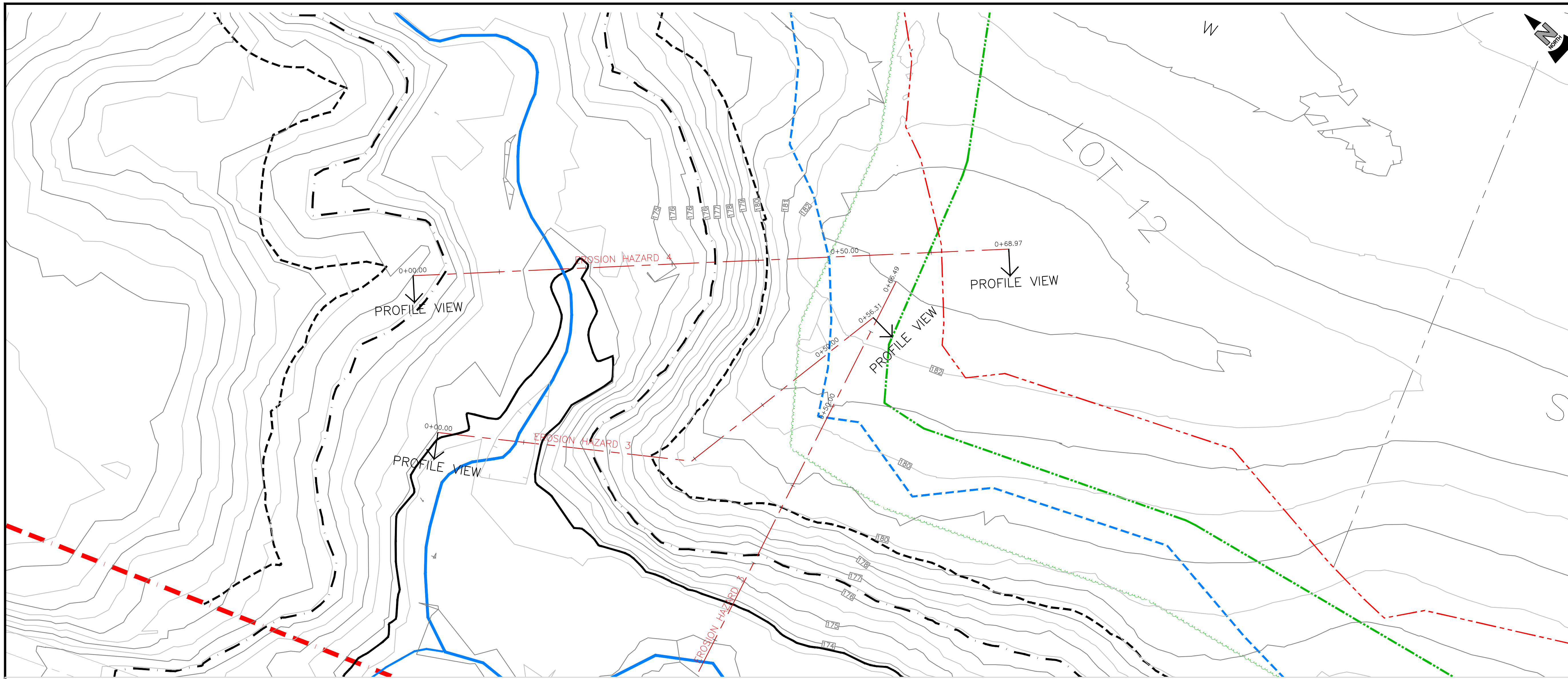
GARDEN HILL ESTATES
EROSION HAZARD LIMIT
LIMIT 2

DRAWING No.
EHL-02

0 7.5 15 Meters

C:\Users\Patrick Quinn\Monument Geomatics\L&D-Garden Hill - General\4-Design\1-C3D\C3022-21-01358ASE Concept 5.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.



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
-	-	-	-

NOT FOR CONSTRUCTION

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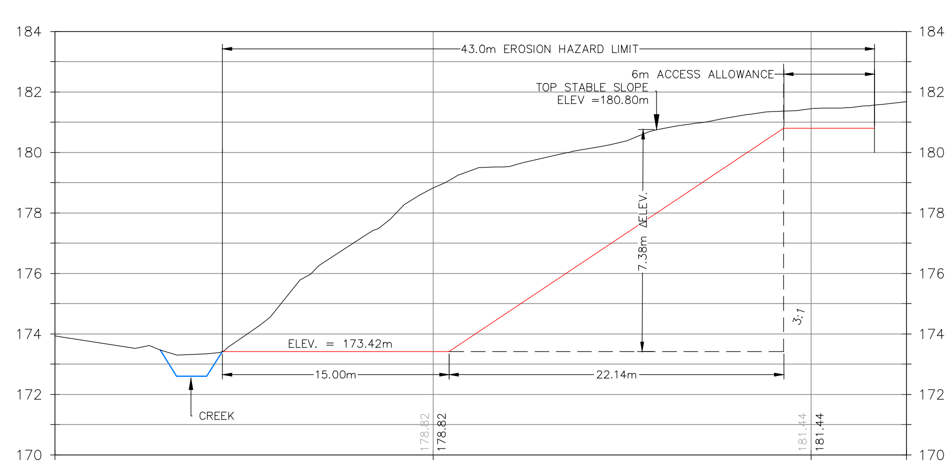
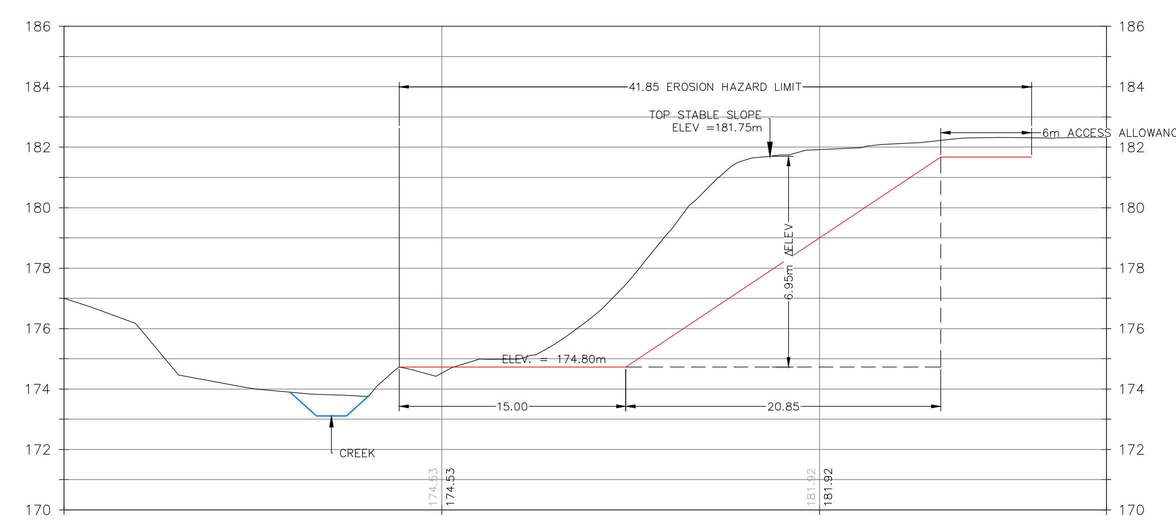
PROFILE - ALN - Hazard Limit 3(1)



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

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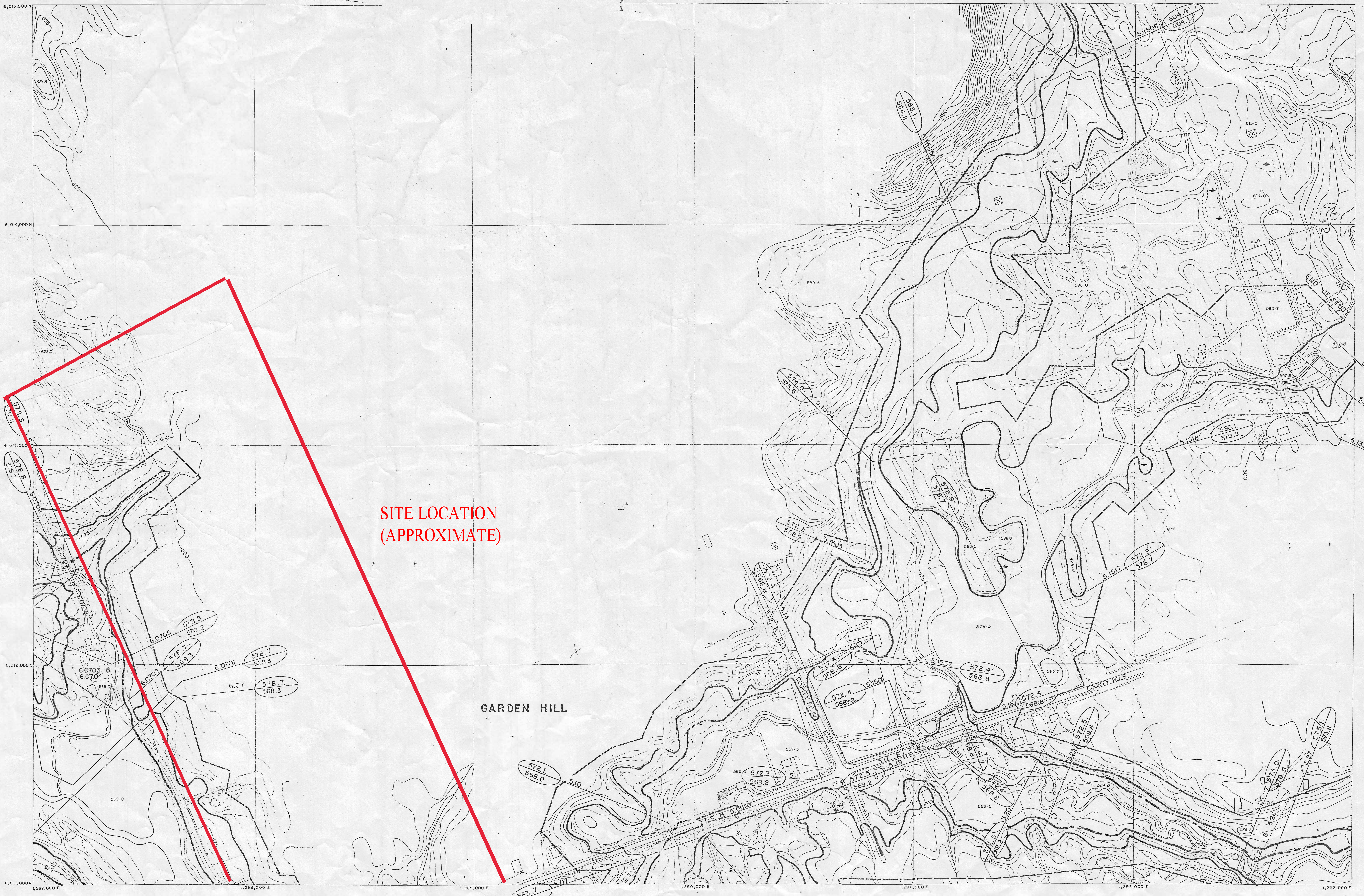
 GENERAL NOTES: ALL INFORMATION TO BE VERIFIED ON SITE PRIOR TO COMMENCING ANY WORK.ALL UTILITY LOCATIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. ALL DIMENSIONS SHOWN ARE IN METERS OR MILLIMETERS, UNLESS OTHERWISE NOTED.

Appendix 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

t... ...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, 2½ FEET INTERPOLATION
GEODETIC DATUM — MEAN SEA LEVEL AS ESTABLISHED BY GEODETIC SURVEY OF CANADA.
PROJECTION SYSTEM — 3° TRANSVERSE MERCATOR

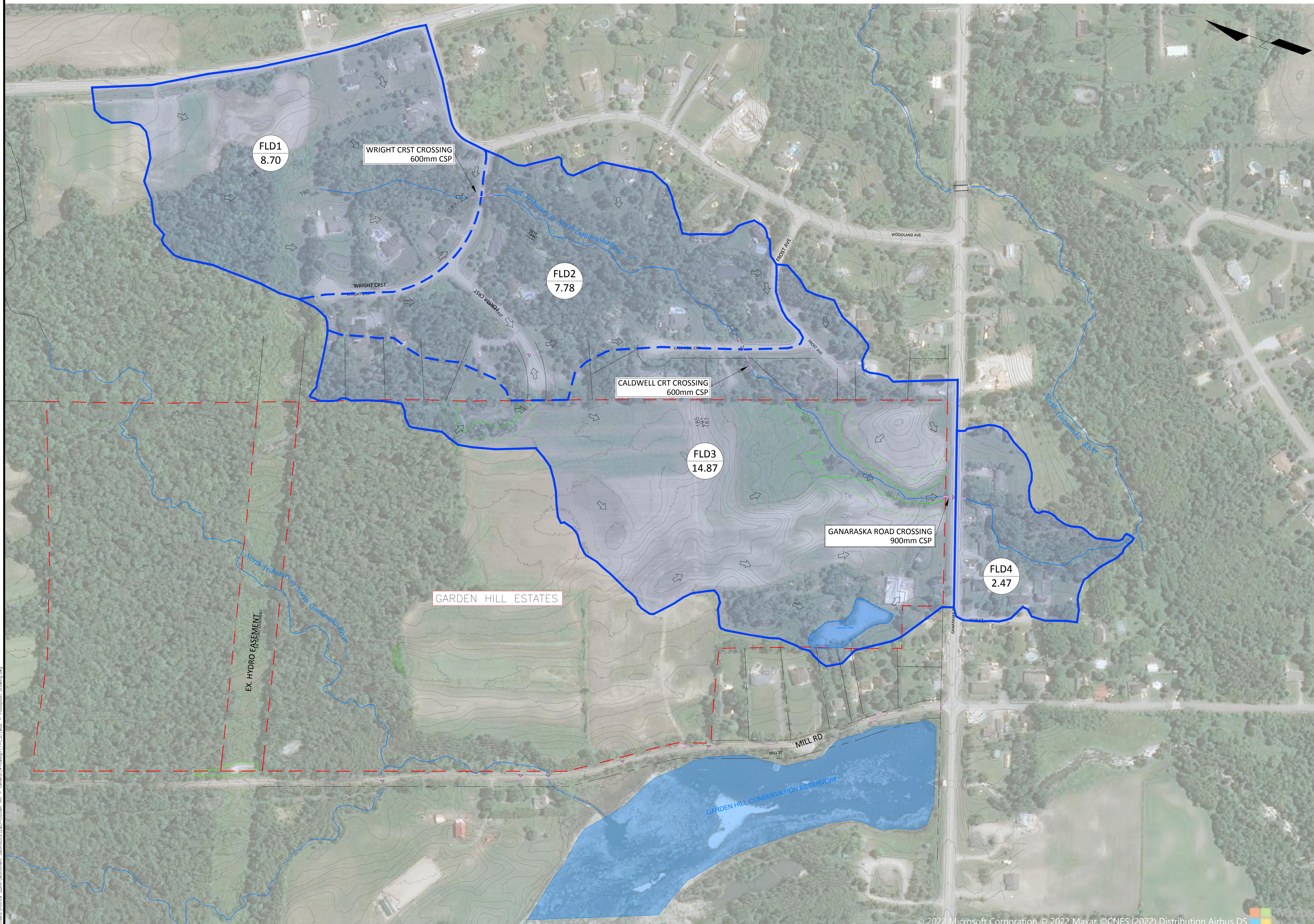
DATE JAN. 1990

SHEET 82 of 117

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
–	–	–	–

NOT FOR CONSTRUCTION

MONUMENT GEOMATICS
MONUMENTGE.COM

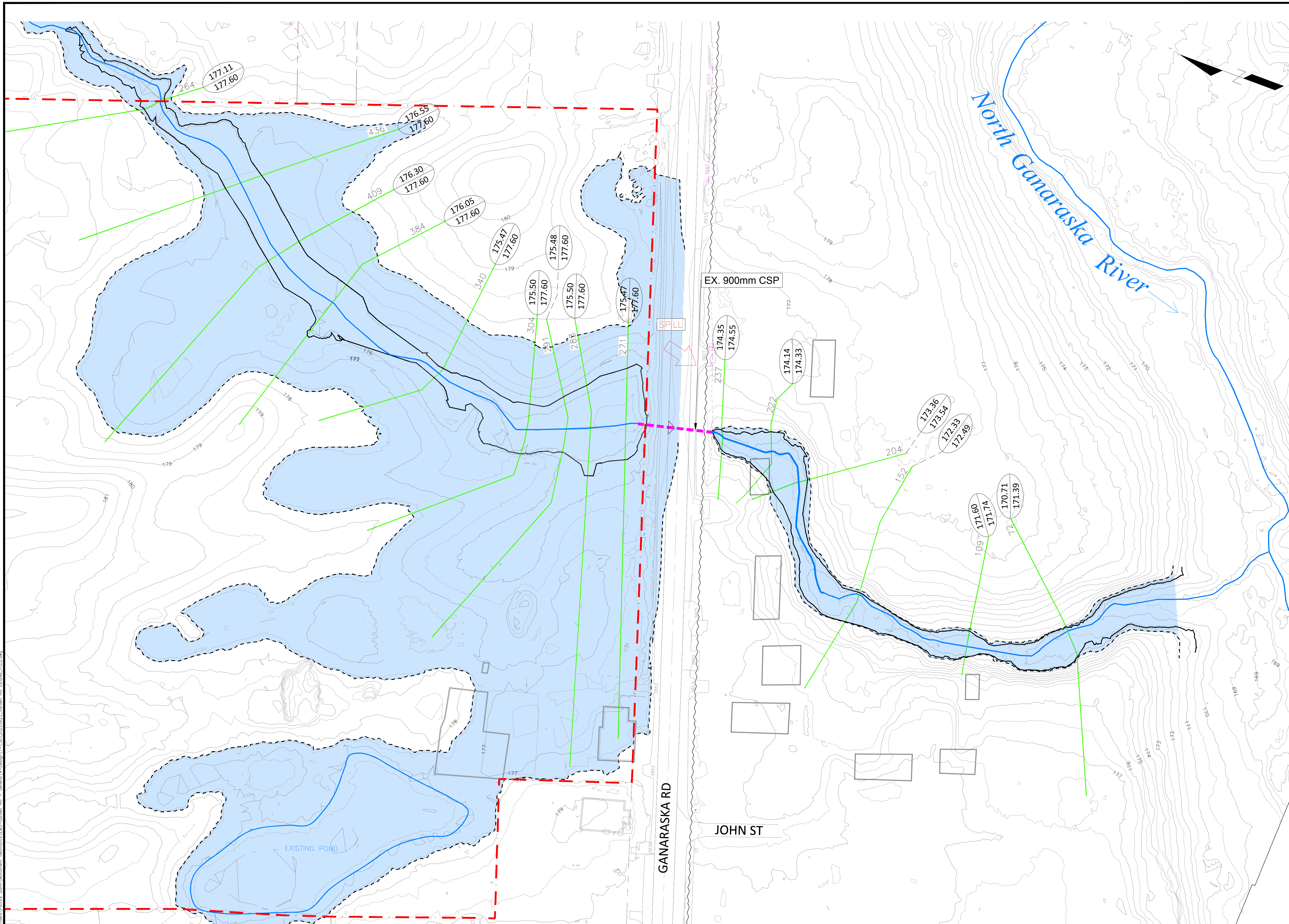
MONUMENT PROJECT No.:	21–0135
DRAWN:	P.Q.
CHECKED:	–
ENGINEER:	–
DATE:	11/04/2022
SCALES:	HORZ. 1:2000
SCALES:	VERT.

MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES

MUNICIPALITY OF PORT HOPE
SOUTH TRIBUTARY FLOODLINE
CATCHMENT AREA DRAWING

DRAWING No.
FL-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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MONUMENT PROJECT No.:	21-0135
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SCALES:	HORZ. 1:600
SCALES:	VERT.

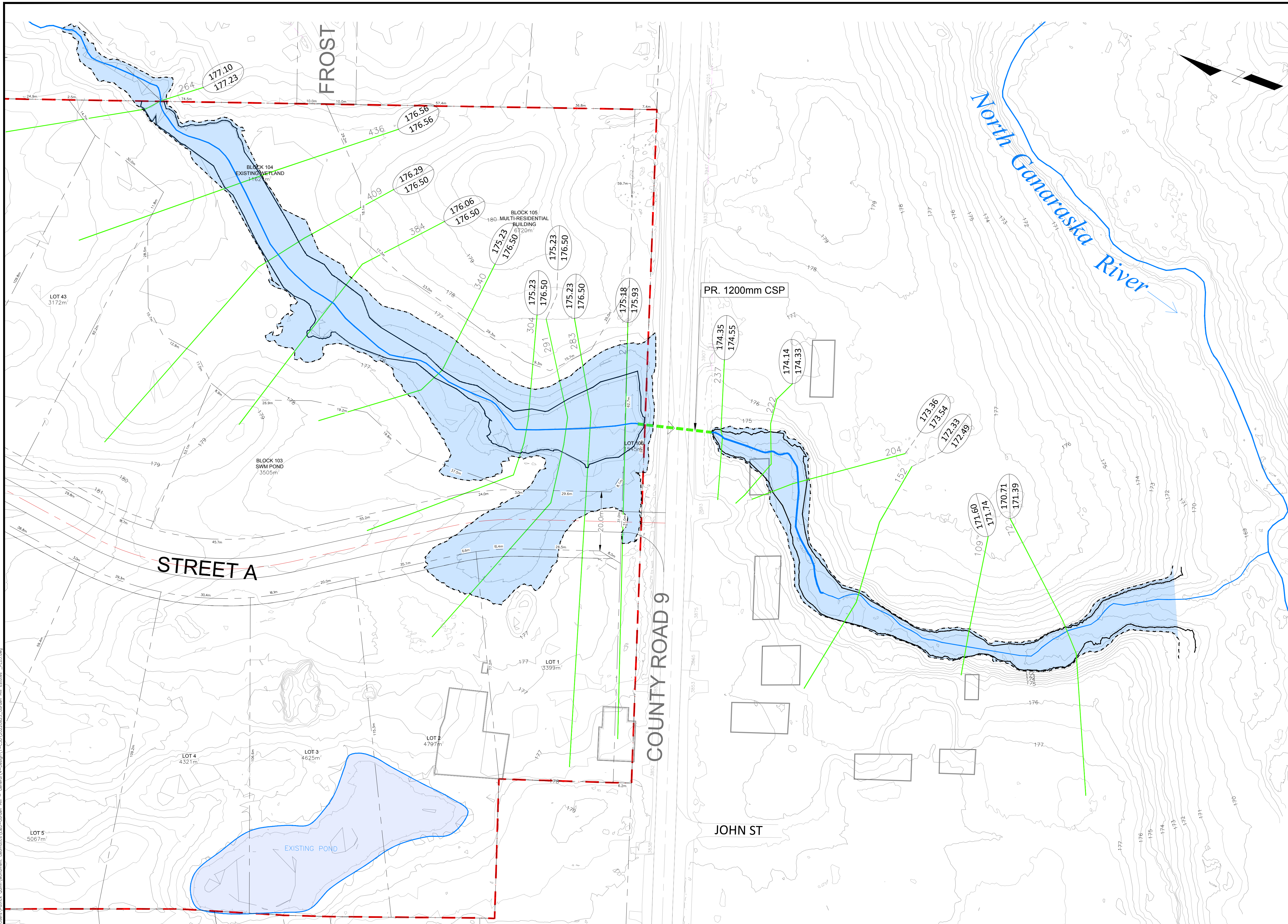
MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES
MUNICIPALITY OF PORT HOPE

SOUTH TRIBUTARY FLOODLINE
EXISTING CONDITIONS

DRAWING No.
FL-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
–	–	–	–

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SCALES:	HORZ. 1:600
SCALES:	VERT.

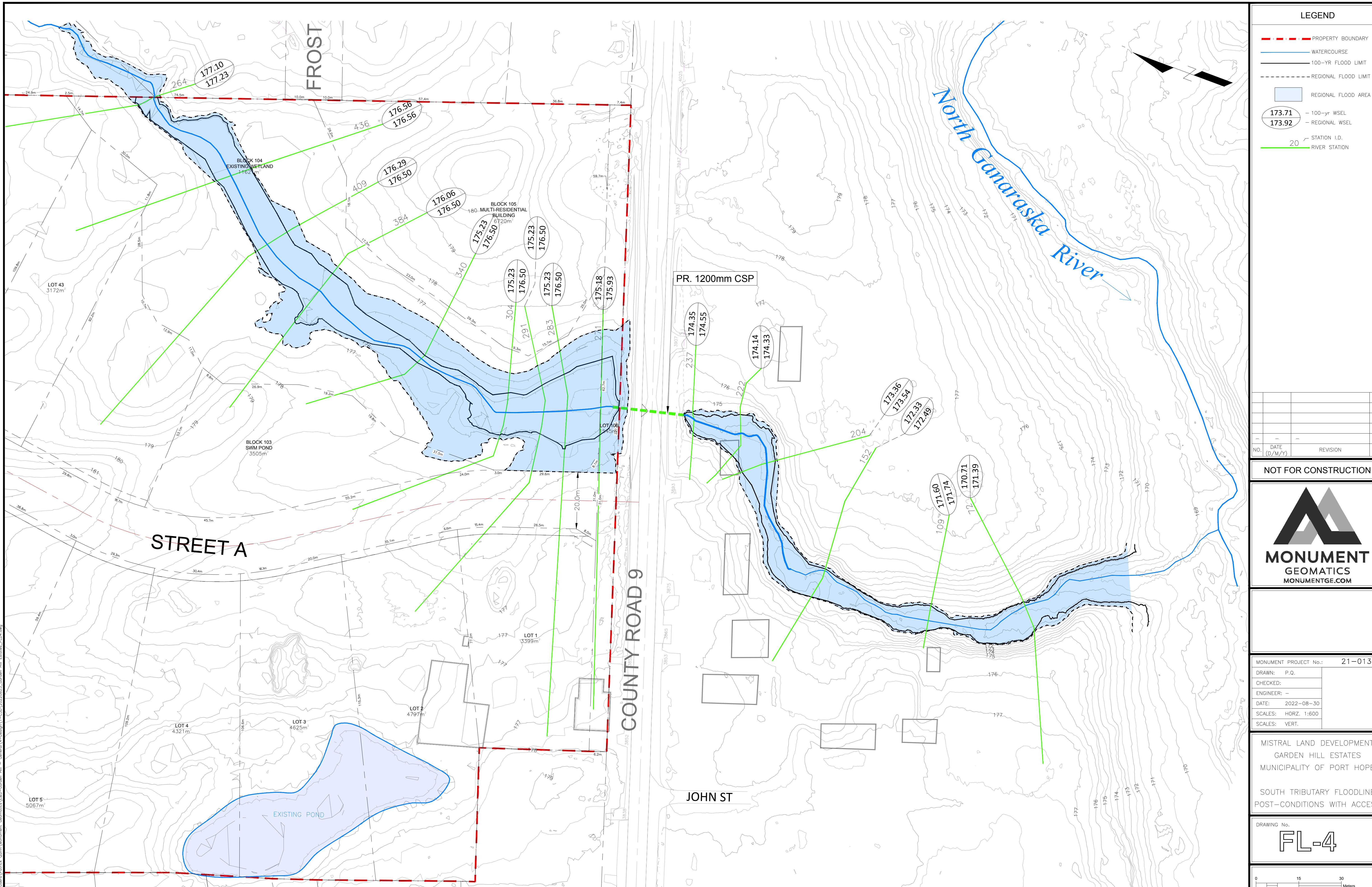
MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES
MUNICIPALITY OF PORT HOPE

SOUTH TRIBUTARY FLOODLINE
POST-CONDITIONS

DRAWING No.
FL-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.



NO.	DATE (D/M/Y)	REVISION	BY
–	–	–	–

NOT FOR CONSTRUCTION

MONUMENT GEOMATICS
MONUMENTGE.COM

MONUMENT PROJECT No.:	21-0135
DRAWN: P.Q.	
CHECKED:	
ENGINEER: –	
DATE: 2022-08-30	
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.
FL-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

% 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**

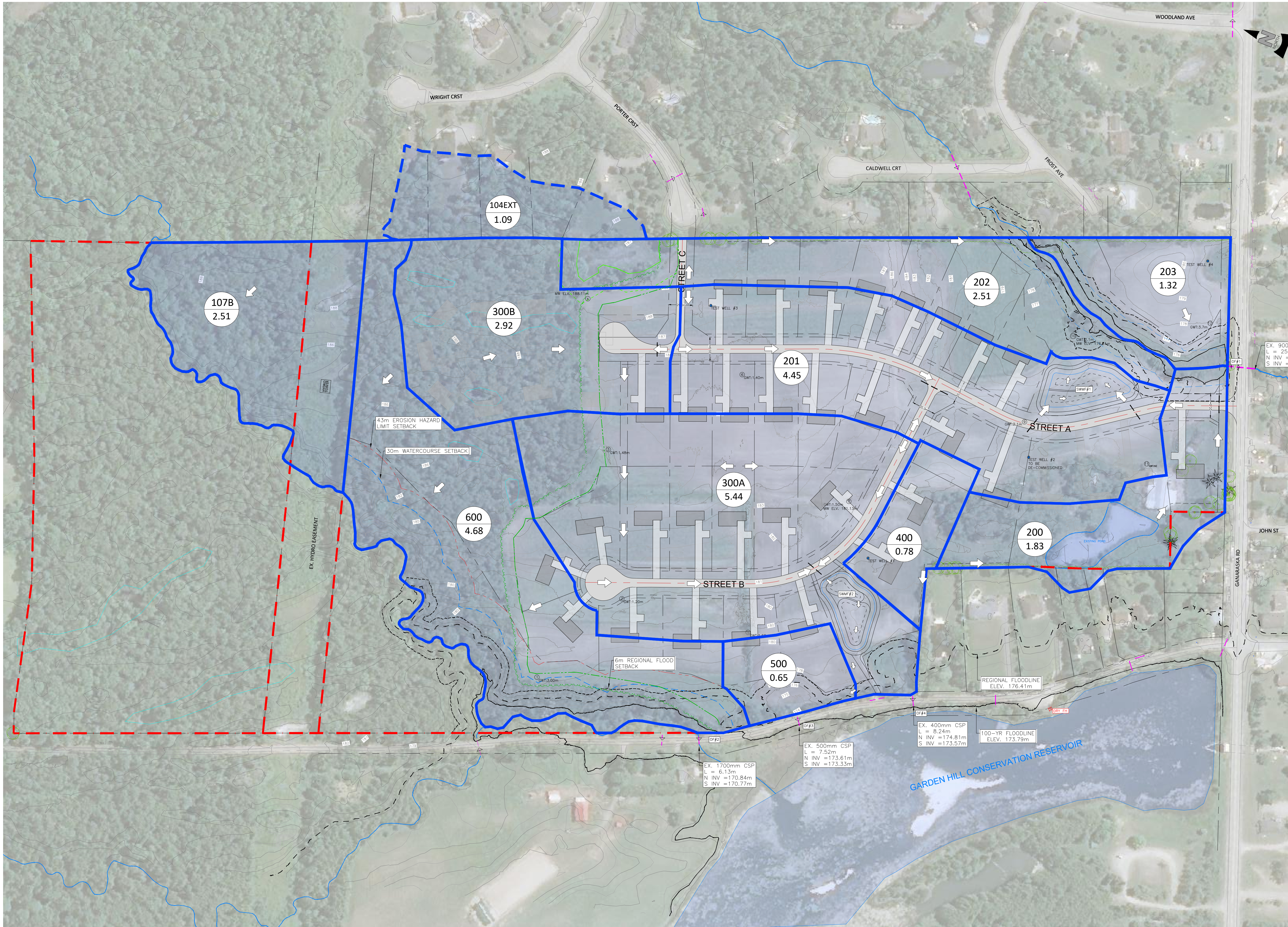
Drainage Area Slope using 85/10 Method

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

Appendix J – Post-Development Catchment Drawing And Preliminary Grading Plan



LEGEND

- PROPERTY BOUNDARY
- 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); 60 — CURVE NUMBER
- CATCHMENT BOUNDARY
- EXTERNAL CATCHMENT
- OVERLAND FLOW ARROW
- 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
–	–	–	–

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MONUMENT GEOMATICS
MONUMENTGE.COM

MONUMENT PROJECT No.:	–
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SCALES:	HORZ. 1:1500
SCALES:	VERT.

MISTRAL LAND DEVELOPMENT
GARDEN HILL ESTATES

MUNICIPALITY OF PORT HOPE
POST-DEVELOPMENT
DRAINAGE PLAN

DRAWING No.
SWM-02

0 30 60 Meters

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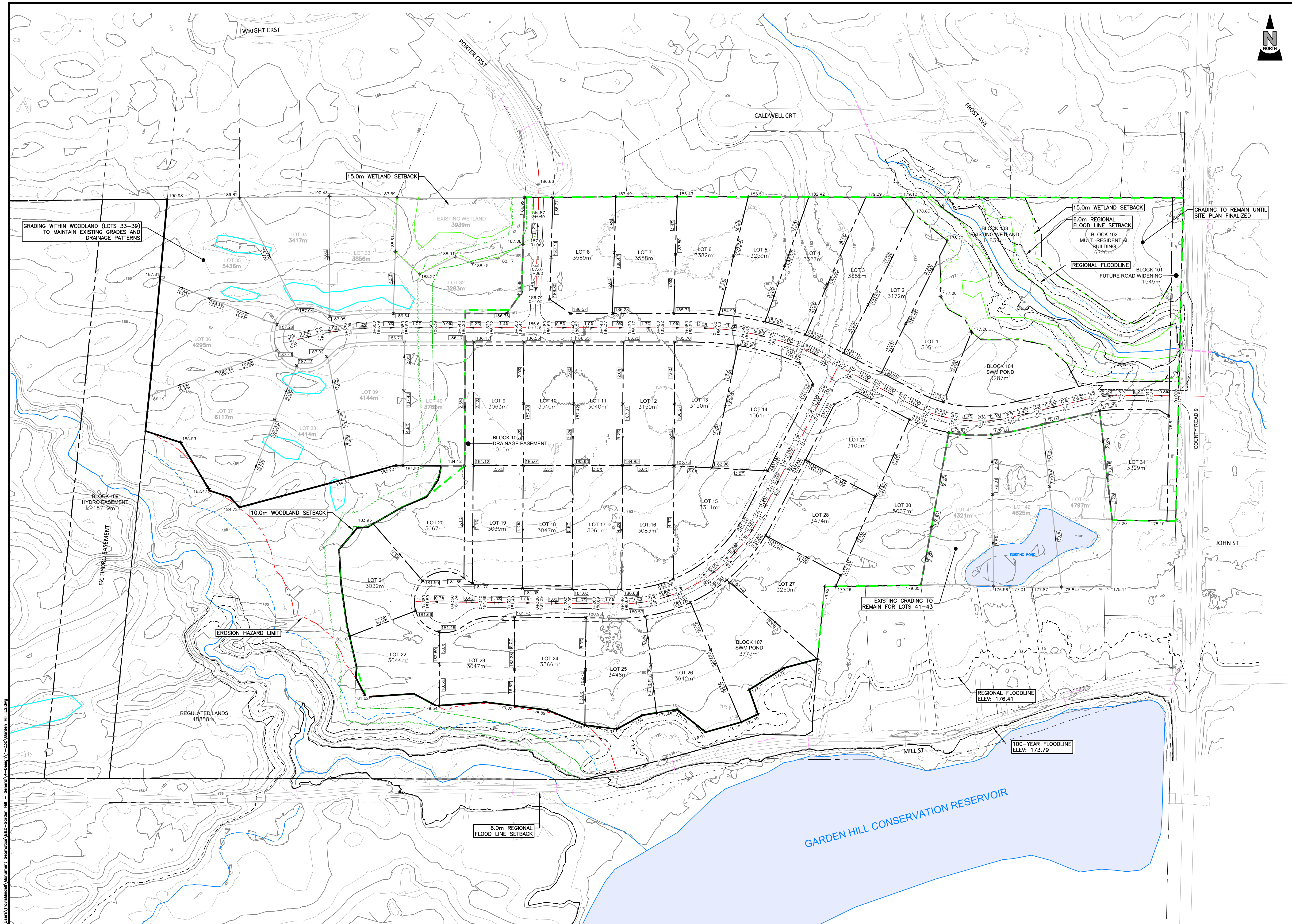

LEGEND

EXISTING

- EX. TEST PIT & DRILL HOLE (TP, DH)
- EX. BENCHMARK & SURVEY HUBS (BM)
- EX. SIB/IB & CM's (S.I.B., MON)
- EX. LEGAL LINE
- EX. LEGAL R.O.W. & EASEMENT
- EX. SANITARY & MANHOLE
- EX. SANITARY FORCE MAIN (FORCE)
- EX. STORM & MANHOLE
- EX. SINGLE & DOUBLE CATCHBASIN c/w CB LEADS (DCB)
- EX. CATCHBASIN MANHOLE
- EX. CULVERT
- EX. FIRE HYDRANT & VALVE ASSEMBLY (WV)
- EX. WATERMAIN & VALVE (WV)
- EX. BLOW-OFF ASSEMBLY
- EX. CURB STOP (CS)
- EX. ROAD & SIDEWALK
- EX. ROAD SIGN(s)
- EX. SHOULDER
- EX. TOP OF SLOPE
- EX. TOE OF SLOPE
- EX. DITCH or SWALE
- EX. FENCE
- EX. OVERHEAD LINES (OH)
- EX. UNDERGROUND LINES (UG)
- EX. POWER POLE & ANCHOR (PP)
- EX. GAS MAIN (GAS)

PROPOSED

- PR. LEGAL LINE
- PR. LEGAL R.O.W. & EASEMENT
- PR. SANITARY & MANHOLE (SM)
- PR. STORM & MANHOLE (DM)
- PR. SINGLE & DOUBLE CATCHBASIN c/w CB LEADS (DCB)
- PR. CATCHBASIN MANHOLE
- PR. CULVERT
- PR. DITCH or SWALE
- PR. FIRE HYDRANT & VALVE ASSEMBLY
- PR. WATERMAIN & VALVE
- PR. GRAVEL SHOULDER
- PR. ASPHALT
- PR. CURB & SIDEWALK
- PR. GRAVEL
- PR. ROAD SIGN(s)
- EX / PR. ELEVATIONS (100.00)
- PR. GRADES (1.0%)

NO.	DATE (D/M/Y)	REVISION	BY
1.	22/09/01	SECOND SUBMISSION	C.O.

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MONUMENTGE.COM

MONUMENT PROJECT No.:	21–0135
DRAWN:	T.M.
DESIGNED:	T.M.
CHECKED:	P.Q.
ENGINEER:	–
DATE:	22/09/01
H SCALE:	1:1250
V SCALE:	N/A

MISTRAL LAND DEVELOPMENTS INC.

GARDEN HILL ESTATES
MUNICIPALITY OF PORT HOPE

LOT GRADING PLAN

DRAWING No.
LG

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.

Post-development

Garden Hill Estates

Catchment	Area (ha)
200	1.83
201	4.46
202	2.51
203	1.23
300A	5.44
300B	2.92
400	0.78
500	0.66
600	4.67

Drainage Area Slope using 85/10 Method

Catchment	$L_{watershed}$	Slope
200	100	1.0%
201	150	3.5%
202	120	4.0%
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	202	203
Outlet Point	OF1	OF1	OF1	OF1
Area	1.82	4.45	2.95	1.23
Flow Length	100	150	120	100
Slope	2%	3.5%	4%	2%
Percent Impervious (%)	6.5%	31.0%	3.9%	0.0%
N Impervious	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.06
Subarea Routing (%)	80%	30%	80%	-
Curve Number (AMCII)	60	73	72	72

Name	300A	300B	400	500	600	107B
Outlet Point	OF3	OF3	OF4	OF3	OF2	OF2
Area (ha)	5.44	2.48	0.78	0.66	4.67	2.51
Flow Length (m)	140	66	100	75	240	177
Slope	2.5%	2.0%	2.0%	3.0%	2.3%	1.1%
Percent Impervious (%)	16.8%	29.4%	15.6%	4.4%	1.7%	0
N Impervious	0.013	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.25	0.3	0.13
Subarea Routing	30%	30%	80%	80%	80%	-
Curve Number (AMCII)	75	75	78	78	56	30

Appendix K – Stage-Storage-Discharge Relationship

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 =	175.20
Permanent Pool Elevation (P.P) =	176.2
Extended Detention (E.D.) =	176.50
Active Storage (A.S.) =	176.90
Top of Berm Elevation (T.B.) =	177.20
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 =	176.2	Invert =	176.50	Invert =	176.90
Diameter =	0.085	Weir Length =	0.6	Weir Length =	3
C =	0.60				
Obvert =	176.29				
Radius =	0.04				
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 = $Q = 1.84LH^{\frac{3}{2}}$

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)
	175.20	573.5	0.0	0	0.00	0.000	0.000	0.00	0.000	0.00	0.000	0.0	0.0	0.000
	175.30	636.5	60.5	60.5	0.00	0.000	0.000	0.00	0.000	0.00	0.000	60.5	0.0	0.000
	175.40	701.4	66.9	127.4	0.00	0.000	0.000	0.00	0.000	0.00	0.000	127.4	0.0	0.000
	175.50	767.4	73.4	200.8	0.00	0.000	0.000	0.00	0.000	0.00	0.000	200.8	0.0	0.000
	175.60	834.4	80.1	280.9	0.00	0.000	0.000	0.00	0.000	0.00	0.000	280.9	0.0	0.000
	175.70	902.4	86.8	367.8	0.00	0.000	0.000	0.00	0.000	0.00	0.000	367.8	0.0	0.000
	175.80	971.6	93.7	461.5	0.00	0.000	0.000	0.00	0.000	0.00	0.000	461.5	0.0	0.000
	175.90	1041.9	100.7	562.1	0.00	0.000	0.000	0.00	0.000	0.00	0.000	562.1	0.0	0.000
	176.00	1113.2	107.8	669.9	0.00	0.000	0.000	0.00	0.000	0.00	0.000	669.9	0.0	0.000
	176.10	1185.7	114.9	784.8	0.00	0.000	0.000	0.00	0.000	0.00	0.000	784.8	0.0	0.000
P.P.	176.20	1259.2	122.2	907.1	0.00	-0.043	0.000	0.00	0.000	0.00	0.000	907.1	0.0	0.000
	176.30	1333.8	129.6	1036.7	0.10	0.057	0.004	0.00	0.000	0.00	0.000	0.0	129.6	0.004
	176.40	1409.5	137.2	1173.9	0.20	0.157	0.006	0.00	0.000	0.00	0.000	0.0	266.8	0.006
E.D.	176.50	1486.3	144.8	1318.7	0.30	0.257	0.008	0.00	0.000	0.00	0.000	0.0	411.6	0.008
	176.60	1625.6	155.6	1474.3	0.40	0.357	0.009	0.10	0.035	0.00	0.000	0.0	567.2	0.044
	176.70	1726.5	167.6	1641.9	0.50	0.457	0.010	0.20	0.099	0.00	0.000	0.0	734.8	0.109
	176.80	1829.6	177.8	1819.7	0.60	0.557	0.011	0.30	0.181	0.00	0.000	0.0	912.6	0.193
A.S.	176.90	1934.6	188.2	2007.9	0.70	0.657	0.012	0.40	0.279	0.00	0.000	0.0	1100.8	0.292
	177.00	2041.7	198.8	2206.7	0.80	0.757	0.013	0.50	0.390	0.10	0.158	0.0	1299.6	0.562
	177.10	2150.8	209.6	2416.3	0.90	0.857	0.014	0.60	0.513	0.20	0.448	0.0	1509.3	0.975
T.P.	177.20	2261.9	220.6	2637.0	1.00	0.957	0.015	0.70	0.647	0.30	0.823	0.0	1729.9	1.485

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

MONUMENT GEOMATICS

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 =	177.70
Permanent Pool Elevation (P.P) =	178.7
Extended Detention (E.D.) =	179.00
Active Storage (A.S.) =	179.40
Top of Berm Elevation (T.B.) =	179.70
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 =	178.7	Invert =	179.00	Invert =	179.40
Diameter =	0.1	Weir Length =	0.85	Weir Length =	3
C =	0.60				
Obvert =	178.80				
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)
	177.70	513	0.0	0	0.00	0.000	0.000	0.00	0.000	0.00	0.000	0.0	0.0	0.000
	177.80	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
	177.90	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
	178.00	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
	178.10	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
	178.20	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
	178.30	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
	178.40	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
	178.50	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
	178.60	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	178.70	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
	178.80	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
	178.90	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
E.D	179.00	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
	179.10	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
	179.20	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
	179.30	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.	179.40	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
	179.50	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
	179.60	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.	179.70	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

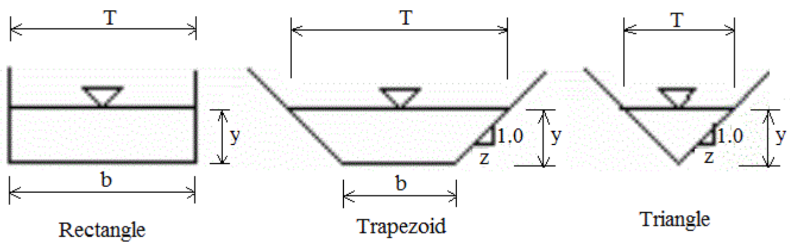
Manning's Open Channel Flow Sheet

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

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				Calculated Values	
Bottom width, b =	0.5	m		Top Width, T =	3.200 m
Channel depth, y =	0.45	m		Area, A =	0.833 m^2
Side Slope, z =	3	:1		Wetted Perimeter, P =	3.346 m
Longitudinal Slope, S =	0.03	m/m		Hydraulic Radius, R =	0.249 m
Roughness, n =	0.035	Grassed			

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.

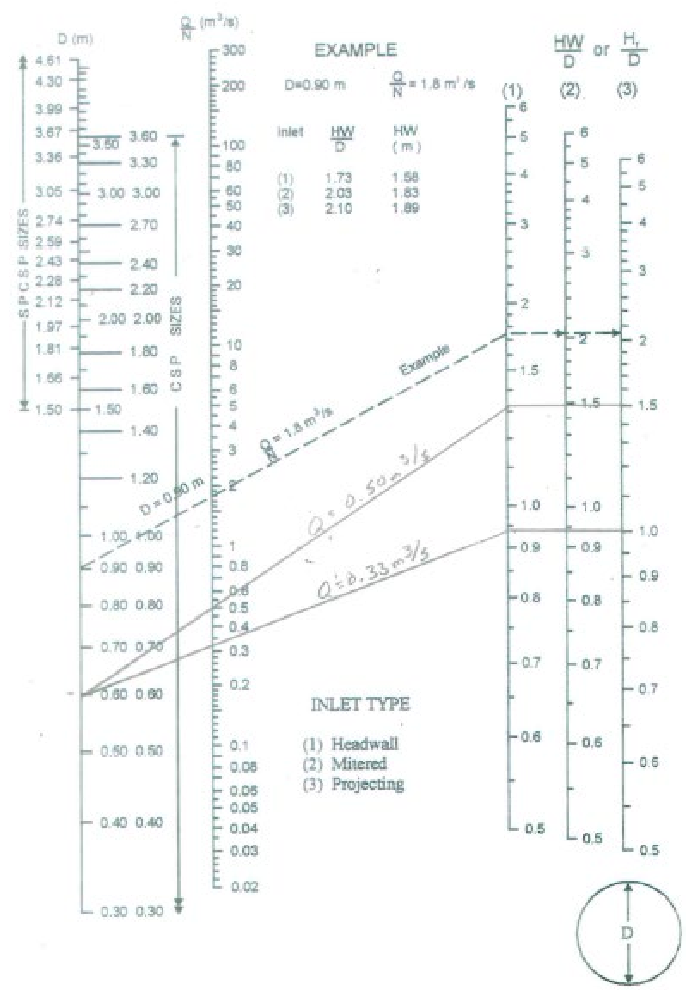
Appendix M – Design Chart 2.32: Inlet Control Nomograph

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)

Appendix N – PCSWMM Output Files

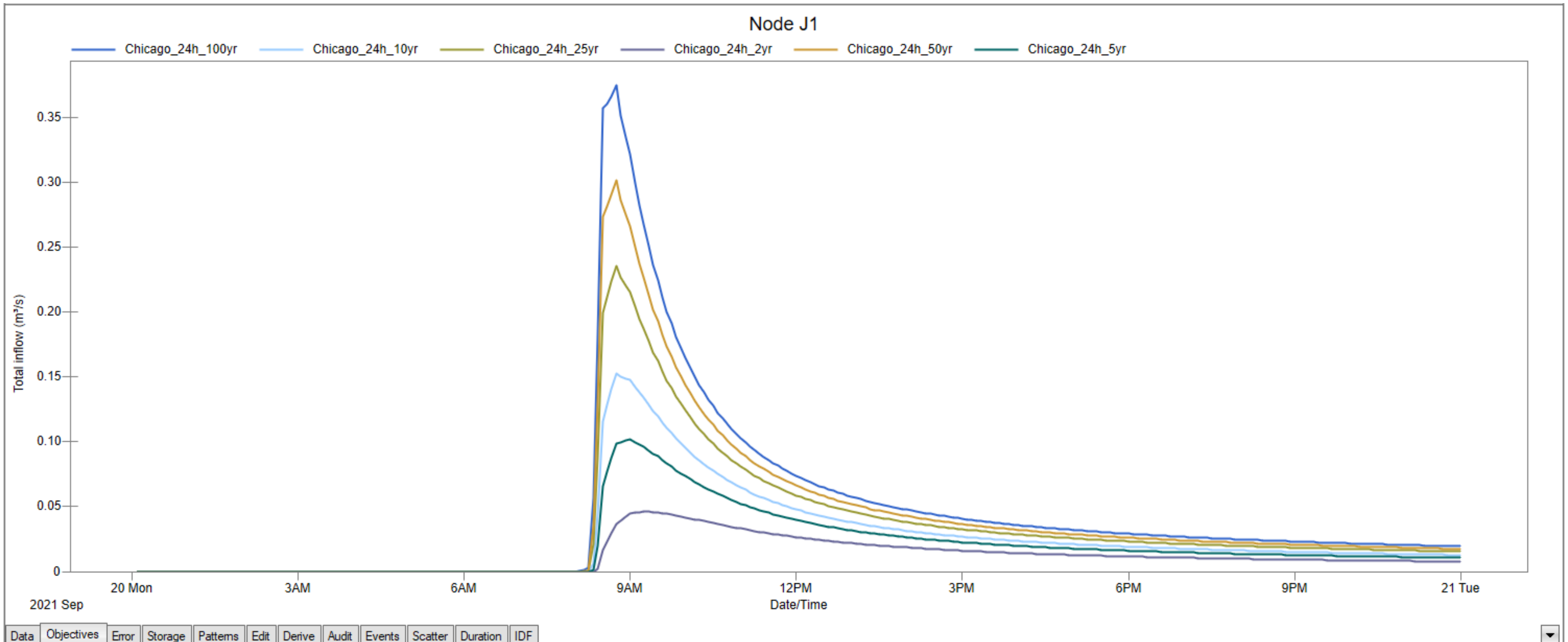
Table 1: Existing Conditions - Catchment Summary

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
107B	30	2.512	141.921	177	1.1	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
107B	2.512	141.921	177	1.1	0	0.013	0.13	1	5	25	OUTLET	100	30
200	1.8249	182.49	100	2	6.5	0.013	0.25	1	5	25	PERVIOUS	80	60
201	4.4451	296.34	150	3.5	31	0.013	0.25	1	5	25	PERVIOUS	30	73
202	2.5133	209.442	120	4	3.9	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	5.4423	388.736	140	2.5	16.8	0.013	0.25	1	5	25	PERVIOUS	30	75
300B	2.9175	442.045	66	2	29.4	0.013	0.25	1	5	25	PERVIOUS	30	75
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	4.6747	194.779	240	2.3	1.7	0.013	0.3	1	5	25	PERVIOUS	80	56

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



Objective functions for Total inflow (m³/s)

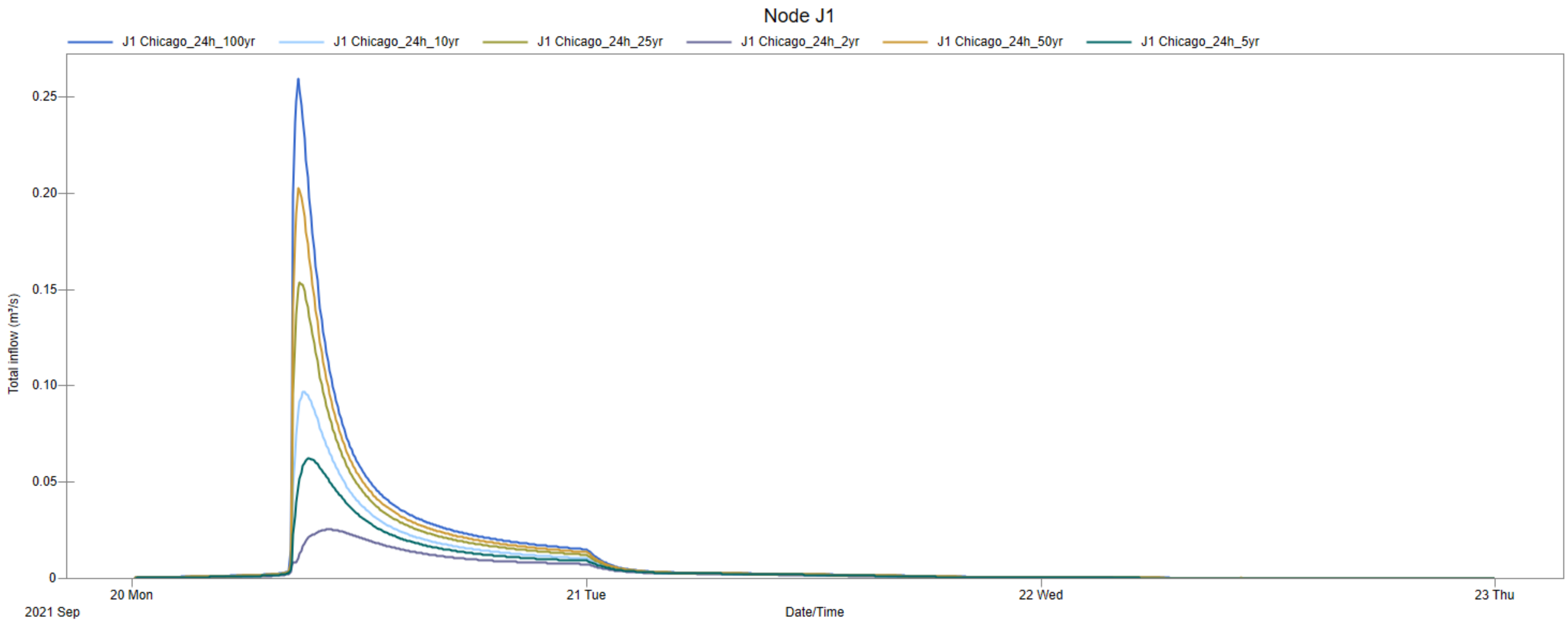
From Sep 19, 2021 10:53 PM to Sep 21, 2021 1:11 AM (26.31 hours)

	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 8:29 PM to Sep 23, 2021 3:35 AM (3.3 days)

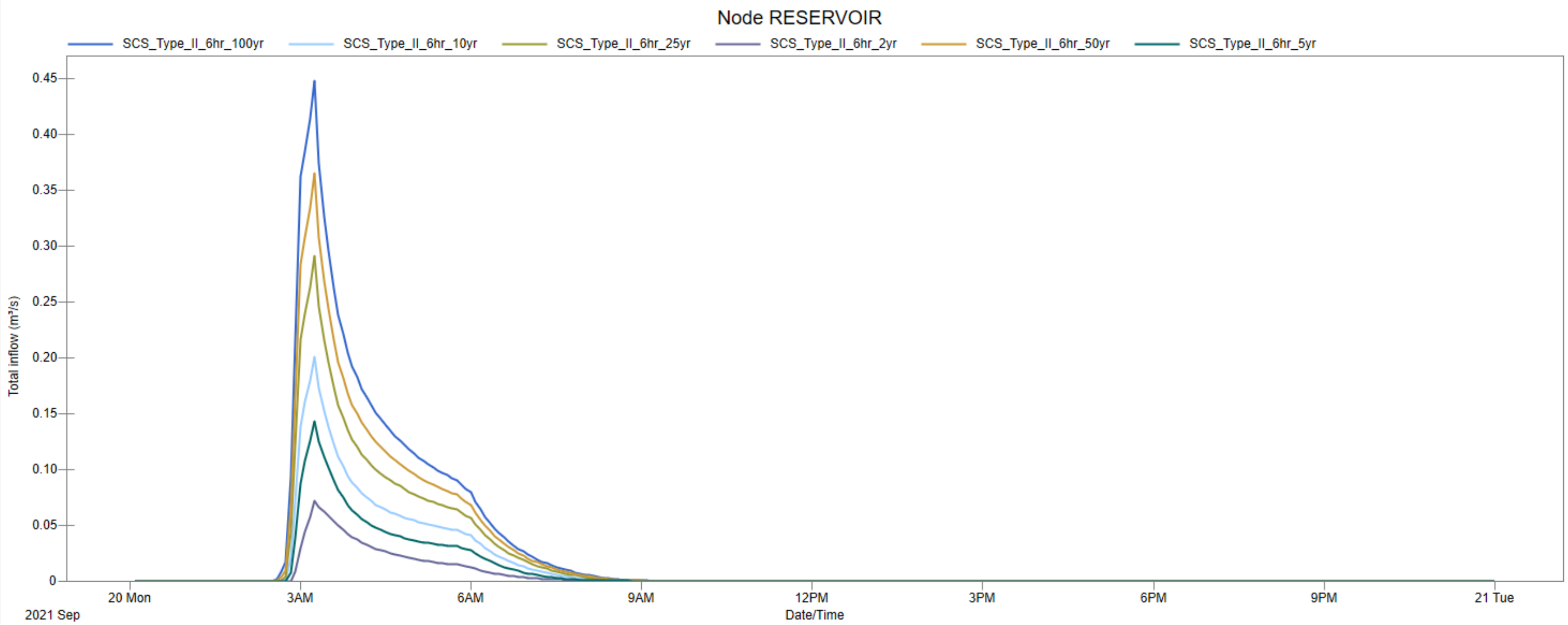
	J1 Chicago_24h_100yr	J1 Chicago_24h_10yr	J1 Chicago_24h_25yr	J1 Chicago_24h_2yr	J1 Chicago_24h_50yr	J1 Chicago_24h_5yr
Maximum Total inflow (m³/s)	0.2597	0.09683	0.1536	0.02514	0.2029	0.06205
Minimum Total inflow (m³/s)	0	0	0	0	0	0
Mean Total inflow (m³/s)	0.01206	0.007	0.009019	0.00349	0.01049	0.005543
Duration of Exceedances (h)	71.92	71.92	71.92	71.92	71.92	71.92
Duration of Deficits (h)	17.17	18.17	17.67	20.25	17.25	18.83
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	2	1	1	2	2	2
Volume of Exceedances (m³)	3123	1814	2338	903.5	2719	1435
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total inflow (m³)	3123	1814	2338	903.5	2719	1435

Total inflow (m³/s) Exceedance: 0

Total inflow (m³/s) Deficit: 0

Min interevent time (hrs): 0

Existing Peak Flows to Garden Hill Pond – 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)

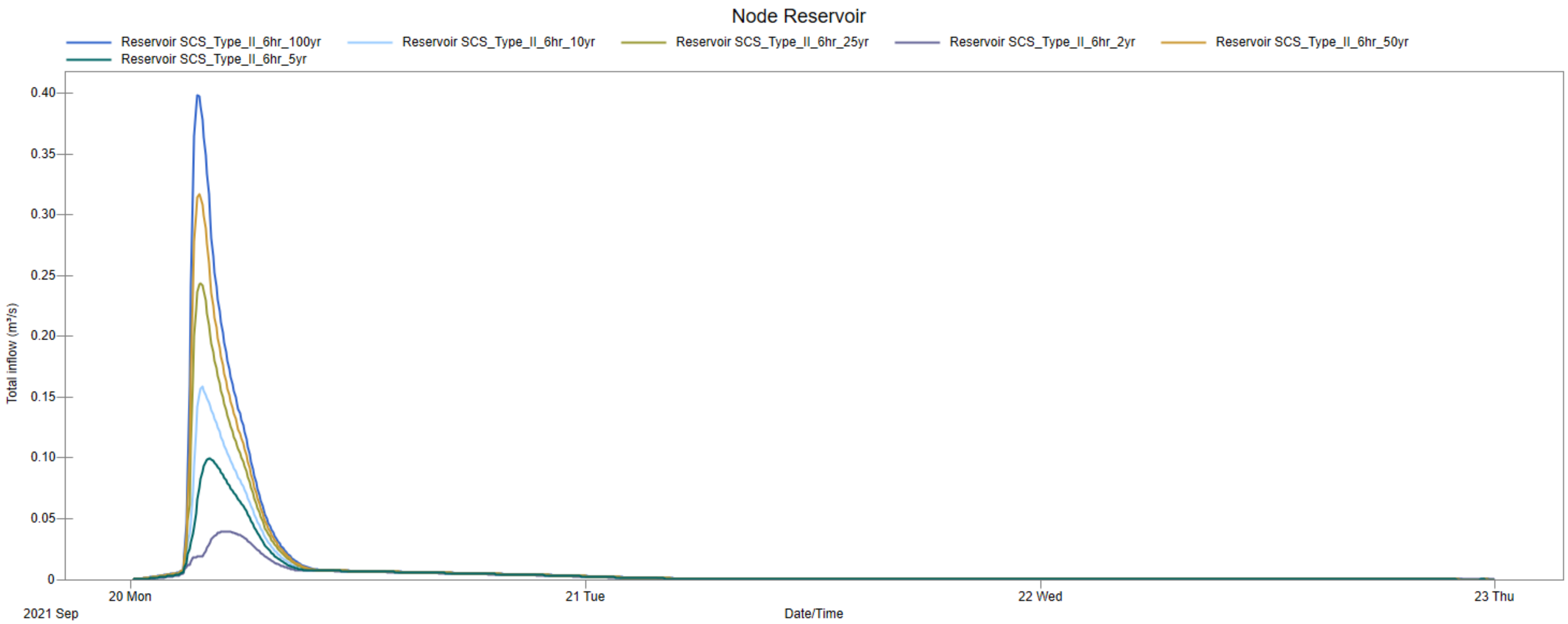
From Sep 19, 2021 10:53 PM to Sep 21, 2021 1:11 AM (26.31 hours)

	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 Garden Hill Pond – 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 8:29 PM to Sep 23, 2021 3:35 AM (3.3 days)

	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)	0.3985	0.1589	0.244	0.03968	0.3172	0.09921
Minimum Total inflow (m³/s)	3.98E-12	3.979E-12	3.979E-12	3.31E-08	3.98E-12	3.978E-12
Mean Total inflow (m³/s)	0.01366	0.00728	0.009732	0.003236	0.01164	0.005508
Duration of Exceedances (h)	71.92	71.92	71.92	71.92	71.92	71.92
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)	3546	1889	2525	839.8	3021	1429
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total inflow (m³)	3546	1889	2525	839.8	3021	1429

Total inflow (m³/s) Exceedance: 0

Total inflow (m³/s) Deficit: 0

Min interevent time (hrs): 0

APPENDIX O – WATER BALANCE METHODOLOGY AND CALCULATIONS

This section describes the components of the monthly and annual water balances.

Precipitation (P) - Average temperature and precipitation data for the period of 1981-2010 were obtained from Environment Canada for the Bowmanville Mostert monitoring station for use in the water balance. Total average precipitation rate for this station is 866.4mm/yr. The climate data used is provided in Appendix #.

Potential Evapotranspiration (PET) - PET is the amount of evapotranspiration that would occur if there were a sufficient water source available. The potential evapotranspiration rate is a function of average temperature and was calculated for the site using the Thornthwaite method as follows:

(Eqn. 1)

$$PET = 16d[\frac{10T}{I}]^a$$

Where:
I = annual monthly heat index
T = mean temperature for the month
d = daylight correction value
a = cubic function of I

Where the annual heat index (I) is the sum of the monthly heat indexes (i) which are calculated from the equation below:

(Eqn. 2)

$$i = (\frac{T}{5})^{1.514}$$

Where *a* is a cubic function of I and is calculated as follows:

(Eqn. 3)

$$a = 0.49 + 0.0000771\, I^2 + 0.000000675\ I^3$$

Precipitation surplus (P-PET)- Precipitation surplus is the moisture surplus (or deficiency) after potential evapotranspiration (PET) has been removed and thus is the difference between precipitation and potential evapotranspiration. When the precipitation surplus is negative there is a moisture deficit.

Actual Evapotranspiration (AET) - The actual evapotranspiration (AET) the evapotranspiration that will occur based on the actual precipitation available for the area which results in changes in the soil moisture storage. The AET for the site has been calculated in the monthly water balance provided below. The moisture surplus available after AET has been removed is the water available for infiltration and runoff.

Monthly Water Balance Analysis North 100

Soil Capacity (mm): 250

Land Description	
Cover:	Mature Forest
Slope:	
Soil:	Type A

MOE Infiltration Factors	
Topography	0.2
Soils	0.4
Cover	0.2
Total	0.8

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Total
Precipitation (mm) (P)	63.1	50.5	55.0	70.6	75.9	83.8	63.2	78.1	98.7	70.8	88.6	68.1	866
Potential Evapotranspiration (PET)	0.0	0.0	0.0	33.4	75.9	111.3	129.4	114.3	76.2	38.9	12.0	0.0	591
P-PET (mm)	63	51	55	37	0	-28	-66	-36	23	32	77	68	275
Accumulated Potential Water Loss (mm)	0	0	0	0	0	-28	-94	-130	0	0	0	0	
Soil Water Storage (mm)	250	250	250	250	250	223	191	148	171	202	250	250	
Change in Soil Moisture Storage(mm)	0	0	0	0	0	-27	-32	-43	23	32	48	0	
Actual Evapotranspiration (mm) (AET)	0	0	0	33	76	111	95	121	76	39	12	0	563.4
Moisture Deficit	0	0	0	0	0	1	34	-7	0	0	0	0	28
Moisture Surplus (P-PET -Chng. in S M Storage)	63	51	55	37	0	0	0	0	0	0	29	68	303
Potential Infiltration	50.48	40.4	44	29.8	0.0	0.0	0.0	0.0	0.0	0.0	23.2	54.48	242.4
Potential Surface Runoff	12.62	10.1	11	7.4	0.0	0.0	0.0	0.0	0.0	0.0	5.8	13.62	61

Monthly Water Balance Analysis South 101

Soil Capacity (mm): 75

Land Description	
Cover:	oderately Rooted Cro
Slope:	
Soil:	Type A

MOE Infiltration Factors	
Topography	0.2
Soils	0.4
Cover	0.1
Total	0.7

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Total
Precipitation (mm) (P)	63.1	50.5	55.0	70.6	75.9	83.8	63.2	78.1	98.7	70.8	88.6	68.1	866
Potential Evapotranspiration (PET)	0.0	0.0	0.0	33.4	75.9	111.3	129.4	114.3	76.2	38.9	12.0	0.0	591
P-PET (mm)	63	51	55	37	0	-28	-66	-36	23	32	77	68	275
Accumulated Potential Water Loss (mm)	0	0	0	0	0	-28	-94	-130	0	0	0	0	
Soil Water Storage (mm)	75	75	75	75	75	51	20	12	35	66	75	75	
Change in Soil Moisture Storage(mm)	0	0	0	0	0	-24	-31	-8	23	32	9	0	
Actual Evapotranspiration (mm) (AET)	0	0	0	33	76	108	94	86	76	39	12	0	524.4
Moisture Deficit	0	0	0	0	0	4	35	28	0	0	0	0	67
Moisture Surplus (P-PET -Chng. in S M Storage)	63	51	55	37	0	-24	-31	-8	23	32	77	68	342.0
Potential Infiltration	44	35	39	26	0	-17	-22	-6	16	22	54	48	239.4
Potential Surface Runoff	19	15	17	11	0	-7	-9	-2	7	10	23	20	103

WATER BALANCE CATCHMENT AREA DRAWING

LEGEND

— CATCHMENT BOUNDARY

100 — CATCHMENT ID

15 — AREA (ha)

NO.	DATE (D/M/Y)	REVISION	BY
–	–	–	–

NOT FOR CONSTRUCTION

MONUMENT GEOMATICS
MONUMENTGE.COM

MONUMENT PROJECT No.:	21–0135
DRAWN:	G.W.
CHECKED:	P.Q.
ENGINEER:	–
DATE:	23/08/2022
SCALES:	HORZ. 1:1500
SCALES:	VERT.

MITRAL LAND DEVELOPMENTS
GARDEN HILL ESTATES

MUNICIPALITY OF PORT HOPE
WATER BALANCE
CATCHMENT AREA DRAWINGS

DRAWING No.

WB-01

0 40 80 Meters

WATER BALANCE CALCULATIONS

1. **Pre-development water balance.**
2. **Post-development water balance.**
3. **Post-development with mitigation measure water balance.**

Pre-Development
Garden Hill Estates

Catchment Parameters	101	100	Total
	Pervious	Pervious	
Area (m2)	185100	181000	366100
Infiltration Factors			
Topography Infiltration Factor	0.15	0.15	
Soil Infiltration Factor	0.35	0.35	
Land Cover Infiltration Factor	0.1	0.1	
Total Infiltration Factor	0.6	0.6	
Actual Infiltration Factor	0.7	0.8	
Enhanced Infiltration Factor	0	0	
Runoff Coeficient	0.3	0.2	
Inputs (mm/yr)			
Precipitation	866.4	866.4	
Run-On	0	0	
Other Inputs	0	0	
Total Inputs	866.4	866.4	
Outputs (mm/yr)			
Precipitation Surplus	342.0	303.0	
Net Surplus	342.0	303.0	
Actual Evapotranspiration	524.4	563.4	
Infiltration	239.4	242.4	
Enhanced Infiltration	0.0	0.0	
Total Infiltration	239.4	242.4	
Runoff	102.6	60.6	
Adjusted Runoff	102.6	60.6	
Total Outputs	866.4	866.4	
Difference (Inputs - Outputs)	0	0	
Inputs (m3/yr)			
Precipitation	160370.6	156818.4	
Run-on	0.0	0.0	
Other Inputs	0.0	0.0	
Total Inputs	160370.6	156818.4	
Outputs (m3/yr)			
Precipitation Surplus	63297.1	54836.1	118133.2
Net Surplus	63297.1	54836.1	118133.2
Evapotranspirtation	97073.5	101982.3	199055.9
Infiltration	44307.4	43868.9	88176.2
Infiltration Features	0.0	0.0	0.0
Total Infiltration	44307.4	43868.9	88176.2
unadjusted Runoff	18989.7	10967.2	29956.9
Runoff	18989.7	10967.2	29956.9
Total Outputs	160370.6	156818.4	317189.0
Difference (Inputs-Outputs)	0.0	0.0	0.0

Post-Development No Mitigation
Garden Hill Estates

Evaporation Factor for Impervious = 0.2
* Evaporation from impervious Areas is assumed to be 20%

Catchment Parameters	101		100	Total
	Pervious	Impervious	Pervious	
Area (m2)	146500	38600	181000	366100
Infiltration Factors				
Topography Infiltration Factor	0.15	0	0.15	
Soil Infiltration Factor	0.35	0	0.35	
Land Cover Infiltration Factor	0.1	0	0.1	
Total Infiltration Factor	0.6	0	0.6	
Actual Infiltration Factor	0.7	0	0.8	
Enhanced Infiltration Factor	0	0	0	
Runoff Coeficient	0.3	1	0.2	
Inputs (mm/yr)				
Precipitation	866.4	866.4	866.4	
Run-On	0	0	0	
Other Inputs	0	0	0	
Total Inputs	866.4	866.4	866.4	
Outputs (mm/yr)				
Precipitation Surplus	342.0	866.4	303.0	
Net Surplus	342.0	866.4	303.0	
Actual Evapotranspiration	524.4	0.0	563.4	
Infiltration	239.4	0.0	242.0	
Infiltration Features	0.0	0.0	0.0	
Total Infiltration	239.4	0.0	242.0	
Runoff	102.6	866.4	61.0	
Adjusted Runoff	102.6	866.4	61.0	
Total Outputs	866.4	866.4	866.4	
Difference (Inputs - Outputs)	0	0	0	
Inputs (m3/yr)				
Precipitation	126927.6	33443.0	156818.4	
Run-on	0.0	0.0	0.0	
Other Inputs	0.0	0.0	0.0	
Total Inputs	126927.6	33443.0	156818.4	
Outputs (m3/yr)				
Precipitation Surplus	50097.4	33443.0	54836.1	138376.5
Net Surplus	50097.4	33443.0	54836.1	138376.5
Evapotranspirtation	76830.2	0.0	101982.3	178812.5
Infiltration	35068.2	0.0	43802.0	78870.2
Infiltration Features	0.0	0.0	0.0	0.0
Total Infiltration	35068.2	0.0	43802.0	78870.2
unadjusted Runoff	15029.2	33443.0	11034.1	59506.3
Runoff	15029.2	33443.0	11034.1	59506.3
Total Outputs	126927.6	33443.0	156818.4	317189.0
Difference (Inputs-Outputs)	0.0	0.0	0.0	0.0

Post-Development With Mitigation
Garden Hill Estates

Evaporation Factor for Impervious = 0.2
* Evaporation from impervious Areas is assumed to be 20%

Catchment Parameters	101			100	Total
	Pervious	Paved	Roofs	Pervious	
Area (m2)	146500	26259	12341	181000	366100
Infiltration Factors					
Topography Infiltration Factor	0.15	0	0	0.15	
Soil Infiltration Factor	0.35	0	0	0.35	
Land Cover Infiltration Factor	0.1	0	0	0.1	
Total Infiltration Factor	0.6	0	0	0.6	
Actual Infiltration Factor	0.7	0	0	0.8	
Enhanced Infiltration Factor		0.2	1		
Runoff Coeficient	0.3	0.8	0	0.2	
Inputs (mm/yr)					
Precipitation	866.4	866.4	866.4	866.4	
Run-On	0	0	0	0	
Other Inputs	0	0	0	0	
Total Inputs	866.4	866.4	866.4	866.4	
Outputs (mm/yr)					
Precipitation Surplus	342.0	693.1	693.1	303.0	
Net Surplus	342.0	693.1	693.1	303.0	
Actual Evapotranspiration	524.4	173.3	173.3	563.4	
Infiltration	239.4	0.0	0.0	242.0	
Infiltration Features	0.0	138.6	693.1	0.0	
Total Infiltration	239.4	138.6	693.1	242.0	
Runoff	102.6	693.1	693.1	61.0	
Adjusted Runoff	102.6	554.5	0.0	61.0	
Total Outputs	866.4	866.4	866.4	866.4	
Difference (Inputs - Outputs)	0	0	0	0	
Inputs (m3/yr)					
Precipitation	126927.6	22750.8	10692.2	156818.4	
Run-on	0.0	0.0	0.0	0.0	
Other Inputs	0.0	0.0	0.0	0.0	
Total Inputs	126927.6	22750.8	10692.2	156818.4	
Outputs (m3/yr)					
Precipitation Surplus	50097.4	18200.6	8553.8	54836.1	131687.9
Net Surplus	50097.4	18200.6	8553.8	54836.1	131687.9
Evapotranspirtation	76830.2	4550.2	2138.4	101982.3	185501.1
Infiltration	35068.2	0.0	0.0	43802.0	78870.2
Infiltration Features	0.0	3640.1	8553.8	0.0	12193.9
Total Infiltration	35068.2	3640.1	8553.8	43802.0	91064.1
unadjusted Runoff	15029.2	18200.6	8553.8	11034.1	52817.7
Runoff	15029.2	14560.5	0.0	11034.1	40623.8
Total Outputs	126927.6	22750.8	10692.2	156818.4	317189.0
Difference (Inputs-Outputs)	0.0	0.0	0.0	0.0	0.0