PENTA PROPERTIES INC.
PIN OAK PROPERTY
CITY OF NIAGARA FALLS

STORMWATER MANAGEMENT REPORT

Prepared By:

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Hamilton, ON L8H 7T4

Date: April 2023 **MCI FILE: P18002**

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1.0 - INTRODUCTION

The property under study is 0 Pin Oak Drive (13.5 ha) owned by Penta Properties Inc (Penta). The property is bounded by Kalar Road to the west, Niagara Peninsula Energy yard to the north, Pin Oak Drive to the east, and an existing residential home to the southwest.

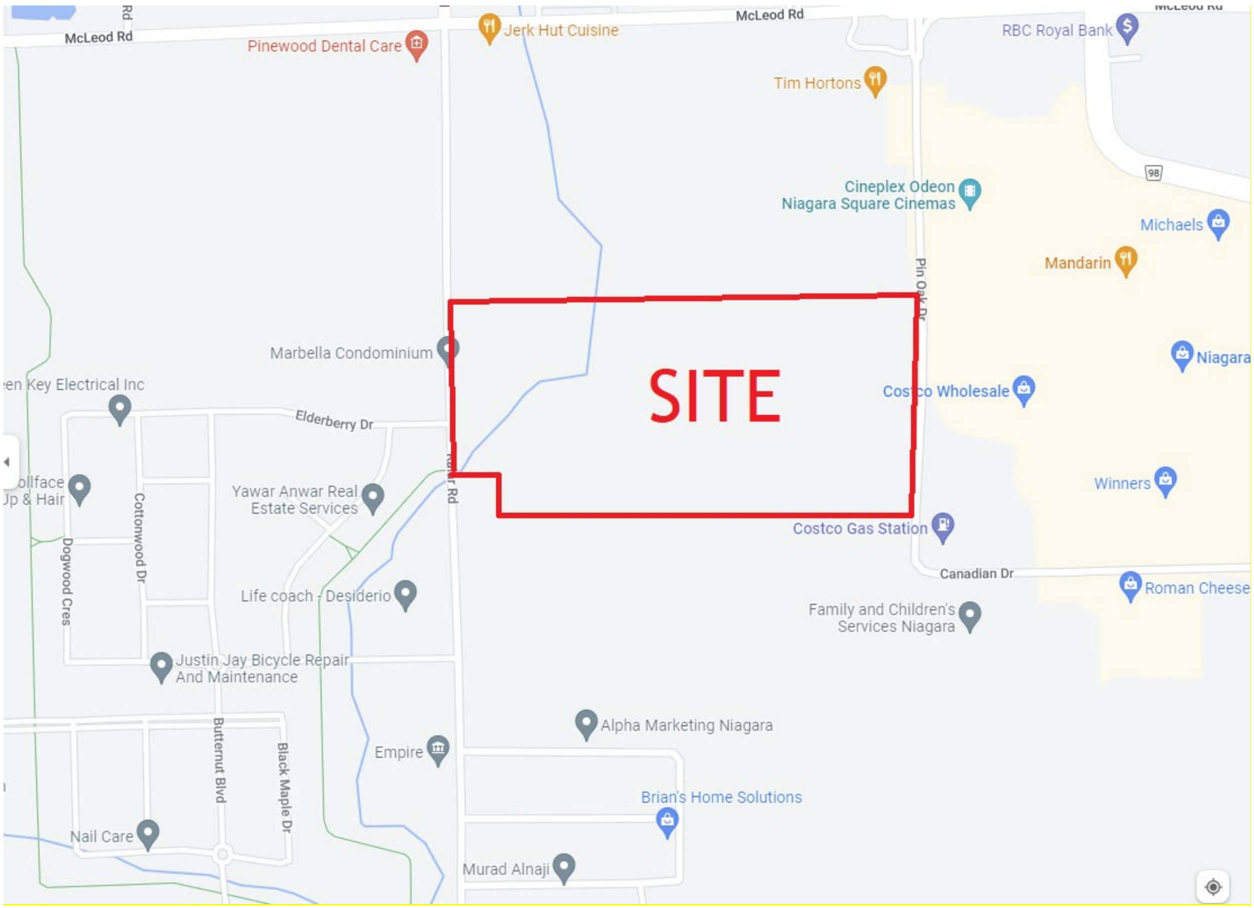


Figure 1 Location of Property

Metropolitan Consulting Inc. (MCI) has been retained by Penta to provide a Stormwater Management Report ("SWM Report") for the proposed development. This report is being submitted in support of a proposed draft plan of subdivision application for the subject lands. This report will outline the minimum requirements to service the site with respect to stormwater management to meet the Niagara Region requirements for land development. Refer to the *Water and Wastewater Generation Report* (Metropolitan Consulting Inc., Dec 2022) for more information regarding the strategy for sanitary and water servicing.

A creek flows through the west side of the property, and there are multiple protected forested areas that must be preserved. As per the hydrological assessment, there will be no significant change in the hydrology of the wetland areas. Refer to the Environmental Impact Study prepared by GEI Consultants for information regarding the strategy for meeting the environmental requirements.

1.1 - PROPOSED DEVELOPMENT

It is proposed to construct a residential plan of subdivision which consists of 29 blocks, including freehold townhomes, a proposed townhouse site plan block, a medium density block sized for 55 units, for a total of 219 residential units, and one (1) new Stormwater Management Pond. In addition, it is proposed to construct a new public road ("Street A") through the site, connecting Kalar Road in the west to Pin Oak Drive in the east.

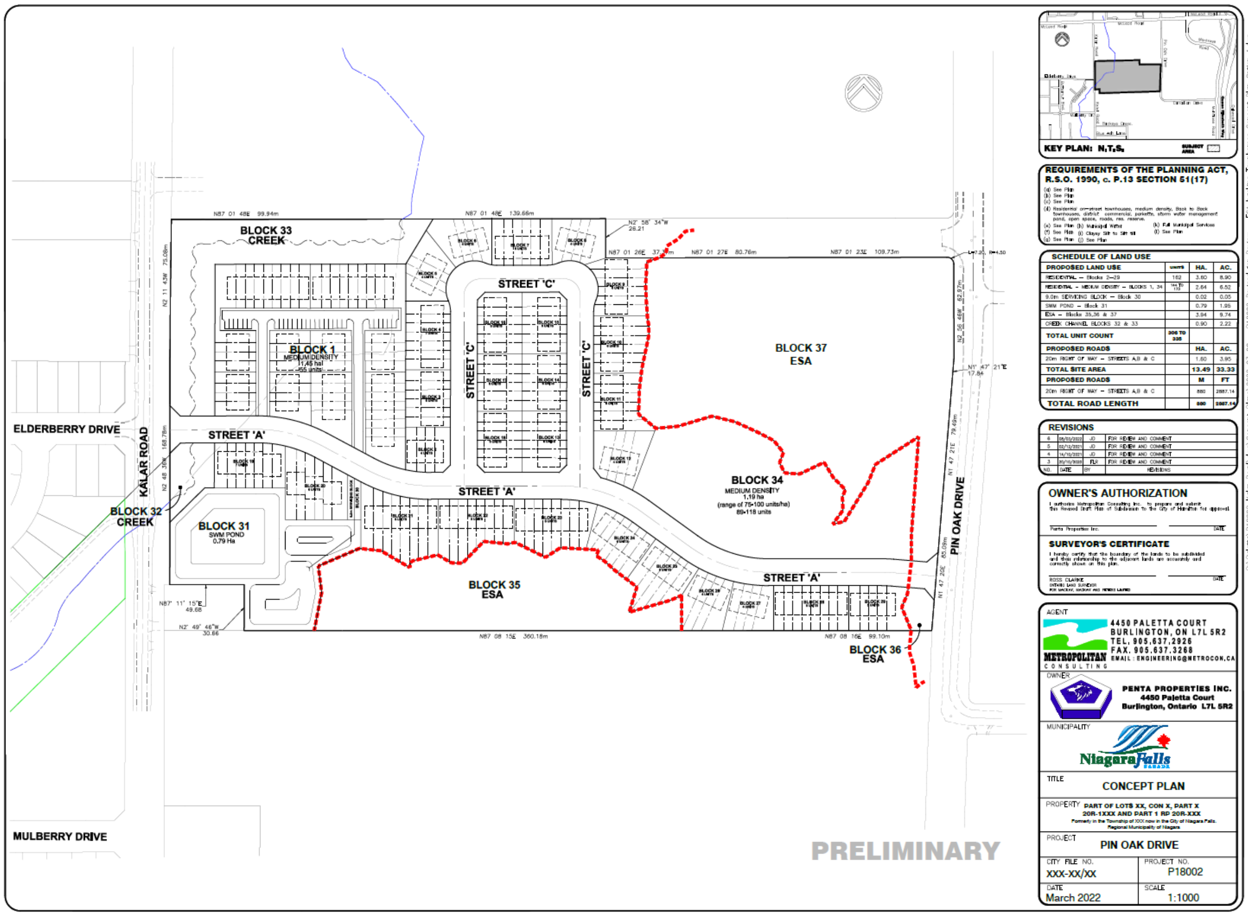


Figure 1-2 Proposed Development

2.0 – DRAINAGE CONDITIONS

2.1 - Existing Drainage Conditions

Figure 2 Existing Conditions

Figure 2 above shows the existing drainage conditions of the development area at the site. There are no existing storm sewer connections on this property. A portion of Warren Creek runs through the northwest corner of the property. There is an existing 525mm storm sewer in Kalar Road to the west of the site.

2.2 - Proposed Drainage Conditions

Design criteria for the recommended SWM facilities are defined in city design criteria and can be summarized as follows:

- Quality treatment to Enhanced level.
- Storage based on MOE SWM Planning and Design Manual, 2003 for enhanced protection 80% long-term Suspended Solids removal.
- Extended detention storage for Pond 25mm storm with 48hr drawdown; and
- Quantity storage to control post-development peak flows during the 2-Year through 100-Year storms

The total proposed development area is **8.67ha**. This includes the proposed drainage area of **7.97ha** to the Pond and **0.70ha** which will drain uncontrolled directly to Warren Creek. The proposed development areas are outlined in Figures 3 and Figure 4, and summarized in the following table:

Table 1 PROPOSED DRAINAGE AREA			
ID	**Area (ha)**	**IMP (%)**	**A x IMP**
2	0.655	51	0.3341
3	0.252	69	0.1739
4	0.287	60	0.1722
5	0.783	55	0.4307
6	0.037	100	0.0370
7	0.337	84	0.2831
8	0.209	78	0.1630
9	0.294	81	0.2381
10	1.091	64	0.6982
11	1.367	56	0.7655
12	0.396	84	0.3326
13	0.239	79	0.1888
Z	1.199	90	1.0791
Total:	**7.15**	**69**	**4.90**

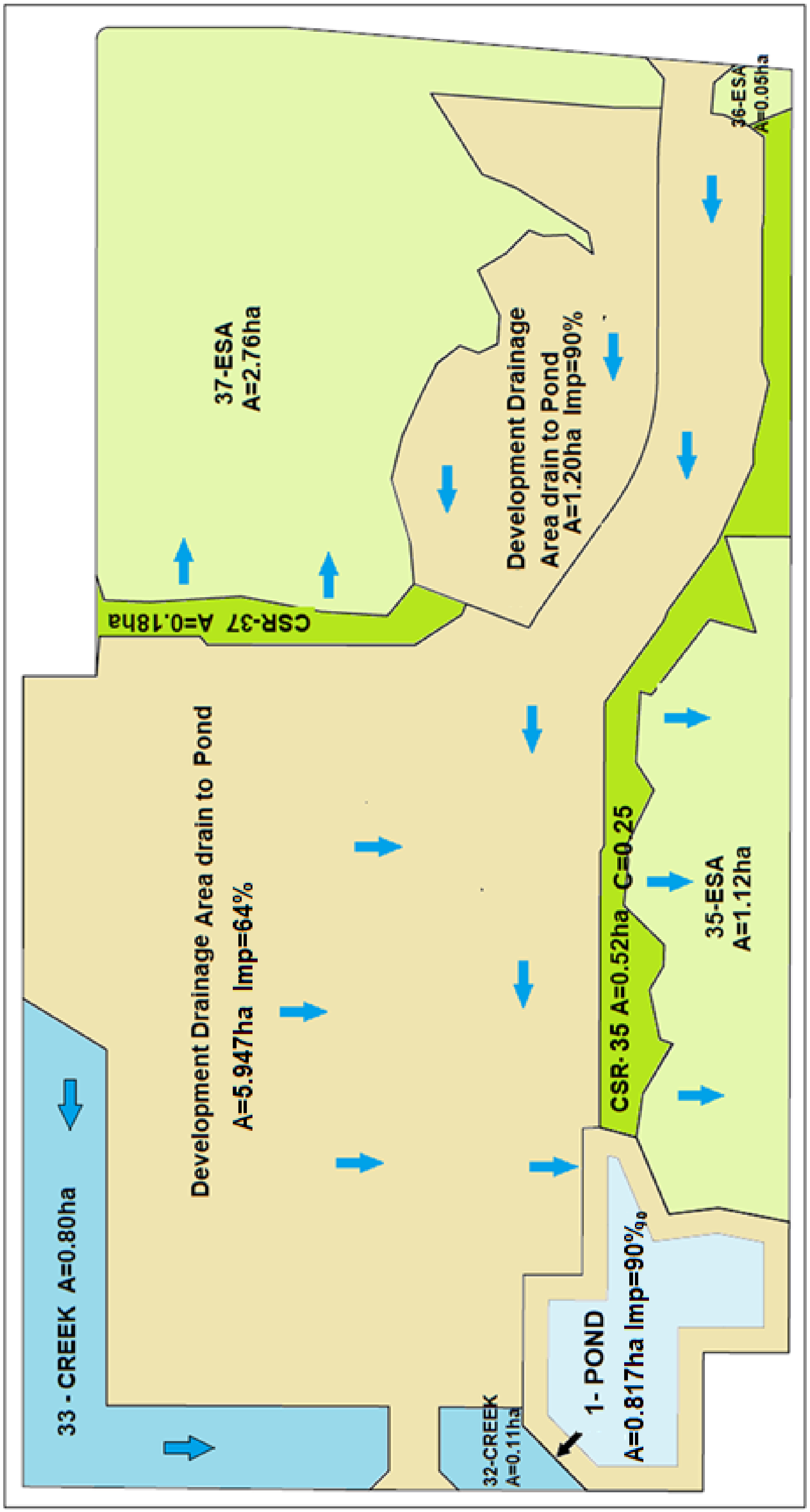


Figure 3 Total drainage area (Incl. Area 35,36, & 37 – ESA)

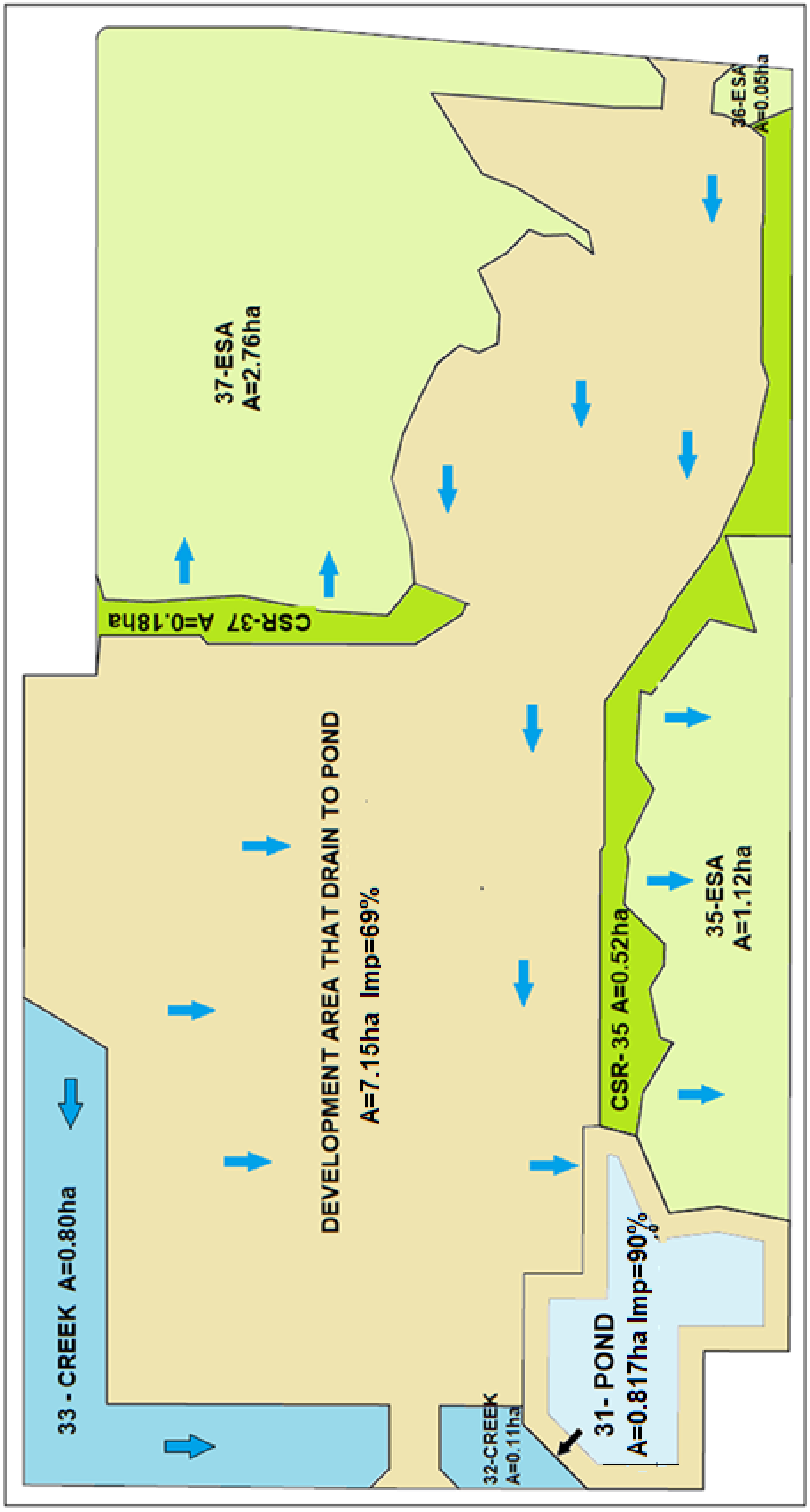


Figure 4 Controlled and Uncontrolled Flow from Development Area

3.0 – STORMWATER MANAGEMENT

The development site consists of residential areas. The proposed Pond will be used to control post-development flows to pre-development runoff from the residential area. A sediment forebay is proposed for quality control of the runoff from the residential area.

The SWM Pond design is based on the design criteria as laid out by the Ministry of Environment, Conservation, and Stormwater Management Planning and the Design Manual by the City of Niagara Falls Criteria and Guidelines for Stormwater Infrastructure Design. Table 3-1 shows the Niagara Falls IDF curve data, which has been used as the basis for designing the proposed Pond.

Table 2 Niagara Falls IDF Curve

Return Period (Year)	A	B	C
2-Year	521.97	5.28	0.7590
5-Year	719.50	6.34	0.7687
10-Year	870.09	6.81	0.7738
25-Year	1020.69	7.29	0.7790
50-Year	1142.00	7.50	0.7800
100-Year	1264.57	7.72	0.7814

In order to determine the post-development peak flows (both controlled and uncontrolled, a model of the post-development flows has been created with SWMHYMO for each catchment area.

Bottom lining is required to the bottom of the proposed Pond. For more design details for the Pond, refer to the servicing drawings included in the Appendix at the back of this report.

A total drainage area of **7.97ha** with an imperviousness of **71%** will drain to the proposed Pond, while a total drainage area **0.70ha** with an imperviousness of 0% will drain uncontrolled directly to Warren Creek. These results are summarized in Table 3 and 4 below:

Table 3 DRAINAGE AREA TO POND			
ID	**Area (ha)**	**IMP (%)**	**A x IMP**
2	0.655	51	0.3341
3	0.252	69	0.1739
4	0.287	60	0.1722
5	0.783	55	0.4307
6	0.037	100	0.0370
7	0.337	84	0.2831
8	0.209	78	0.1630
9	0.294	81	0.2381
10	1.091	64	0.6982
11	1.367	56	0.7655
12	0.396	84	0.3326
13	0.239	79	0.1888
Z	1.199	90	1.0791
Total:	**7.15**	**69**	**4.90**

Table 4 DRAINAGE AREA INCLUDING POND			
ID Area	**Area (ha)**	**Imp (%)**	**A*Imp (ha)**
Development	7.150	69%	4.898
1 -POND	0.817	90%	0.735
TOTAL:	**7.967**	**71%**	**5.633**

Table 5 summarizes the drainage areas that contribute to the proposed Pond and the areas that flow uncontrolled to Warren Creek:

Table 5 TOTAL DEVELOPMENT AREA

ID Area	Area (ha)	Imp (%)	A*Imp (ha)
Development	7.150	69%	4.898
1-POND	0.817	90%	0.735
CSR-37	0.180	0.00%	0.000
CSR-35	0.520	0.00%	0.000
TOTAL:	**8.667**	**65%**	**5.633**

Figure 5 below shows the drainage areas that contribute to the proposed Pond and the areas that flow uncontrolled to Warren Creek.

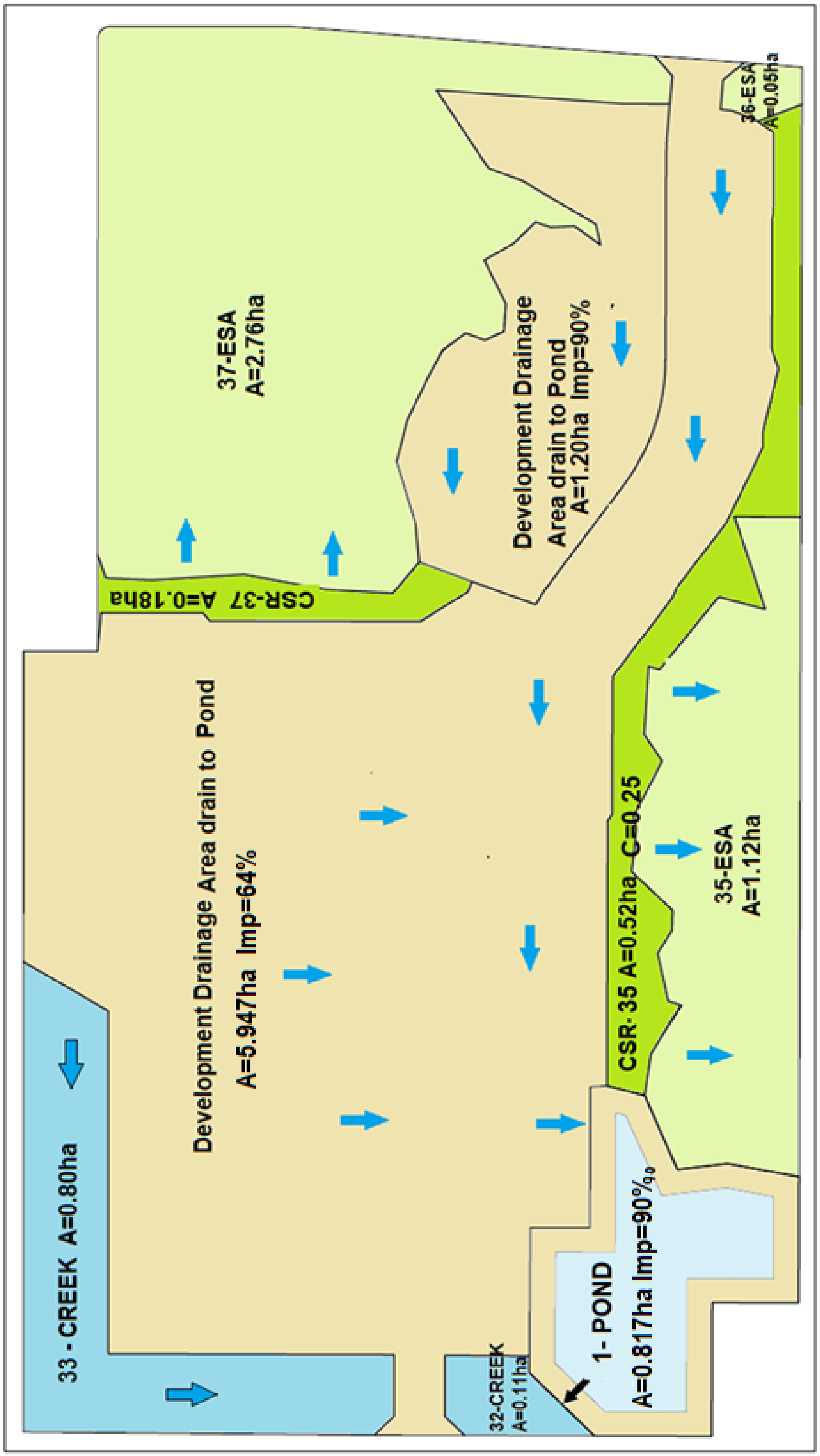


Figure 5 Total Drainage Area

3.1 – Minor Storm Drainage System

The minor storm drainage system shall be designed to convey stormwater runoff for the 1 in 5-year return period storm event, thereby providing safe and convenient use of the streets, parking lots, and other areas. Components of the minor storm drainage system could include:

- Swales, subsurface interceptor drains, curb and gutters, catchbasins, manholes, pipes or conduits and service lateral lines in those areas where a piped storm drainage system is required.

The total post development design drainage area **A=8.67ha** consists of the following areas:

- A=7.97ha drainage area including area of proposed Pond
- A=0.70ha drains uncontrolled to Warren Creek

Minor flows from post-development Pond area **of A= 7.97 ha** and an imperviousness of **71%** drain to the proposed stormwater management facility via storm sewers. Sizing calculations for storm sewer system were conducted using the rational method for the 5-year storms, using Niagara Falls IDF curves.

The minor system flows will discharge to the sediment forebay of proposed Pond.

3.2 – Target Release Rate

The target release rates for proposed Pond were set to controlled post to predevelopment flow. Table 3-5 below summarizes the pre-development flows from the uncontrolled drainage areas:

Table 6 Pre-Development flow from Area 35 and 37							
CSR - 35 (A=0.52ha) IDF Niagara Falls				CSR - 37 (A=0.18ha) IDF Niagara Falls			Total Uncontrolled Flow
Storm Events	Q(m3/s)	V(m3)		Storm Events	Q(m3/s)	V(m3)	Q(m3/s)
25mm	0.007	22		25mm	0.002	8	0.009
2-Year	0.010	35		2-Year	0.004	12	0.014
5-Year	0.017	57		5-Year	0.006	20	0.023
10-Year	0.023	75		10-Year	0.008	26	0.031
25-Year	0.029	94		25-Year	0.010	32	0.039
50-Year	0.035	113		50-Year	0.012	39	0.047
100-Year	0.042	132		100-Year	0.014	46	0.056
Regional	0.068	1065		Regional	0.024	369	0.092

Table 7 below summarizes the allowable release rate from the proposed Pond:

Table 7 Allowable Release from Proposed Pond

Result from SWHYMO (Pre-Development Flow) for Drainage area A=8.67ha Tp=0.65hr using IDF Niagara Falls				Total Uncontrolled Flow	Allowed release from Pond
Storm Events	**Q(m3/s)**	**Q(m3/s/ha)**	**V(m3)**	**Q(m3/s)**	**Q(m3/s)**
25mm	**0.050**	0.0063	362	**0.009**	**0.041**
2-Year	**0.078**	0.0098	581	**0.014**	**0.064**
5-Year	**0.128**	0.0161	**942**	**0.023**	**0.105**
10-Year	**0.172**	0.0216	1252	**0.031**	**0.141**
25-Year	**0.217**	0.0272	**1565**	**0.039**	**0.178**
50-Year	**0.262**	0.0329	1876	**0.047**	**0.215**
100-Year	**0.308**	0.0386	**2193**	**0.056**	**0.252**
Regional	**0.931**	0.1168	**17760**	**0.092**	**0.839**

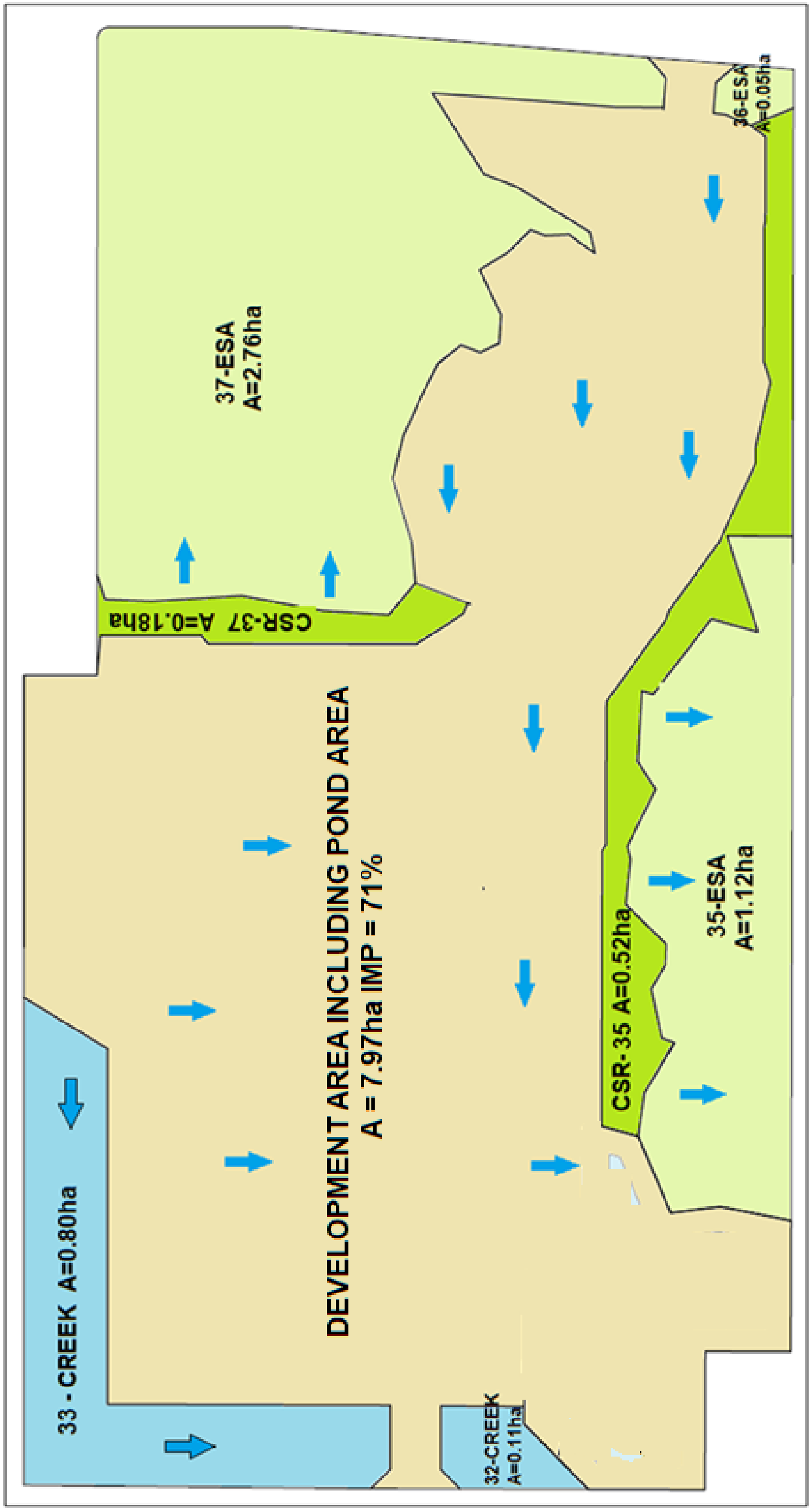


Figure 6 Drainage Areas for Controlled and Uncontrolled Flow

Table 8 and Figure 6 show the uncontrolled flow and controlled flow from the proposed Pond and the required storage for controlled flow.

Table 8 - Result from SWHYMO for Drainage area A=7.97ha with Imp=71% using IDF Niagara Falls

Storm Events	Q(m^3/s)	V(m^3)	Releasing Q(m^3/s)	Required Storage (m^3)	Allowed Releasing Q(m^3/s)
25mm	0.588	1388	0.011.	1262	0.041
2-Year	0.716	1859	0.043	1563	0.064
5-Year	**0.983**	**2533**	**0.095**	**1851**	0.105
10-Year	1.185	3052	0.129	2137	0.141
25-Year	**1.385**	**3542**	**0.168**	**2402**	0.178
50-Year	1.567	4004	0.201	2666	0.215
100-Year	**1.751**	**4456**	**0.239**	**2913**	0.252

As seen in the Table above, the controlled flow is less than the allowable release rate for the drainage area of 7.97ha.

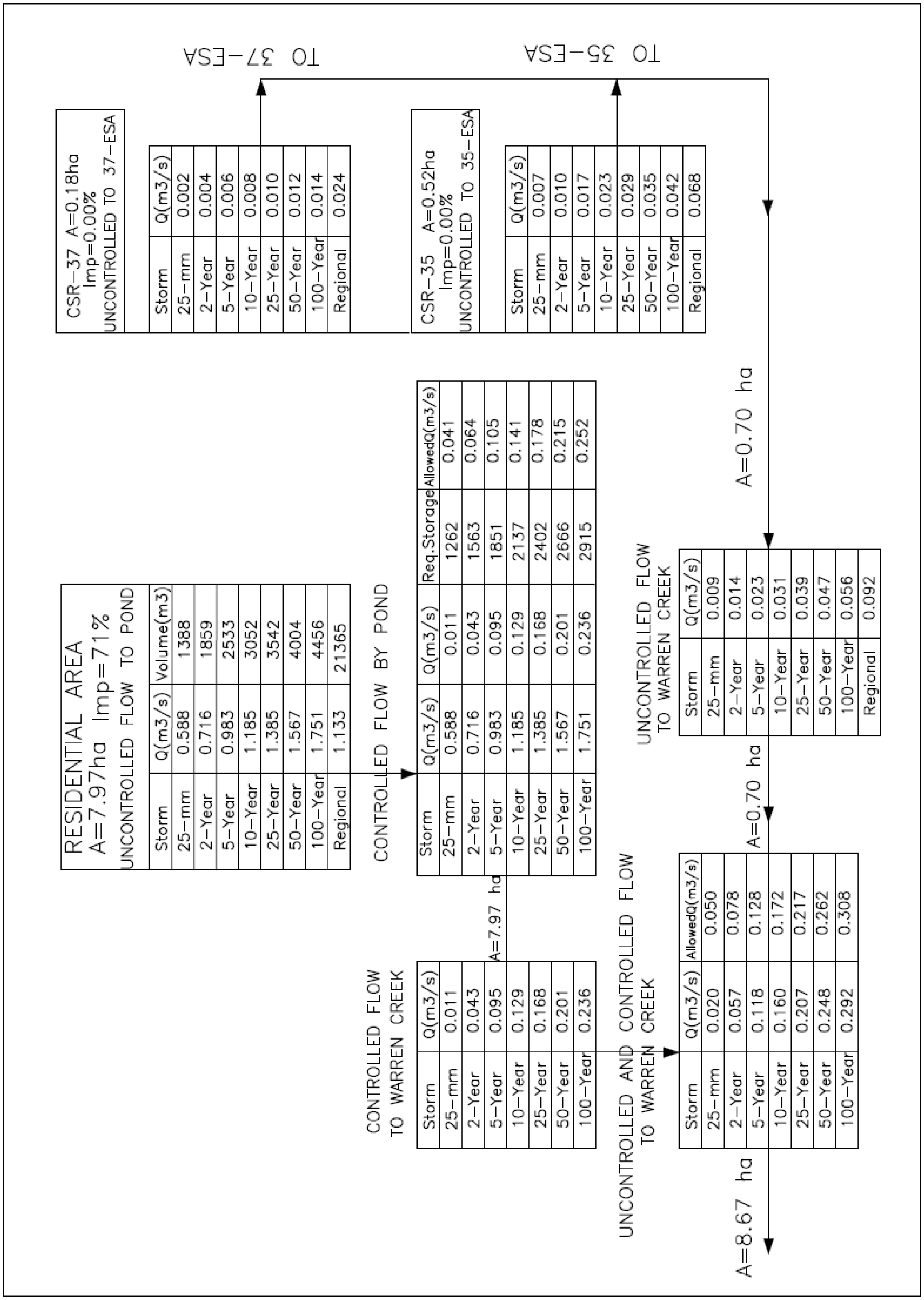


Figure 7 Schematic of Controlled and Uncontrolled Flow

3.3 – Permanent Pool

The permanent pool contributes to the quality control for post development runoff. The permanent pool provides a buffer for dilution of runoff during rainfall events prior to draining to the quantity cell of the proposed pond.

Sediments within the permanent pool have additional time to settle out between rainfall events. Sizing for the permanent pool can be found in Appendix A.

The proposed Pond permanent pool has been sized based on MECP Guidelines to provide enhanced protection (i.e., 80% TSS removal). The required permanent pool volumes are based on the area and imperviousness of the contributing lands.

Sizing of Permanent Pool

Total drainage area contributing to Pond.
A=7.97ha @ 71% Imperviousness
Storage volume for 80% impervious = 226 m^3/ha
Required Storage Volume: (226 - 40) = 186 m^3/ha
Required the Permanent Pool Storage: 7.97 x 186= **1482 m^3** **Provided 2493m^3**

Table 9 - Total Permanent Pool Storage

Required Storage	1,482	m^3
Total Permanent Pool Provided	2,493	m^3

The sediment forebay, at the inlet of the pond, will settle out larger particles before they enter the main pond area. This facilitates easier maintenance of the pond as the majority of the sediment will be accumulated in one area.

Minor system flows contributing to the Pond drainage area of 7.97ha will discharge to the sediment forebay.

Detailed design calculations for the sizing of the sediment forebay can be found in Appendix A.

3.4 – Extended Detention Storage

Extended detention storage is the first level of active (fluctuating) storage within the pond. Storage in this range is released slowly to provide increased detention time and reduce downstream erosion effects caused by frequent rainfall events as well as providing additional settling time to remove sediments. Additionally, this will help maintain base flows in the downstream watercourse for a longer time period following storm events.

The City of Niagara Falls recommended that Erosion Control Storage (Extended Detention) should be calculated as per MECP Design criteria:

- 48-hour detention time for the volume captured during the 25mm, 4-hour storm event

Detention time has been calculated using modified equation 4.11 from the SWM Planning and Design Manual MECP (2003).

Table 10 and Figure 8 below show that for a release rate of 0.011m^3/s the required detention time of 48hours in is accordance with the design criteria MOE and city Niagara Falls.
Result from SWHYMO showing releasing rate for 25mm storm Q=0.011m^3/s with required storage of 1,262m^3**.**

Table 10 Pond 6 Detention Time Summary

Description	Stage(m)	Total Outflow (m^3/s)	Active Storage (m^3)	Increment Time (sec)	Total Time (sec)	Total Time (hr)
Extended Detention	**178.85**	**0.0115**	**1318**	**17637**	**4.9**	**48**
	178.8	0.0105	1114	37167	10.32	43
	178.7	0.0082	722	45639	12.68	33
	178.6	0.0048	349	73419	20.39	20
Permanent Pool	**178.50**	**0.00**	**0.00**	**0.00**	**0.00**	**0.00**

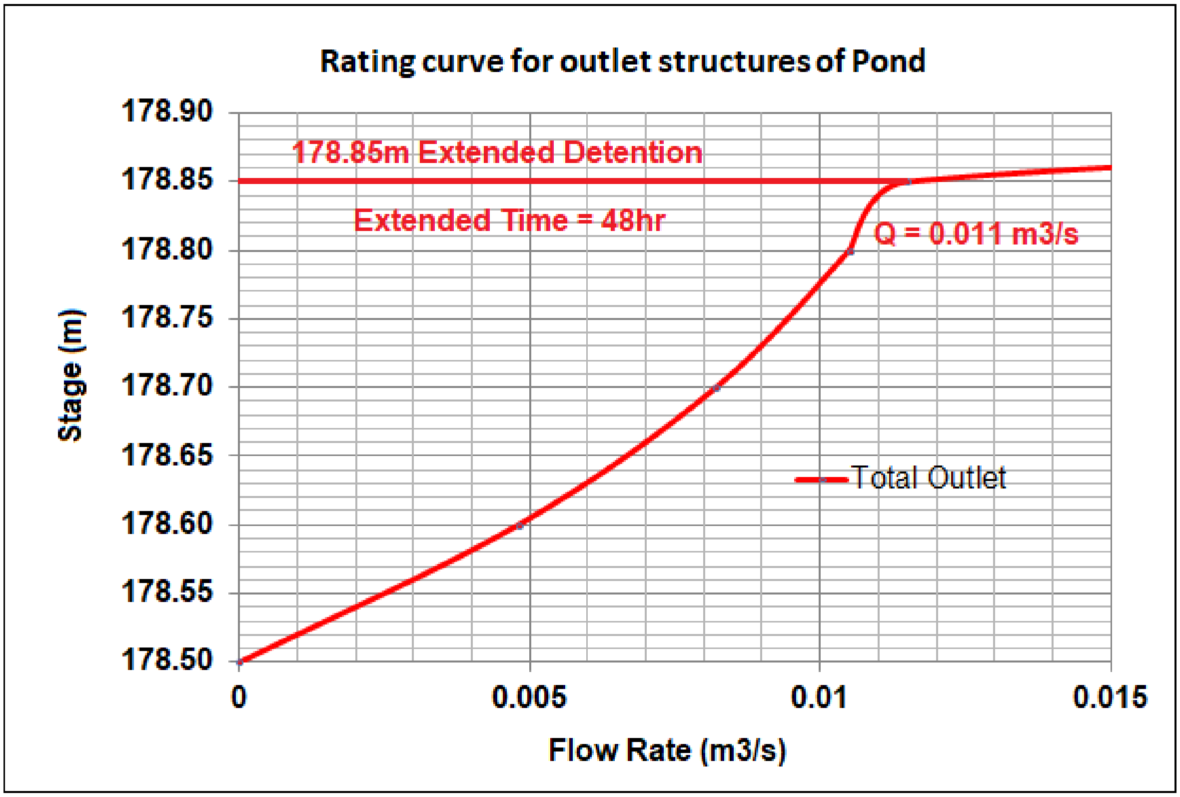


Figure 8 Pond Rating Curve

3.5 – Outlet Structure

There will be one Outlet structures from Pond to Channel (Creek).
The outlet discharges to the Channel (Creek) and consists of the following structures:

- Reversed slope pipe D=300mm to MH-00 with orifice D=98mm at invert 178.50m (Permanent Pool).
- DICB -1 at invert elevation 178.85m.
- DICB-1 lead pipe diameter (D) = 450mm and orifice diameter (D) = 260mm with invert elevation 178.50m.

- DICB -2 at invert elevation 179.00m.
- DICB-2 lead pipe diameter (D) = 450mm and orifice diameter (D) = 180mm with invert elevation 178.50m.

- DICB -3 at invert elevation 179.22m.
- DICB-3 lead pipe diameter (D) = 450mm and orifice diameter (D) = 180mm with invert elevation 178.50m.

- Outlet pipe to channel (Creek) by diameter of D=525mm; and
- Spillway B=2.00m.

3.6 – Flood Control

Additional active storage is provided above the extended detention storage to meet target release rates for the 5 through 100-year storm events.

The SWMHYMO model was utilized to assess the proposed development and control measures. SWMHYMO is a single event hydrologic model that is based on unit hydrograph theory.

As shown in the tables and figures below, the controlled flow from the Proposed Pond is considerably less than the predevelopment release rate.

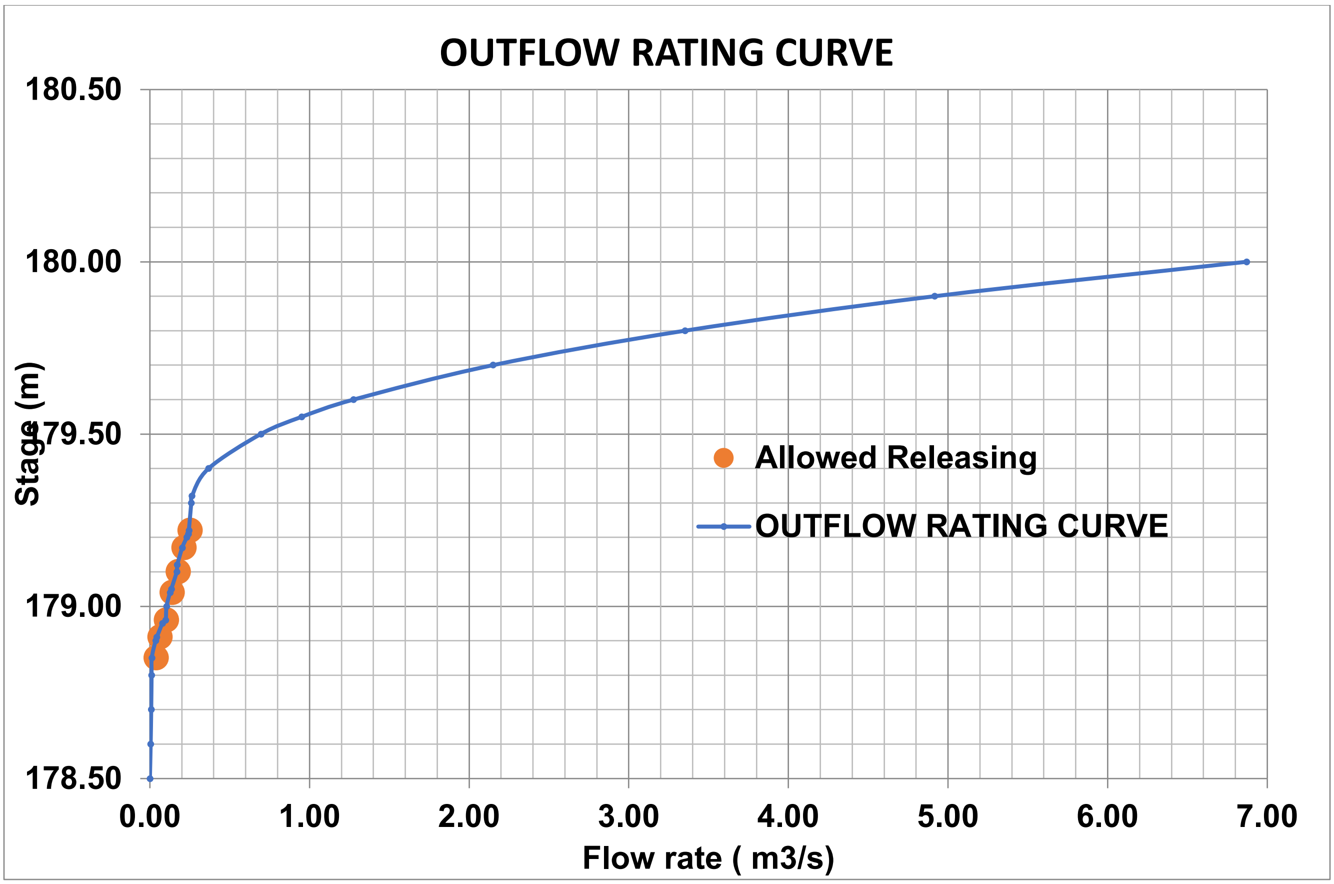


Figure 9 Pond Rating Curve (Outlet to Channel (Creek))

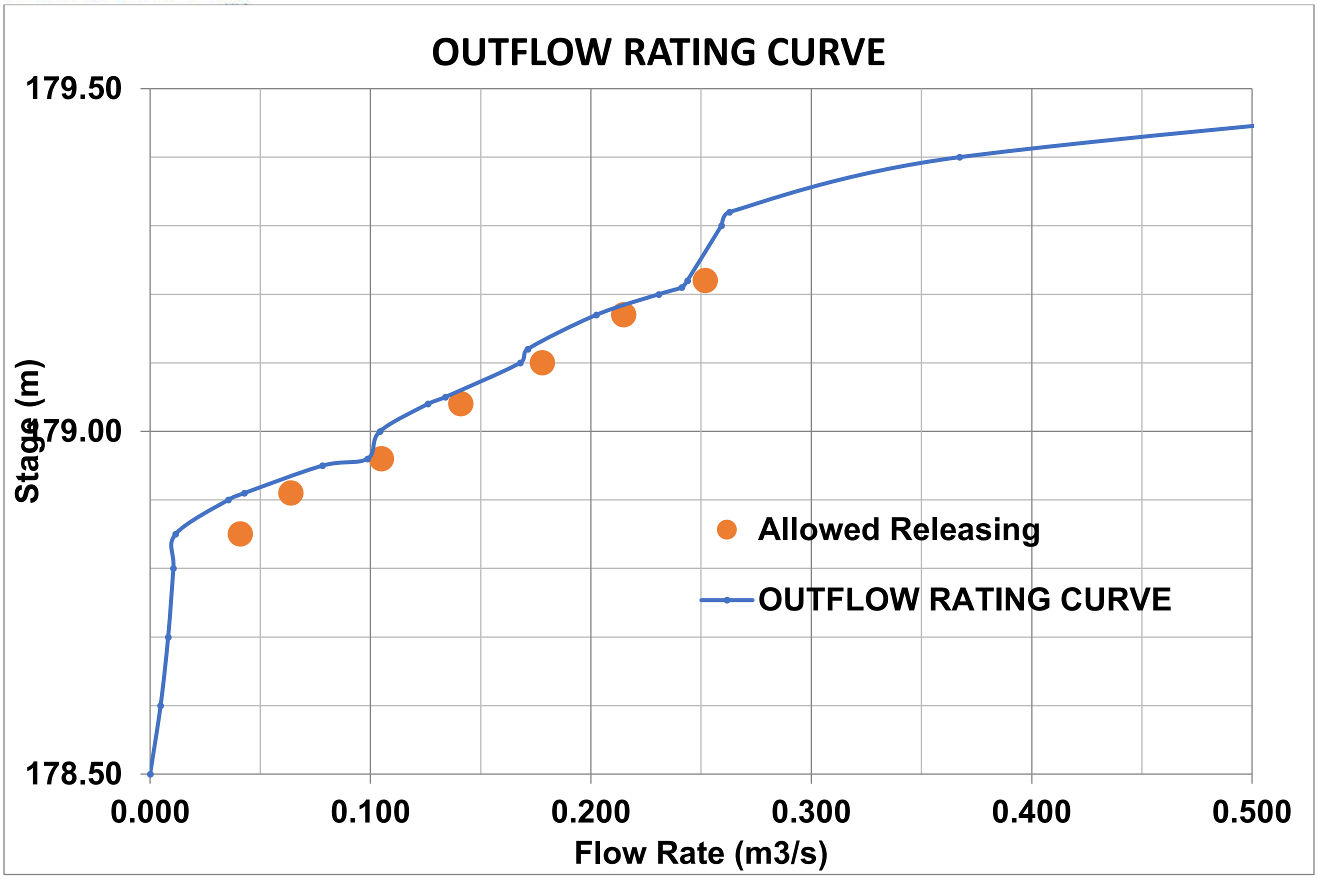


Figure 10 Pond Rating Curve

Table 11 Comparison of Controlled Flow from Pond with Allowable Release Rate

Storm Events	Stage (m)	Controlled Flow by Pond- (m^3/s)	Allowed releasing rate (m^3/s)
25-mm	178.85	0.011	0.041
2-Year	178.91	0.043	0.064
5-Year	178.96	0.095	0.105
10-Year	179.04	0.129	0.141
25-Year	179.10	0.168	0.178
50-Year	179.17	0.201	0.215
100-Year	179.22	0.239	0.252

Table 12 Pond Rating curve

Description	Elevation (m)	Stage (m)	Total flow (m3/s)	Total Volume (m3)	Released Rate (m3/s)	Required Storage (m3)	Allowed Released Rate (m3/s)
TOP OF POND	**180.00**	**1.500**	6.8698	**7090**			
	179.90	1.400	4.9151	6503			
	179.80	1.300	3.3516	5933			
Q100=1.751m3/s(Spillway)	**179.70**	**1.200**	2.1493	5380			
	179.60	1.100	1.2757	4843	**Qregi=1.055m3/s**	**V=4664m3**	
	179.55	1.050	0.9510	4580			
	179.50	1.000	0.6948	4322			
	179.40	0.900	0.3672	3817			
Invert Spillway	**179.32**	**0.820**	**0.2629**	**3423**			
	179.30	0.800	0.2592	3326			
100-Year	**179.22**	**0.720**	**0.2438**	**2944**	**Q100=0.239m3/s**	**V=2913m3**	**0.252m3/s**
	179.20	0.700	0.2307	2851	**Q50=0.201 m3/s**	**V=2666m3**	**0.215m3/s**
	179.12	0.620	0.1713	2483			
	179.10	0.600	0.1679	2393	**Q25=0.168 m3/s**	**V=2402m3**	**0.178m3/s**
	179.05	0.550	0.1339	2170	**Q10=0.129 m3/s**	**V=2137m3**	**0.141m3/s**
	179.00	0.500	0.1043	1951	**Q5=0.095 m3/s**	**V=1851m3**	**0.105m3/s**
	178.96	0.460	0.0884	1779	**Q2=0.043 m3/s**	**V=1563m3**	**0.064m3/s**
	178.90	0.400	0.0354	1525			
Extended Detention	**178.85**	**0.350**	**0.0115**	**1318**	**Q25mm=0.011m3/s**	**V=1262m3**	**0.041m3/s**
	178.80	0.300	0.0105	1114			
	178.70	0.200	0.0082	722			
	178.60	0.100	0.0048	349			
Permanent Pool	**178.50**	**0.000**	**0.0000**	**0**			

4.0 – SWM Pond Operation & Maintenance

4.1 – Inspection & Monitoring

The proposed stormwater management facility will require regular monitoring, particularly during the initial years of operation.

4.2 - Hydraulic Operation of Facilities

Periodic monitoring, typically 3 times per year, should be undertaken. If water level is higher than normal, then the outlet structure or downstream receiver should be checked. Too low of a water level may be attributed to leakage or due to evaporation during extended dry weather periods.

4.3 – Clogging

Large storms transfer significant amounts of debris. A number of stormwater components should be inspected periodically [typically 3 times per year]. The monitoring inspection should include:

- Inlets / Outlets.
- Downstream and Upstream Channels.
- Low Flow Orifices.
- Trash Racks.
- Weirs; and
- Spillways / Emergency Spillway.

4.4 – Pipe Repairs

Physical inspections should be carried out 3 times per year. The inspection may involve identification of obvious failures such as damaged inlet or outlet structures. Periodic inspections should also consider impacts associated with vandalism, corrosion, fatigue, and U/V deterioration. If the pipes or risers are damaged, then the functions of the facility will be impacted. The consequences may be particularly severe if pipes through embankments are jeopardized, as this may lead to piping of water along the pipe or riser, which can lead to bank failure.

4.5 – Grass Cutting

Grass cutting around stormwater facilities is generally limited to areas adjacent to walkways or maintenance access.

Grass cutting is one maintenance activity which is solely undertaken to enhance the perceived aesthetics of the facility. The frequency of grass cutting depends on surrounding land uses, and local municipal by-laws. Grass cutting should be done as infrequently as possible, recognizing the aesthetic concerns of nearby residents.

Generally, it is recommended that grass-cutting be limited or eliminated around SWM facilities since allowing grass to grow tends to enhance water quality.

4.6 – Vegetation Management

All vegetation communities should be monitored to confirm their health and identify any factors that may act as stressors and contribute to the eventual decline of specific species or the vegetation community as a whole. At a minimum, vegetation communities should be assessed at least twice a year to identify signs of dieback, infestation, or disease.

Monitoring of the vegetation community should occur between June 1, and September 30, in any given year, as it's during this period the majority of species will have emerged and leafed out. Monitoring inspections should be completed on a routine basis in order to gauge the health of the plant community and ensure that the desired functional performance of the vegetation community in the context of the overall SWM facility is achieved over the long-term.

In the event of a drought, spring/summer monitoring inspections should be conducted at least twice between June and September to evaluate the effect on the vegetation community. If there are signs of stress due to lack of water, then a watering program should be incorporated into the maintenance of the vegetation. Watering should occur in the early morning or late evening to ensure the best results.

4.7 – Dredging and Sediment Removal Management

Sources of solid and semisolid materials that are retained in a pond or wetland include:

- Soil loss from lawns and open areas,
- Erosion from upstream conveyance swales,
- Construction sediments,
- Natural leaf litter and down branches,
- Grit from roofing shingles, and
- Atmospheric deposition wash off.

Table 13 Annual Sediment Loadings

Catchment Imperviousness	Annual Loading (m^3/ha)
71%	2.87

Sediment accumulation for 10 years for the catchment area of **7.97 ha** and annual loading of 2.87 m^3/ha is 183 m^3 for 80% efficiency. Provided decant area of 314m^2 for the Pond can be accommodate remove sediment for 10 Years. To ensure long-term effectiveness, the sediment that accumulates in SWM Facilities should be periodically removed.

Two common monitoring inspections:

- Visual inspection
- Undertaking bathymetric surveys

Visual inspections should be undertaken as part of other general monitoring activities [typically 3 times per year] and would generally be limited to defining whether sediment plumes are visible within the facility. Bathymetric surveys require the use of total station equipment, sediment probes and a small craft. Typically, the bathymetric survey is undertaken every 5 to 10 years.

4.8 – Access

Access for inspection and maintenance purposes is generally required at the following locations:

- Inlet and Outlet structures,
- Embankments,
- Riser structures,
- Perimeter of the pond, and
- Pond bottom (during dredging).

Access is generally required to address items mentioned above including inspection of the permanent pool, pipes, and vegetation or vandalism issues.

4.9 – Inspection Program and Checklist

The facility will require regular monitoring, particularly during the initial years of operation. An inspection check list is shown in Table 23. This check list should be completed during each inspection and copies kept, together with a record of any repairs carried out.

The recommended frequency of inspection of the SWM facility is as follows:

- After every significant rainfall (>25 mm)
- Minimum of 3 visits per year (Spring, Summer, Fall)

Table 14 Checklist for Inspection of SWM Facility

Stormwater Management – Proposed Pond

Inspection/ Monitoring Check List
Date: ________________________
Inspected By: _________________
In case of pond failure, the following personnel should be notified immediately:

Item		**Maintenance Required(Y/N)**	**Comments**
1.	Check pond level. Is pond level higher than normal more than 24hours after rainfall? If so, check outlet for blockage		
2.	Is pond level lower than normal? If so, check inlet headwall for blockage and check that the maintenance outlet valve is closed.		
3.	Is the vegetation around the pond healthy?		
4.	Is there an oily sheen on the water near the outlet? Is the water frothy? Is it an unusual color? Any of these may indicate a spill and the need for cleanup.		
5.	Check the sediment depth in the forebay and pond.		
6.	Are there any signs of erosion of the spillway or outlet channel?		
7.	Check the outlet control manhole including the valves. When checking the maintenance valve carefully crack open the valve only.		
8.	Check the inlet structure for debris build up etc.		
9.	Does the access road need maintenance?		

5.0 – Hydraulic Gradeline

5.1 – Introduction

The development of the hydraulic grade line (HGL) is a last step in the overall design of a storm drainage system.

The hydraulic grade line is used to aid the designer in determining the acceptability of a proposed or evaluation of an existing storm drainage system by establishing the elevation to which water will rise when the system is operating under design conditions. Details on this system performance analysis are presented in this section.

5.2 – Definitions

- Energy Grade Line (EGL) – represents the total available energy in the system (Potential energy plus kinetic energy).
- Hydraulic Grade Line – the HGL, a measure of flow energy, is a line coinciding with the level of flowing water at any point along an open channel.
- In closed conduits flowing under pressure, the HGL is the level to which water would rise in a vertical tube at any point along the pipe. If the HGL is above the inside top (Crown) of the pipe, pressure flow conditions exist.

5.3 – Design Guidelines

Storm drainage systems operating under surcharged conditions (pressure flow) shall evaluate the HGL and minor losses such as manhole losses, bends in pipe, expansion, and contraction losses, etc.

Pressure flow design must assure the HGL is below the rim elevation of any drainage structure which may be affected. The EGL must also be at or below the rim elevation of any drainage structure which may be affected.

Evaluation of the HGL for a storm drainage system begins at the system outfall with the tail water elevation.

Most storm drains discharge into natural receiving streams or drainages which do not submerge the outlet of the system. However periodically there are locations where a storm system must discharge into a stream or river (Pond) where major floods in those streams will submerge the storm drain and back water into the system.

When a peak flow of the design magnitude occurs in the system simultaneously with a high flow in the receiving stream (Pond), the system becomes surcharged and operates under pressure flow conditions.

Under the surcharged condition, backwater from the receiving stream may raise the elevation of the water surface (HGL) in the system high enough that water could overflow the manholes and inlets at low points in the road grade.

When the HGL rises above ground level, storm water can be found shooting out of catch basins or popping manhole covers, which can lead to damage and inconvenience to pedestrian and vehicular traffic.

5.4 – Input Data for Modeling Storm Sewer System

Figures 11 below shows the plan view of proposed storm sewer for development at Kalar Rd used for modeling of HGL for minor flow. Figure 12 shows minor flow used in modeling. Table 15 and 16 show the geometry data used in modeling.

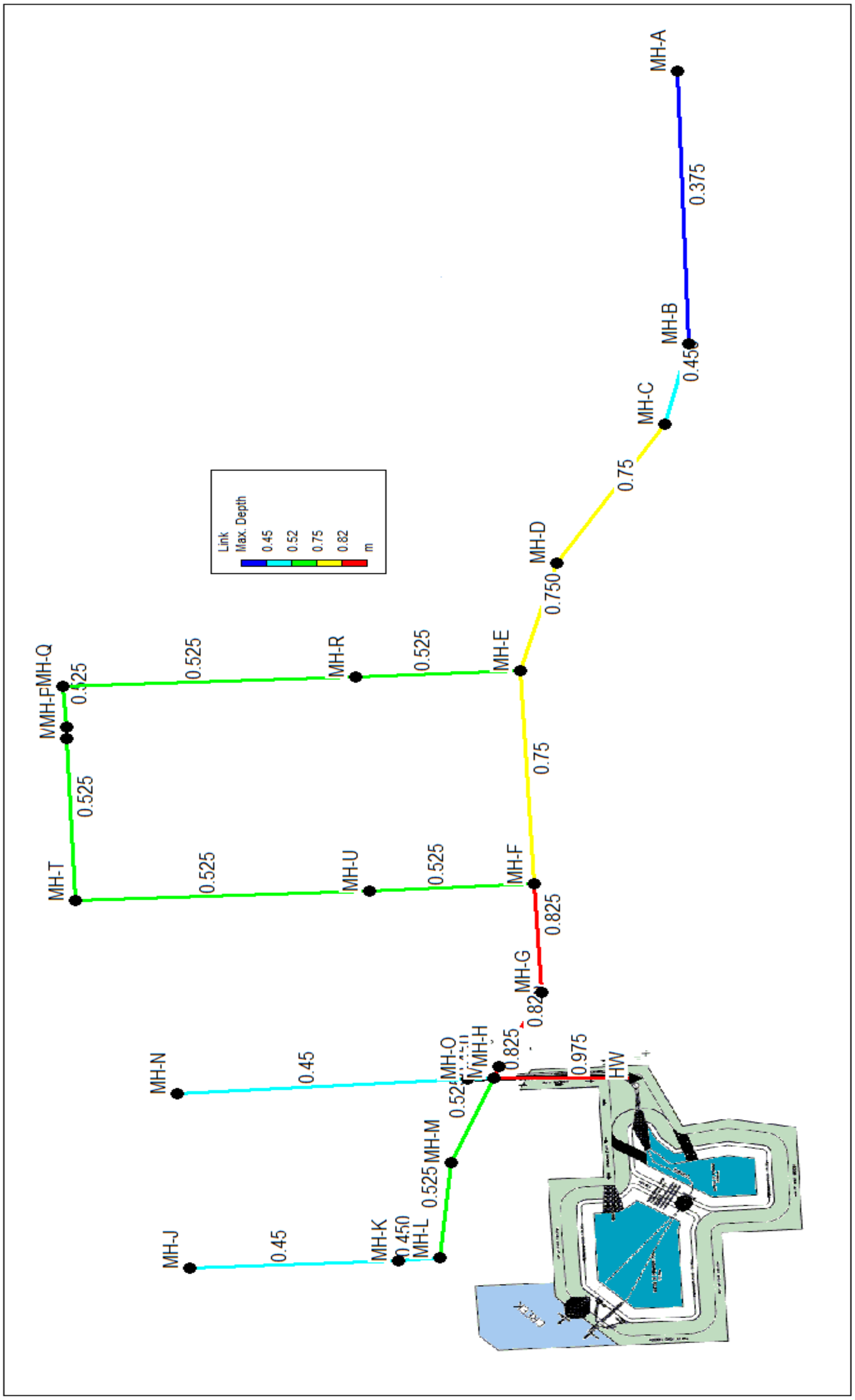


Figure 11 Schematic of Proposed Storm Sewer including diameter of sewer

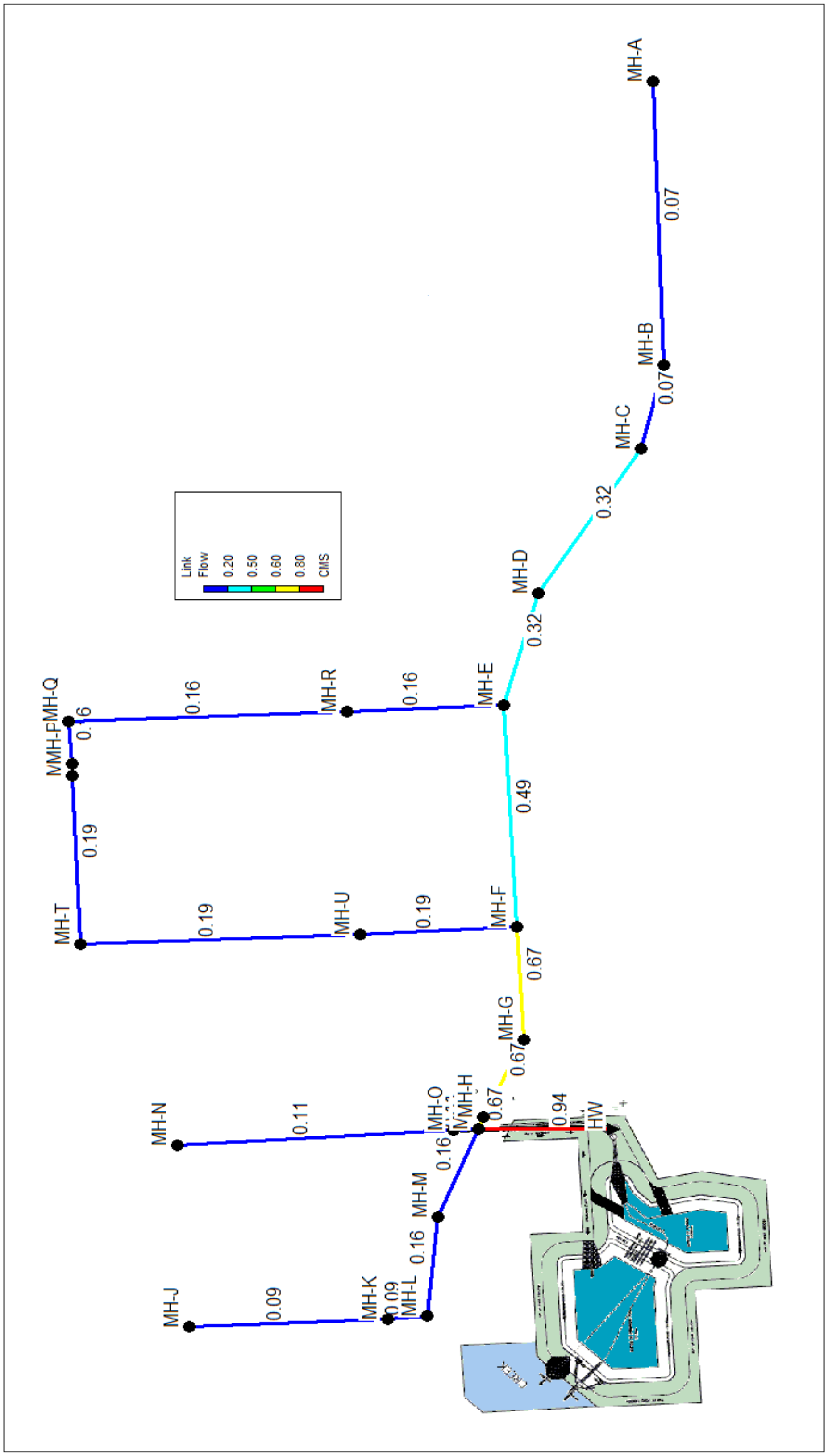


Figure 12 Minor flow for modeling HGL

Table 15 Input geometry data for model HGL

INLET MH	OUTLET MH	L(m)	D(m)	D(mm)
MH-E	MH-F	77.70	0.750	750
MH-A	MH-B	98.60	0.375	375
MH-B	MH-C	31.50	0.450	450
MH-I	HW	40.20	0.975	975
MH-J	MH-K	64.10	0.450	450
MH-K	MH-L	8.60	0.450	450
MH-L	MH-M	38.70	0.525	525
MH-M	MH-I	38.30	0.525	525
MH-N	MH-O	86.30	0.450	450
MH-O	MH-I	6.50	0.450	450
MH-F	MH-G	39.40	0.825	825
MH-G	MH-H	32.00	0.825	825
MH-H	MH-I	2.50	0.825	825
MH-S	MH-T	59.30	0.525	525
MH-T	MH-U	88.90	0.525	525
MH-U	MH-F	49.50	0.525	525
MH-P	MH-Q	14.10	0.525	525
MH-Q	MH-R	89.00	0.525	525
MH-R	MH-E	49.50	0.525	525
MH-C	MH-D	80.30	0.750	750
MH-D	MH-E	39.00	0.750	750

Table 16 Major Flow used for Modeling HGL

MH	FLOW (m3/s)	FLOW (l/s)
MH-J	0.0856	85.60
MH-L	0.0730	73.00
MH-N	0.1071	107.10
MH-S	0.1900	190.00
MH-P	0.1646	164.60
MH-C	0.2531	253.10
MH-A	0.0698	69.80
	TOTAL FLOW:	**943.20**

5.5 – HGL Results

A summary of the HGL output results for the minor flows have been included below as Table 17.

Table 17 HGL using Minor flow

MH	HGL(m)
MH-J	179.91
MH-K	179.79
MH-L	179.74
MH-O	179.60
MH-I	179.44
MH-H	179.61
MH-N	179.83
MH-G	179.73
MH-F	179.85
MH-E	180.09
MH-S	180.44
MH-T	180.24
MH-U	180.01
MH-P	180.53
MH-Q	180.42
MH-R	180.21
MH-D	180.14
MH-C	180.23
MH-B	180.41
MH-A	180.80
MH-M	179.64
HW	179.03

As can be seen in Table 18 elevation of HGL is below Obvert as per city design manual.

Table 18 HGL showing Invert and Obvert data

MH	HGL(m)	INVERT (m)	Obvert (m)	HGL Below Obvert (m)
MH-J	**179.91**	179.67	180.12	**0.21**
MH-K	**179.79**	179.48	179.93	**0.14**
MH-L	**179.74**	179.38	179.91	**0.16**
MH-O	**179.60**	179.27	179.72	**0.12**
MH-I	**179.44**	178.72	179.70	**0.25**
MH-H	**179.61**	178.88	179.71	**0.09**
MH-N	**179.83**	179.53	179.98	**0.15**
MH-G	**179.73**	178.98	179.81	**0.07**
MH-F	**179.85**	179.10	179.93	**0.07**
MH-E	**180.09**	179.41	180.16	**0.07**
MH-S	**180.44**	180.00	180.53	**0.09**
MH-T	**180.24**	179.82	180.35	**0.10**
MH-U	**180.01**	179.55	180.08	**0.07**
MH-P	**180.53**	180.10	180.63	**0.09**
MH-Q	**180.42**	180.06	180.59	**0.17**
MH-R	**180.21**	179.79	180.32	**0.10**
MH-D	**180.14**	179.53	180.28	**0.14**
MH-C	**180.23**	179.77	180.52	**0.29**
MH-B	**180.41**	180.16	180.61	**0.20**
MH-A	**180.80**	180.53	180.91	**0.10**
MH-M	**179.64**	179.27	179.80	**0.16**
HW	**179.03**	178.5	179.48	**0.44**

Figures 13 to 17 shows HGL for all profiles of storm sewers.

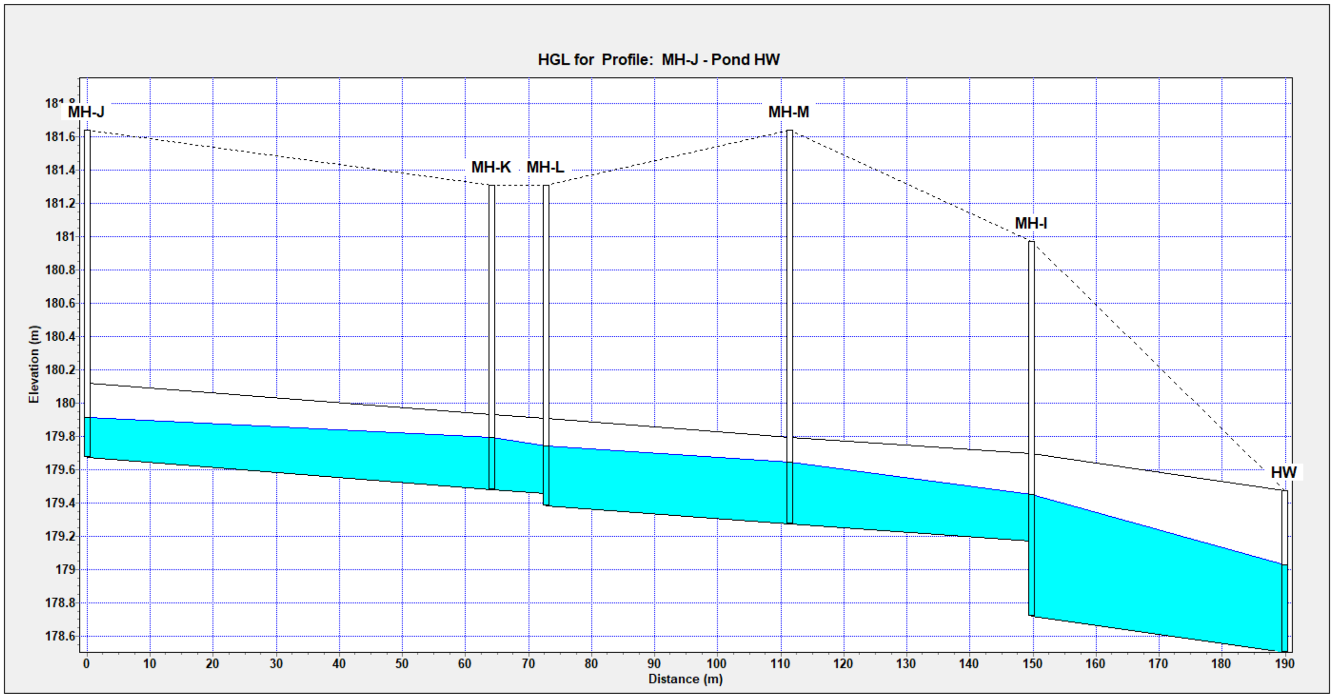


Figure 13 - HGL for Profile MH-J to Head Wall

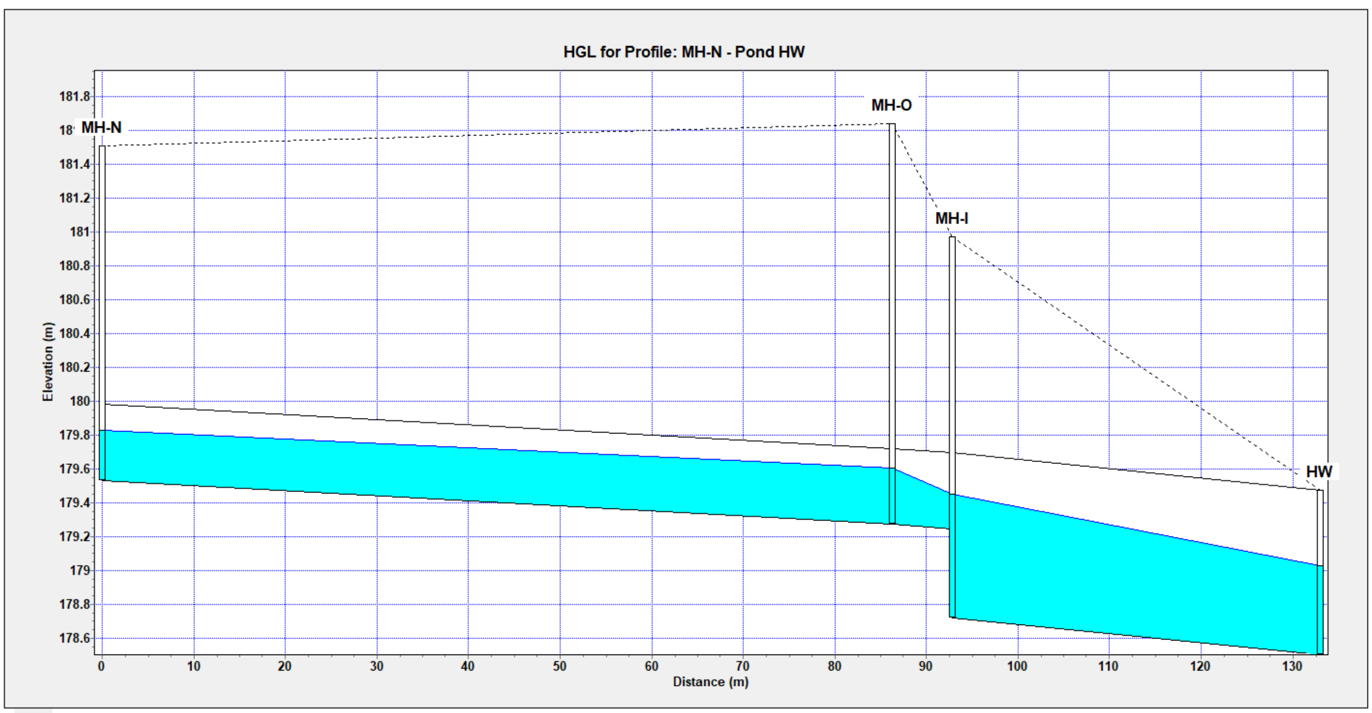


Figure 14 - HGL for Profile MH-N to Head Wall

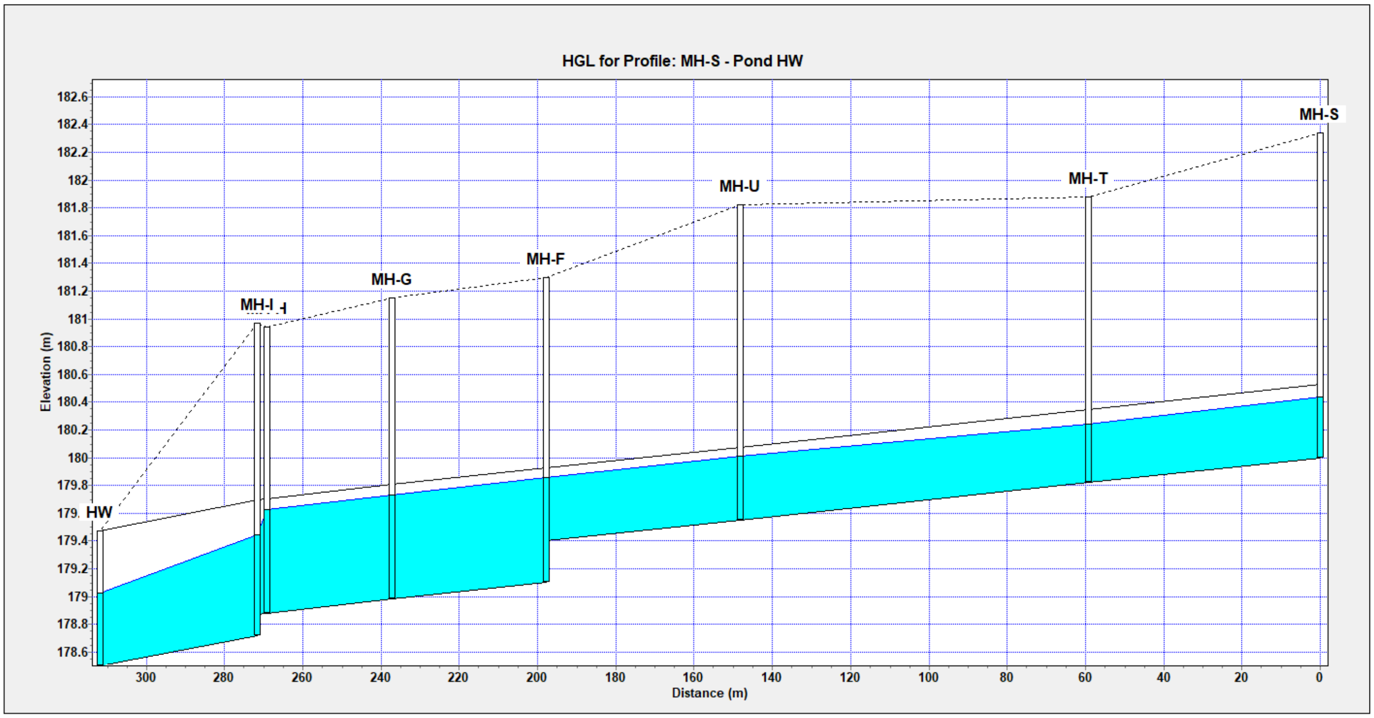


Figure 15 - HGL for Profile MH-S to Head Wall

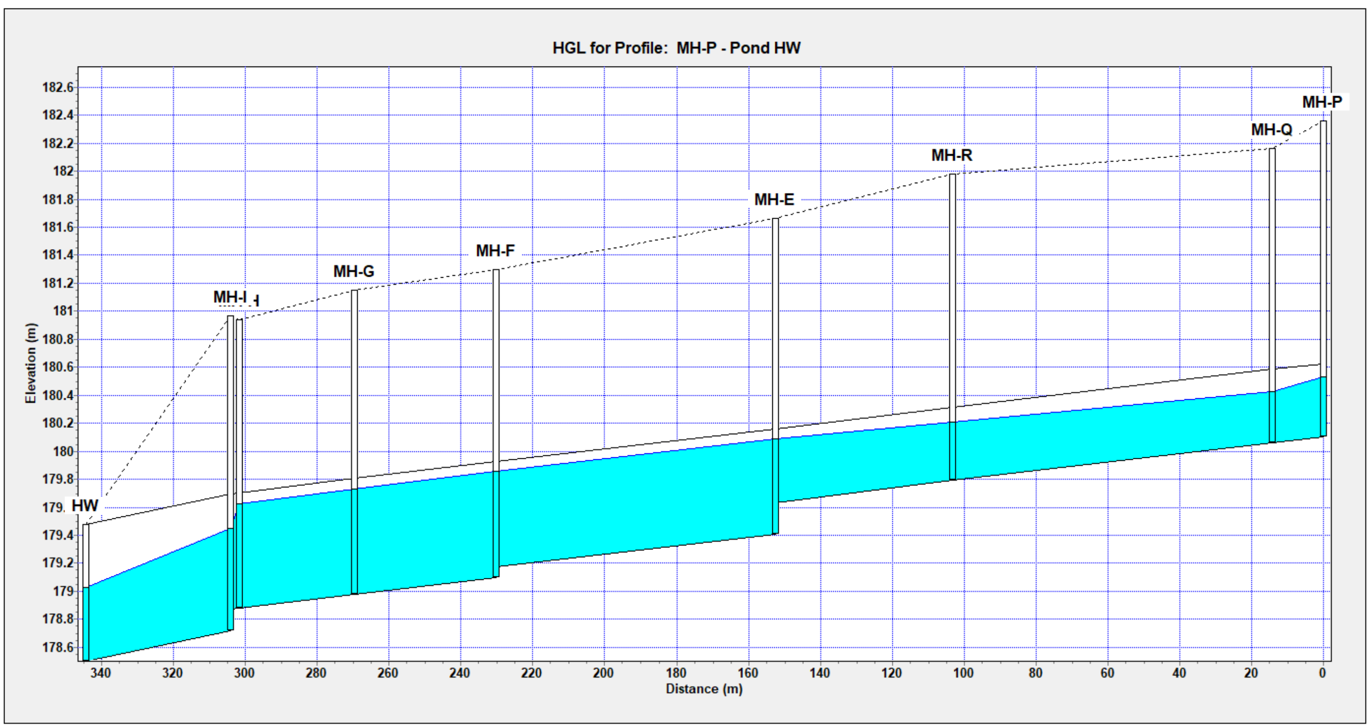


Figure 16 - HGL for Profile MH-P to Head Wall

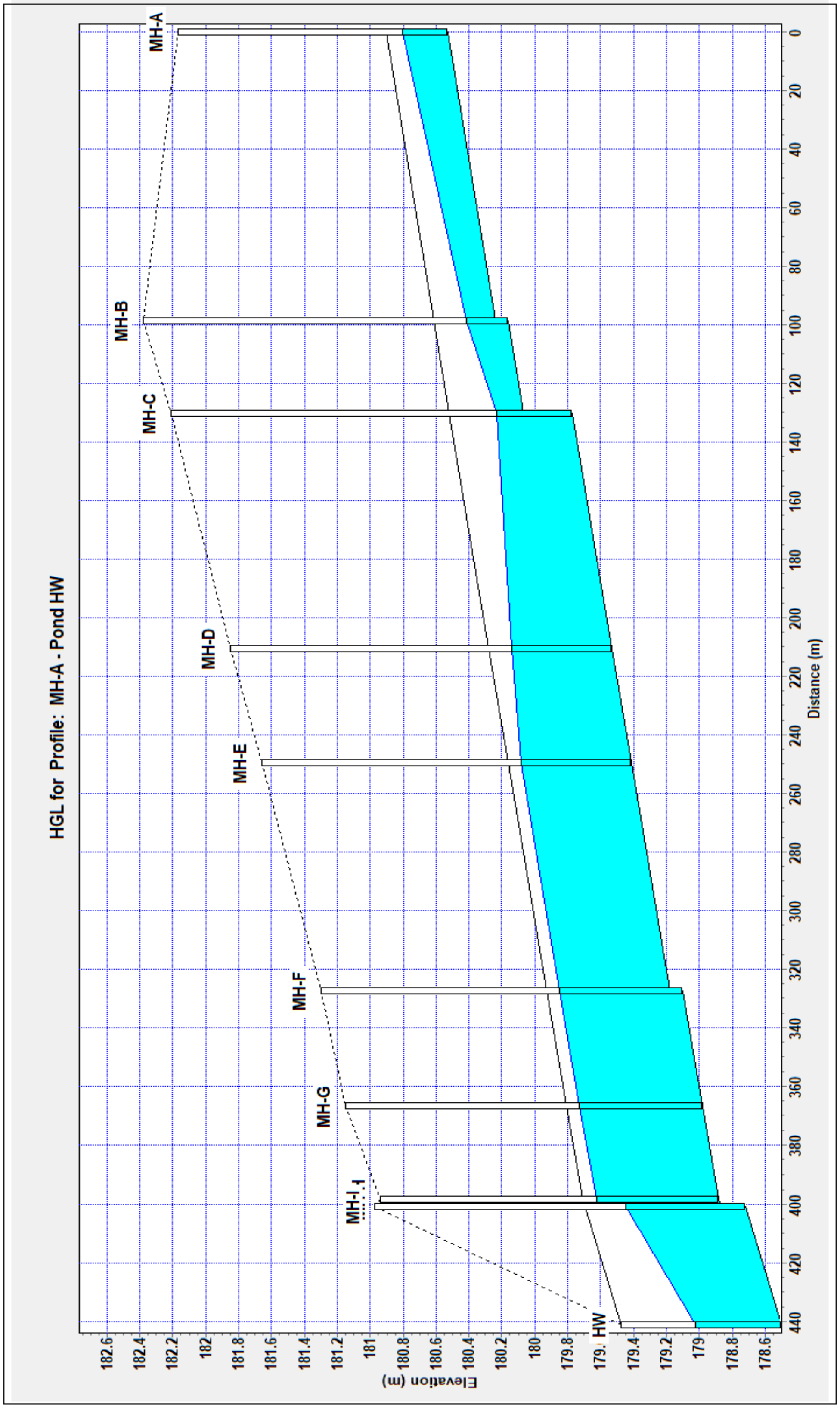


Figure 17 - HGL for Profile MH-A to Head Wall

6.0 - CONCLUSIONS

This report demonstrates that the proposed Stormwater Management Pond including all inlet and outlet facilities has been designed in accordance with all municipal and provincial guidelines and can safely accommodate all storm events from the proposed development drainage area and convey them to Warren Creek with a releasing rate less than predevelopment conditions.

As can be seen from Table 18, the HGL for the proposed storm sewer system using the minor system flows is below the obvert of all sewers as per the City Design Criteria.

Prepared by

Kevin Hollingworth, P. Eng

Dr. Jadran Jelin M Sc., PhD. P.Eng.

APPENDIX A

SIZING OF THE SWM FACILITIES AT KALAR RD

1. Permanent Pool

Total drainage area contributing to SWM Pond

A = 7.97 ha

Imp = 71%

Storage volume for Imp = 71% can be seen in Table 3.2 Water Quality Storage

Requirements based on Receiving Waters (MOE 2003) for Wet Pond.

Storage Volume: 226 m^3/ha

Quality storage: (226 – 40) = 186 m^3/ha

Required storage for Permanent Pool:

Volume: A x 186 m3/ha = 7.97 x 186 = 1482 m^3 Provided 2493m^3

2. Sediment Forebay

Settling Length

r = 2.00 l : w ration recommended by MOE

Qp = 0.010 m3/s Outflow from Pond (Extended detention)

Vs = 0.003 m/s Settling velocity of 150 um particles

Ls = SQRT (r x Qp /Vs)

Ls = 8.16m

Dispersion Length

Ld = 8 x Q / (d x Vf)

Q10 = 1.185 m^3/s 10-Year Storm

Q5 = 0.983 m^3/s 5-Year Storm

d = 1.00m Depth of permanent pool

Vf = 0.50 m/s (Velocity of water jet at exit)

Ld = 8 x Q / (d x Vf) = 18.96m Provided length 41.80m

<u>Width of Sediment Forebay</u>

W = Ld / r

W = 18.96 / 2 = 9.48 m Provided width = 14.30m

<u>Width of Deep Zone</u>

Wd = Ld / 8

Wd = 18.96 / 8 = 2.37 m Provided = 5.10 m

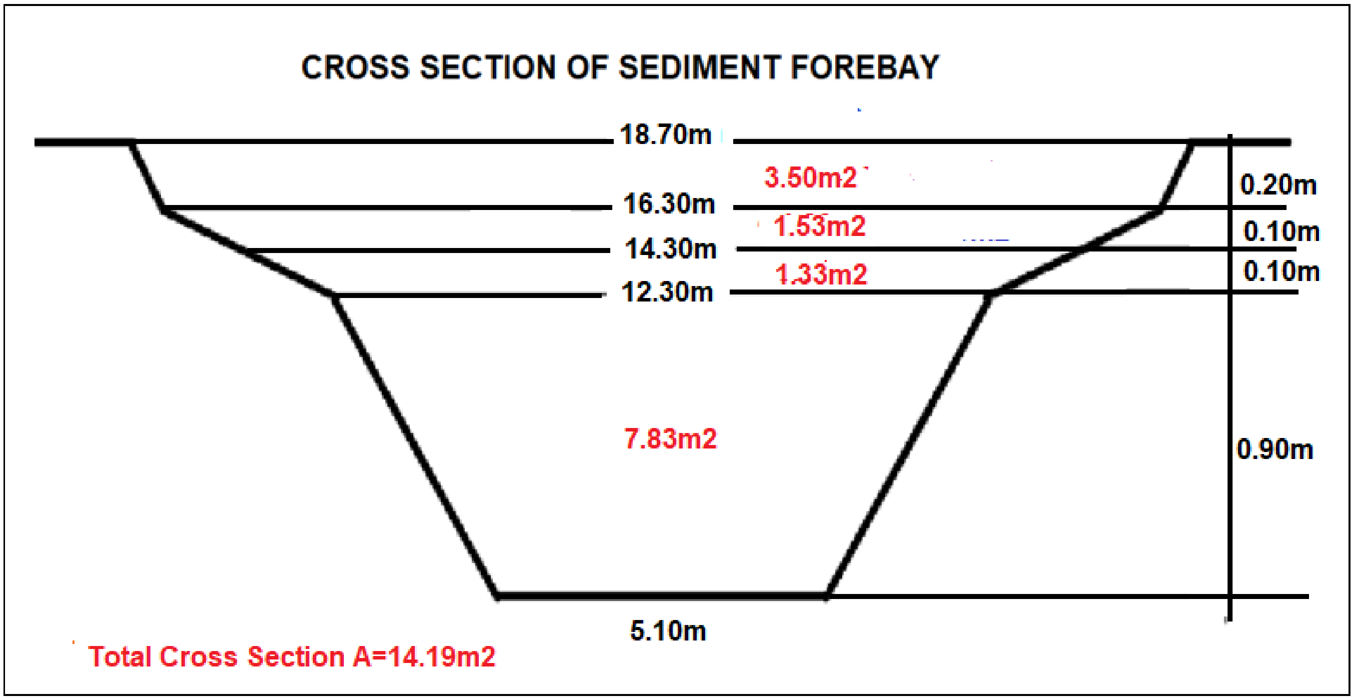
<u>Check for average Velocity</u>

Q

Q = 1.185 m^3/s Check Velocity for 10-Year storm

A = 14.19m2

V = 1.185 / 14.19 = 0.083 m/s Max permissible velocity V = 0.150 m/s



Sediment Removal Frequency

Catchment Area = 7.97 ha

Catchment Imperviousness = 71%

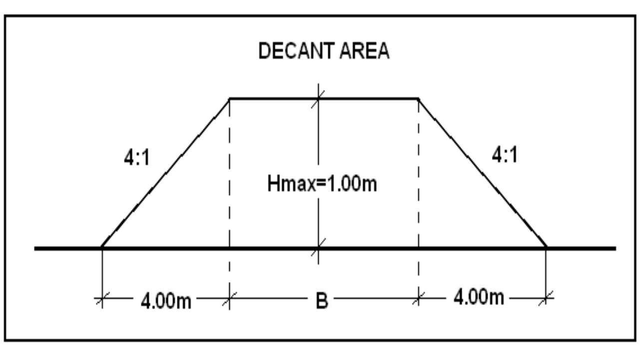
Annual Loading = 2.87 m3/ha

Removal Efficiency = 80%

Annual Sediment Accumulation = 7.97 x 2.87 x 80% = 18.30 m^3/yr.

Sediment Accumulation in 10 years = 182.99 m^3

Required Decant Area



Bottom area = 313.78 m^2

Top area = 84.21 m^2

Total volume = 198.99 m^3

<u>Convey flow from the sediment forebay to the permanent pool</u>

<u>Flow in Pipe</u>

H = 0.30m

D = 450 mm diameter of pipe

A = 0.159

C = 0.63

Q = 0.243 m^3/s

Three pipes can convey 3 x 0.243 = 0.729 m^3/s

<u>Flow over Weir</u>

10-Year Storm Q = 1.185 m^3/s

Over weir = 1.185 – 0.729 = 0.456 m^3/s

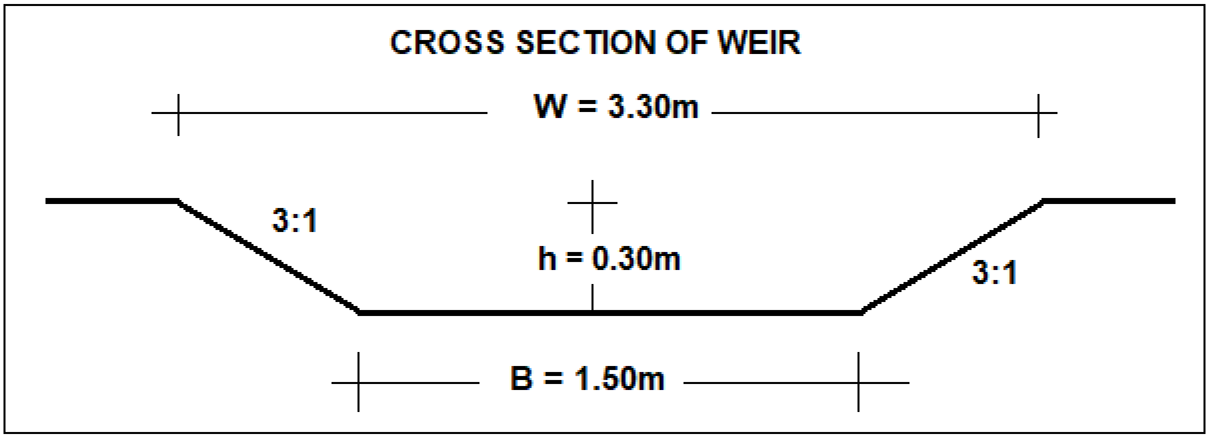
H = 0.30 m

B = 1.50 m

Cvnotch[3] = 1.27

Q weir = 0.553 m^3/s

Total Flow = 0.729 + 0.553 = 1.281 m3/s   > required = 1.185 m^3/s



Overland flow

Input data:

B = 4.00 m

H = 0.30 m

Slope of easement = 1.00%

100 – year storm Q100 = 1.751 m^3/s

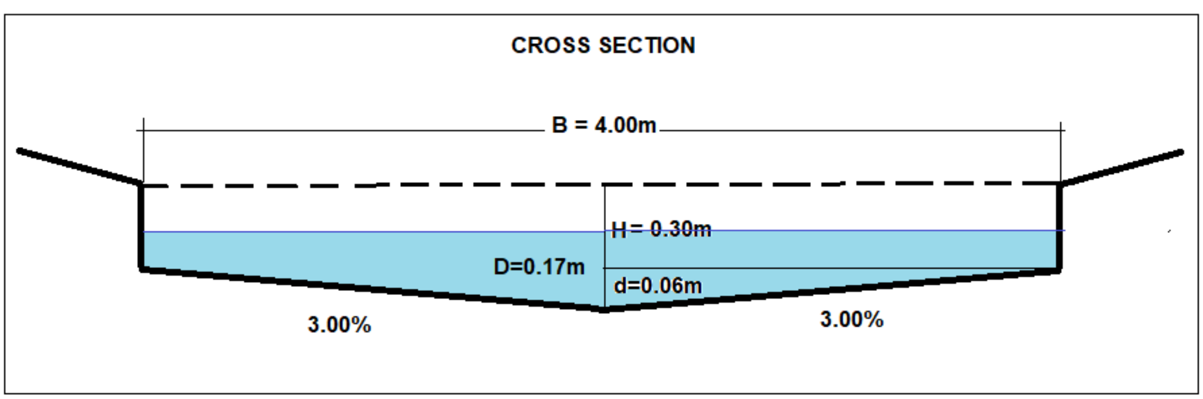
5 – year storm Q5 = 0.983 m^3/s

Overland flow = 1.751 – 0.983 = 0.768m^3/s

Result

D = 0.17 m

Flow rate Q = 0.770 m^3/s > required 0.768 m^3/s



KALAR RD P18002

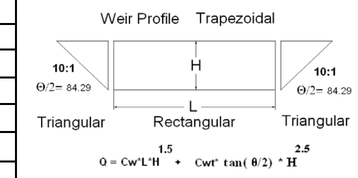
POND

A=7.97ha Imp=71%

Q oriff = **Coriff*A*(2*g*H)^0.50**
Qweir = **Cw*L*H^1.50**

		DICB - 2 ,3 and 4	Quantity Orifice-4	Quantity Orifice-3	Quantity Orifice-2	Quality Orifice-1	SPILLWAY-WEIR	
	D of Pipe		0.450	0.450	0.450	0.300		
	D of Orifice		0.180	0.180	0.260	0.098	L =	2.00
	Area of Orifice		0.025	0.025	0.053	0.008	Cweir =	1.48
	L=	1.20					Cvnotch=	1.27
	Cweir=	1.71					tan 84.29=	10.00
	Coriffice=		0.630	0.630	0.630	0.63		

0 0



Design by : **Dr J.Jelin P.Eng**

Date : **06-Oct-22**

METROPOLITAN CONSULTING INC.

Description	Elevation (m)	Stage (m)	Spillway (m3/s)	DICB - 4(m3/s)	Orifice-4 (m3/s)	DICB - 3 (m3/s)	Orifice-3 (m3/s)	DICB - 2 (m3/s)	Orifice-2 (m3/s)	Quality Orifice-1 (m3/s)	Total flow (m3/s)	Area (m2)	Inc. Volume (m3)	Total Volume (m3)	Released Rate (m3/s)	Required Storage (m3)	Allowed Released Rate (m3/s)	Total Volume (m3)	Inc Time (sec)	Inc Time (hr)	Total Time (hr)
TOP OF POND	**180.00**	**1.50**	**6.502**	**1.694**	**0.084**	**2.052**	**0.084**	**2.531**	**0.173**	**0.0254**	6.8698	**5953**	**587**	**7090**				**9583**	**85**	**0.02**	**54**
	179.90	1.40	**4.561**	**1.414**	**0.081**	1.752	0.081	2.208	0.167	0.0245	4.9151	5785	570	6503				8996	116	0.03	54
	179.80	1.30	**3.012**	**1.151**	**0.078**	1.468	0.078	1.900	0.160	0.0235	3.3516	5618	554	5933				8426	165	0.05	54
Q100=1.751m3/s	**179.70**	**1.20**	**1.824**	**0.906**	**0.075**	**1.202**	**0.075**	**1.608**	**0.153**	**0.0226**	2.1493	**5453**	537	5380				7873	250	0.07	54
	179.60	1.10	**0.965**	**0.682**	**0.071**	0.954	0.071	1.333	0.146	0.0216	1.2757	5289	262	4843	**Qregi=1.055m3/s**	**V=4664m3**		7335	206	0.06	54
	179.55	1.05	**0.649**	**0.579**	**0.070**	0.837	0.070	1.202	0.142	0.0211	0.9510	5208	258	4580				7073	272	0.08	54
	179.50	1.00	**0.401**	**0.481**	**0.068**	0.725	0.068	1.075	0.138	0.0205	0.6948	5127	505	4322				6815	727	0.20	54
	179.40	0.90	**0.090**	**0.304**	**0.064**	0.519	0.064	0.837	0.130	0.0194	0.3672	4969	394	3817				6310	1073	0.30	54
Invert Spillway	179.32	0.82	**0.000**	**0.184**	**0.061**	0.371	0.061	0.661	0.123	0.0185	0.2629	4885	97	3423				5916	369	0.10	53
	179.30	0.80		**0.157**	**0.060**	0.337	0.060	0.619	0.121	0.0182	0.2592	4808	381	3326				5819	1472	0.41	53
100-Year	179.22	0.72		**0.065**	**0.056**	0.212	0.056	0.462	0.114	0.0172	0.2438	4729	94	2944	**Q100=0.239m3/s**	**V=2913m3**	**0.252m3/s**	5437	385	0.11	53
	179.20	**0.70**		**0.046**	**0.055**	**0.184**	**0.055**	**0.425**	**0.112**	**0.0170**	0.2307	**4650**	368	2851	**Q50=0.201 m3/s**	**V=2666m3**	**0.215m3/s**	5344	1593	0.44	53
	179.12	0.62		**0.000**	**0.052**	0.085	0.052	0.288	0.104	0.0159	0.1713	4541	90	2483				4976	527	0.15	52
	179.10	0.60				0.065	0.051	0.256	0.102	0.0156	0.1679	4494	223	2393	**Q25=0.168 m3/s**	**V=2402m3**	**0.178m3/s**	4886	1327	0.37	52
	179.05	0.55				0.023	0.048	0.184	0.096	0.0149	0.1339	4417	219	2170	**Q10=0.129 m3/s**	**V=2137m3**	**0.141m3/s**	4663	1635	0.45	52
	179.00	0.50				**0.000**	**0.000**	0.119	0.090	0.0141	0.1043	4340	172	1951	**Q5=0.095 m3/s**	**V=1851m3**	**0.105m3/s**	4444	1650	0.46	51
	178.96	0.46						0.075	0.085	0.0135	0.0884	4263	254	1779	**Q2=0.043 m3/s**	**V=1563m3**	**0.064m3/s**	4272	2869	0.80	51
	178.90	0.40						0.023	0.077	0.0125	0.0354	4187	207	1525				4018	5858	1.63	50
Extended Detention	**178.85**	**0.35**						**0.000**	**0.000**	**0.0115**	**0.0115**	**4111**	**204**	**1318**	**Q25mm=0.011m3/s**	**V=1262m3**	**0.041m3/s**	**3811**	**17637**	**4.90**	**48**
	178.80	0.30								0.0105	0.0105	4036	392	1114				3607	37167	10.32	43
	178.70	0.20								0.0082	0.0082	3803	373	722				3215	45639	12.68	33
	178.60	0.10								0.0048	0.0048	3663	349	349				2842	73419	20.39	20
Permanent Pool	**178.50**	**0.00**								**0.0000**	**0.0000**	**3317**	**0**	**0**				**2493**	**0**	**0.00**	**0**
Permanent Pool	**178.50**	**0.00**										**2095**	**201**	**1663**	**1206**	**113**	**830**	**2493**	**Required = 1481m³**		
	178.40	-0.10										1919	188	1462	1053	102	717	2179			
	178.30	-0.20										1851	182	1274	993	96	615	1888			
	178.20	-0.30										1784	175	1092	934	91	518	1610			
	178.10	-0.40										1717	168	917	877	85	428	1345			
	178.00	-0.50										1652	162	748	820	79	343	1091			
	177.90	-0.60										1589	156	586	765	74	264	850			
	177.80	-0.70										1526	150	431	711	68	190	621			
	177.70	-0.80										1465	143	281	658	63	121	403			
	177.60	-0.90										1405	138	138	607	58	58	196			
Bottm of Pond	**177.50**	**-1.00**										**1347**	**0**	**0**	**558**	**0**	**0**	**0**			