
**PROPOSED RESIDENTIAL TOWNHOUSE DEVELOPMENT
829 ESSA ROAD – CITY OF BARRIE
STORM WATER MANAGEMENT & FUNCTIONAL
SERVICING REPORT**



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RESIDENTIAL TOWNHOUSE DEVELOPMENT 829 ESSA ROAD – CITY OF BARRIE STORM WATER MANAGEMENT & FUNCTIONAL SERVICING REPORT

1.0 INTRODUCTION

1.1 General

The proposed development is located at 829 Essa Road, east of the Essa Road and Athabaska Road intersection. The property is legally described as Part of Lot 3, Concession 11 in the former Township of Innisfil, now in the City of Barrie.

The subject site is approximately 0.12 hectares (0.3 acres) in area and is currently occupied by a single-family residential home. Access to the site is provided directly from Essa Road and Athabaska Road by two (2) connecting asphalt driveways. The existing site is bounded by Essa Road to the north, Athabaska Road to the east, and residential development to the south and west. The location of the subject site is illustrated on Figure 1.

The property owner is proposing to construct a block of three-storey townhouse units, totaling 8 units. Access to the development will be provided from Athabaska Road along the east side of the site.

1.2 Purpose and Scope

Pinestone Engineering Ltd (PEL) has been retained by the property owner to prepare a Storm Water Management and Functional Servicing Report in support of a Site Plan Approval. The report describes the water and sanitary servicing, and the storm water quality and quantity control strategy for the site.

2.0 REFERENCE REPORTS

The following reports and studies have been used for reference in the preparation of this Storm Water Management Report:

- i) *City of Barrie Storm Drainage and Storm Water Management Policies and Design Guidelines, prepared by the City of Barrie Engineering Department, October 2020*
- ii) *NVCA Stormwater Technical Guide, Effective Date: December 13, 2013*
- iii) *City of Barrie Development Manual, prepared by the City of Barrie Engineering Department, December 2017*
- iv) *Ministry of the Environment Storm Water Management Planning and Design Manual, March 2003.*
- v) *Low Impact Development Manual prepared by Credit Valley Conservation and Toronto and Region Conservation, 2010*
- vi) *City of Barrie Infiltration Low Impact Development Screening Process, 2017*
- vii) *City of Barrie Water Supply Master Plan Update; Final Report prepared by WSP, 2019*
- viii) *City of Barrie Wastewater Collection Master Plan Update; Final Master Plan Report prepared by Cole Engineering Group Ltd., 2019.*



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SITE LOCATION PLAN

DATE: JAN 2019	SCALE: N.T.S.	PROJECT No. 17-11409	FIGURE No. FIGURE 1
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3.0 EXISTING CONDITIONS

3.1 General

The site is currently occupied by a single-family residential dwelling. Access to the property is provided directly from Essa Road and Athabaska Road. The site is located within the City's urban servicing area and municipal services are available for connection along Athabaska Road. Existing services to the residential home not being utilized for the proposed development will be removed.

3.2 Topography

Based on a review of the topographic survey provided by Joe TOPO SURVEYS AND CADD INC., the property is relatively flat and generally slopes from a high point towards the north and eastern boundaries of the subject site. Elevations across the site range between 304.83m ASL at the southwest corner of the site to 302.90m ASL at the northeast corner.

3.3 Site Geology

A geotechnical investigation was completed by Central Earth Engineering in December 2018. Field work for this investigation consisted of 3 boreholes advanced to approximately 6m deep. Based on our review of their report, boreholes reveal topsoil was encountered over fill and native layers of sandy silt glacial till, overlaying clay and silt. The boreholes showed no indication of ground water levels; therefore, the groundwater table is expected to be deeper than approximately 6m below existing grade.

Based on the existing sandy silt condition encountered throughout the site, infiltration of storm water into underlying native soils is recommended.

A copy of the geotechnical investigation, prepared by Central Earth Engineering, is included in Appendix A.

3.4 Drainage Conditions

Drainage from the site is conveyed in the form of overland sheet flow towards the existing catch basin located on Athabaska Road. A small portion of external drainage enters the site from the existing residential development to the southwest as overland sheet flow.

A 450mm diameter culvert exists along the frontage of the subject site on Essa Road and conveys drainage east into a roadside ditch (to be confirmed by the contractor). Drainage is conveyed easterly along Essa Road before ultimately discharging into Bear Creek. The site is not located within a Nottawasaga Valley Conservation Authority (NVCA) regulated area based on available mapping on their website.

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4.0 PROPOSED DEVELOPMENT

The proposed development on the property consists of eight (8) standard townhouses with individual driveways. Access to the development will be provided by eight (8) asphalt driveways from Athabaska Road along the east side of the site.

Catchment 201, consisting of drainage from a portion of the buildings, driveways, and landscape areas will be conveyed to the existing catch basins on Athabaska Road. Storm water will discharge offsite via a sewer connection to the existing roadside ditch network on Essa Road. Catchment 202, consisting of a portion of the building rooftops, and landscape areas will flow into rear-yard rain gardens. Excess storm water will be directed to the rear-yard catch basin and discharge offsite into the 450mm storm culvert that ultimately outlets to the existing roadside ditch network on Essa Road.

Water and sanitary servicing for the development will be provided from existing services along Athabaska Road. Existing services from Essa Road for the single-family dwelling will be utilized by the proposed development or decommissioned, per City of Barrie standards.

5.0 HYDROLOGY

5.1 Design Criteria

Based on a review of the City of Barrie's Storm Water Management (SWM) Guidelines, the following design criteria, in accordance with the current MECP SWM Planning and Design Manual (MECP, 2003) were established for the proposed development:

Quantity Control:

- Peak flow attenuation for the 2-year through 100-year storm events to pre-development rates based on the Rational Method using the City of Barrie's WPCC IDF parameters.

Quality Control:

- The City of Barrie in accordance with the NVCA Stormwater Technical Guide regarding water quality and phosphorus loading to the Nottawasaga River, requires that all new storm water management facilities provide as a minimum the "enhanced" level of protection in accordance with the MECP Storm Water Management Planning and Design Manual (MECP, 2003).
- Preparation of phosphorus and water balance calculations to meet City of Barrie requirements and the NVCA Stormwater Technical Guide.
- Preparation of detailed erosion, sediment control and construction mitigation plan to be implemented as part of the construction program.

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5.2 Design Storms

We have selected the following design storms as part of our evaluation:

- 2-year design storm
- 5-year design storm
- 10-year design storm
- 25-year design storm
- 50-year design storm
- 100-year design storm

Rainfall intensity - duration frequency (IDF) values for the Barrie Area were entered into an equation that expresses the time relationship intensity for specific frequency, in the form of:

$$i = \frac{a}{(t+b)^c}$$

where: i = intensity, mm/hr.
 t = Time of concentration, minutes
 a,b,c = constants developed to fit IDF curve

The design storm parameters are outlined in Table 1 below. A copy of the IDF values taken from the City's SWM guidelines is also included in Appendix B.

**Table 1
Design Storm Parameters - Chicago Rainfall Distribution**

Rainfall Event	Parameter		
	A	B	C
2 Yr	678.085	4.699	0.7810
5 Yr	853.608	4.699	0.7660
10 Yr	975.865	4.699	0.7600
25 Yr	1146.275	4.922	0.7570
50 Yr	1236.152	4.699	0.7510
100 Yr	1426.408	5.273	0.7590

5.3 Drainage Catchments

One (1) pre-development and two (2) post-development catchments have been delineated for the site in order to estimate the corresponding peak runoff rates for the site. The pre-development catchment area represents the existing condition of the proposed development area, which consists of single-family residential development. The post development catchments represent the proposed grading concept for the development area. The pre-development and post-development catchment parameters are listed in Table 2.

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**Table 2
Sub-catchment Parameters**

Catchments	Area (m²)	Slope (%)	Composite Runoff Coefficient “C”
Pre-Development			
101	1742	3.0	0.39
Post-Development			
201	755	2.0	0.69
202	987	2.0	0.44

Pre-development and post-development catchment areas are illustrated on Figure 2, and Figure 3, below. Composition runoff values were increased for the 10 through 100-year storm events per Table 3.2 in the City’s SWM guidelines.

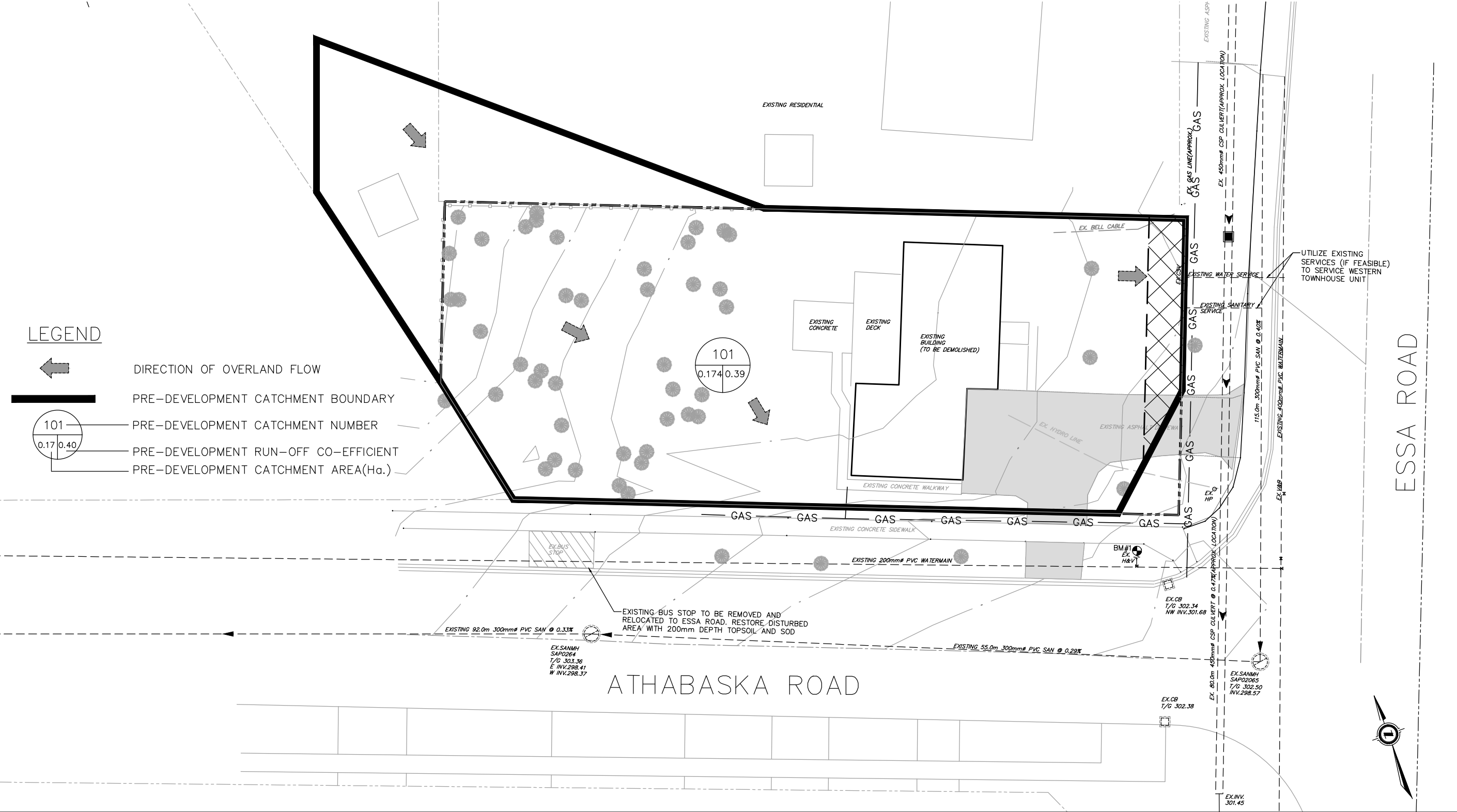
5.4 Rational Method Results

The results from the 2 through 100-year peak storm events are listed in Table 3.

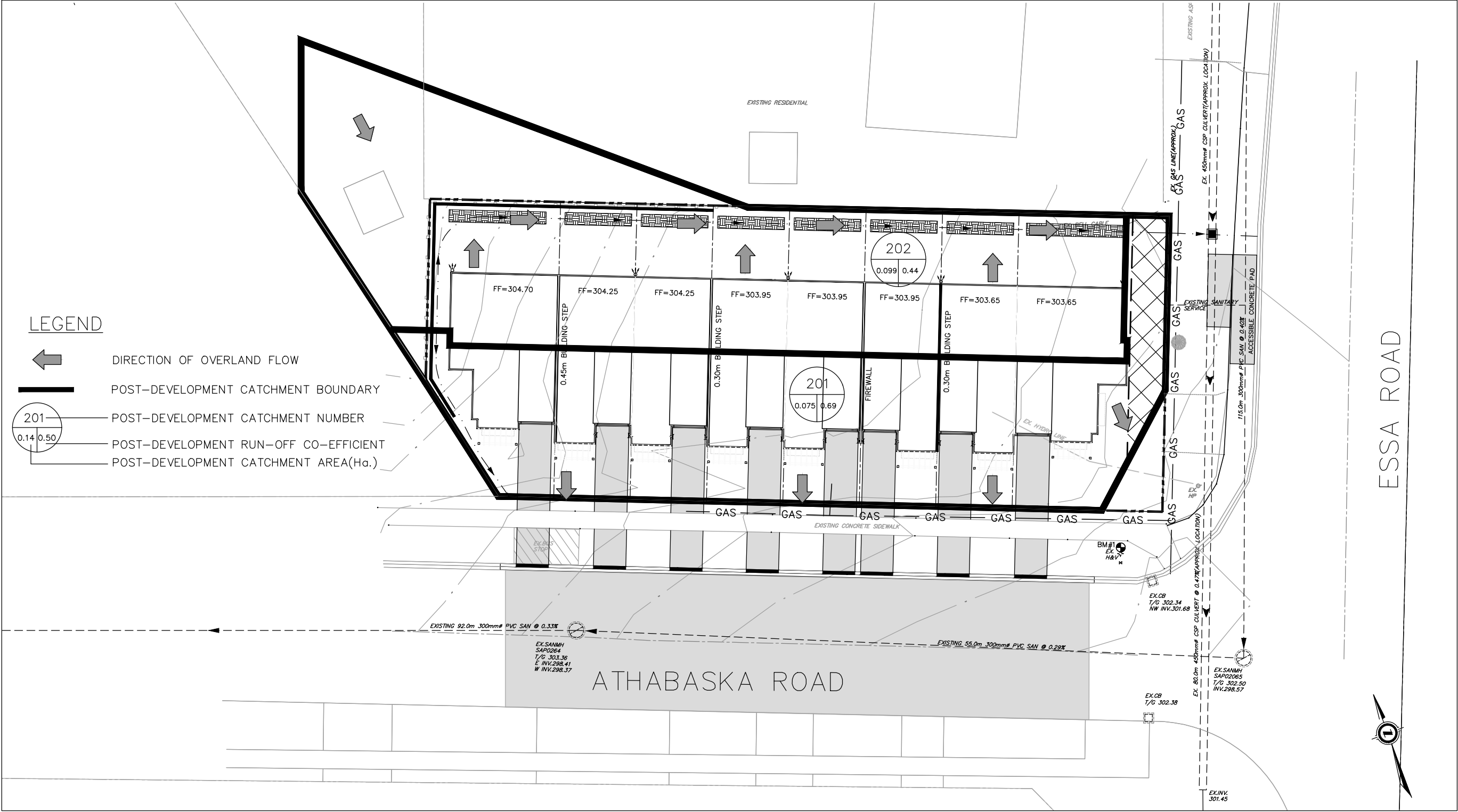
**Table 3
Rational Method – Peak Flows**

	2Yr	5Yr	10Yr	25Yr	50Yr	100Yr
Pre-Development (m³/sec)						
Catchment 101	0.016	0.021	0.024	0.031	0.037	0.042
Total Pre-Development	0.016	0.021	0.024	0.031	0.037	0.042
Post Development (m³/sec)						
Catchment 201	0.012	0.016	0.018	0.024	0.028	0.033
Catchment 202	0.010	0.013	0.015	0.020	0.024	0.027
Total Post Development	0.022	0.029	0.034	0.043	0.052	0.060

Based on the calculated results using the Rational Method, it is expected that post development flows directed to Athabaska Road will marginally increase due to the proposed infill townhouse construction. Rational Method design calculations are included in Appendix B.



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	PRE-DEVELOPMENT CATCHMENT PLAN		SCALE: 1:300	DATE: JAN 2019
			FIGURE 2	



6.0 STORM WATER MANAGEMENT PLAN

6.1 Quantity Control

As noted in the comparison of the pre-development and post development flows, a marginal increase in runoff will occur due to the proposed infill townhouse construction. To satisfy the selected design criteria, peak flow attenuation of post development flows to pre-development levels for all storm events up to and including the 100-year storm event will be provided by directing partial roof drainage to the proposed rain gardens. These have been designed as per the Low Impact Development SWM Manual and MECP SWM Planning and Design Manual standards and criteria.

Drainage from catchment 201 which includes the perimeter landscape areas, a portion of the building rooftop, and asphalt driveways will continue to drain offsite uncontrolled like pre-development conditions.

Peak flow attenuation for catchment 202, which includes runoff generated from a portion of the building rooftop, and landscape areas, will be achieved using rain gardens in the rear yards of each unit. This limits the uncontrolled flow of storm water onto Essa road by allowing runoff to infiltrate into the underlying soils. Each rain garden will compose of 1.0m depth engineered soil for filtration, and a 0.7m depth stone reservoir for stormwater detention. Additional storage for runoff is provided on the surface of the rain gardens as they are designed with a 0.2m depth depression. Rain garden design calculations are included in Appendix B.

The rain gardens are designed to capture and store 100% of the post-development flow generated from a portion of the building rooftops. This is shown in the Modified Rational Method calculations by eliminating the impervious building rooftop area for catchment 202. Major flows from storm events greater than the 100-year event will overflow the rain garden depressions and discharge via a rear-yard swale to the proposed catch basin that connects to the Essa Road ditch network.

Table 4 summarizes the effectiveness of the proposed storm water management attenuation feature based on the hydrologic model results.

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**Table 4
Model Results – Rational Method**

	2Yr	5Yr	10Yr	25Yr	50Yr	100Yr
Pre-Development (m³/sec)						
Catchment 101	0.016	0.021	0.024	0.031	0.037	0.042
Total Pre-Development	0.016	0.021	0.024	0.031	0.037	0.042
Post Development (m³/sec)						
Catchment 201	0.012	0.016	0.018	0.024	0.028	0.033
Catchment 202	0.010	0.013	0.015	0.020	0.024	0.027
Total Post Development	0.022	0.029	0.034	0.043	0.052	0.060
Post Development with SWM (m³/sec)						
Catchment 201 (uncontrolled)	0.012	0.016	0.018	0.024	0.028	0.033
Catchment 202 (controlled)	0.002	0.003	0.004	0.005	0.006	0.006
Total Post Development with SWM	0.014	0.019	0.022	0.028	0.034	0.039

6.2 Quality Control (Total Suspended Solids & Phosphorus Control)

The primary objective of the storm water management plan for this development is to maintain acceptable water quality within the receiving storm sewer and ultimate outfall, Nottawasaga River, by maintaining existing site drainage patterns and flowrates. In order to provide water quality enhancement to an “enhanced” level of protection (80% TSS removal) for this development, we have incorporated a “treatment train” approach consisting of the following elements:

- Construction of a grass drainage swale along the west property line to promote cleansing and infiltration of runoff.
- Installation of rain gardens on each lot for roof drainage.

The potential treatment alternatives have been evaluated with respect to their applicability for this development and implemented in a manner to achieve the best total suspended solids (TSS) removal possible. Table 5 summarizes the proposed measures that in conglomeration will provide an overall TSS removal of greater than or equal to 80% which meets or exceeds the criteria outlined.

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**Table 5
Proposed Approach for Water Quality Treatment**

Catchment	Surface	Method	Effective TSS	Area (m ²)	% Area of Site	Overall TSS Removal
201	Driveways	None	0%	135	7.7	0.00%
	Rooftop	Inherent	100%	370	21.2	21.24%
	Vegetated / natural areas	Filtration / Evapotranspiration	80%	250	14.4	11.48%
202	Rooftop	Inherent	100%	350	20.1	20.09%
	Vegetated / natural areas	Filtration / Evapotranspiration	80%	637	36.6	29.25%
Total	Total			1742	100.0	82.07%

The NVCA P-Tool was utilized to determine pre and post development phosphorus loadings for the proposed development area. In the pre-development condition, low intensity residential development was assumed for most of the development area, except for partial forest area to the south of the site, for phosphorus loading. For the post-development condition, low intensity residential development was assumed. The calculated phosphorus loadings are illustrated on Table 6.

**Table 6
Pre & Post Phosphorus Loading**

	P Load (kg/yr)
Pre-Development	0.02
Post Development with no BMP's	0.02
Post Development with BMP's	0.01

The proposed rain gardens provide 100% phosphorus removal benefit for rooftop drainage and perimeter landscape. A 50% post-development reduction can be achieved on the site when implementing best management practices; considering the good soil conditions for the purposes of infiltration for further phosphorus removal.

Pre and post development phosphorus budget calculations are included in Appendix B.

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6.3 Water Budget

As per City policy 4.1.3 of the Storm Water Policy and Design Guidelines, sites less than 5 ha shall minimize any anticipated changes in the water balance between pre-development and post-development conditions and shall provide a minimum infiltration equivalent to the first 5mm of rainfall. Based on a total development area of 1742m², the first 5mm of rainfall to be retained on the site for infiltration equates to 8.71m³. Initial abstraction values provided in the City's SWM guidelines are shown in Table 7.

**Table 7
Initial Abstraction Values**

Cover	Initial Abstraction / Depression Storage (mm)
Woods	10
Meadows	8
Cultivated	7
Lawns	5
Impervious areas	2

Adapted from UNESCO, Manual on Drainage in Urbanized Areas, 1987

Using the values provided in Table 7 above, approximately 6.14m³ of rainfall will be retained on the site through initial abstraction in the post-development condition.

Based on a review of the geotechnical report prepared for the site, and the infiltration techniques outlined in the MECP SWM Manual (2003) and the LID Manual (2010), infiltration measures are suitable for the site given the soils condition and development-built form. It is anticipated that the proposed rain gardens will provide 23.52m³ of storage for infiltration. The rain gardens will retain a storm water volume of 1.05m³ during the first 5mm of rainfall onsite, yielding an overall post-development abstraction value of 7.19m³. To provide additional retention of runoff onsite, roof leader downspouts will discharge directly to the proposed landscape areas at the front and rear of the townhomes.

7.0 EROSION AND SEDIMENT CONTROL

7.1 Mitigation Measures

Sedimentation and erosion control measures are required during construction and until such a time that site development has been completed and the driveway area has been paved.

The use of various siltation control measures will be implemented to protect the adjacent properties and receiving waterbodies from migrating sediments. These works include but may not be limited to:

- Installation of siltation fencing along perimeter of the development area.
- Filter cloth / silt sack placement over drains.

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- Installation of vehicle tracking mud mat at the entrance to the site.

Prior to carrying out site grading the siltation barriers and mud mats shall be in place. Any onsite storm sewer works will not be permitted to outlet to the municipal sewers until the site has been stabilized.

Other temporary installations of silt fence or other appropriate measures may be required during grading to minimize silt migration from the site. The measures will need to be removed, replaced and relocated as required during the construction period until the site works have been completed and vegetation established. During construction, all stockpiled material will be placed up-gradient of the siltation controls with additional siltation fencing installation around the stock-piles.

7.2 Monitoring and Maintenance During Construction

It is the responsibility of the contractor and owner to maintain the siltation control devices until suitable grass cover has been established upon completion of construction.

A regular review of the facilities by the contractor should be carried out during the construction period to ensure that the facilities are being properly maintained, and if necessary, replaced. Inspection reports are to be completed monthly and distributed to the owner, contractor, and City.

If site works are to continue through the winter and spring the engineer should be contacted by the owner to review the measures in place with the contractor on a regular basis to ensure that the facilities are adequate and in good working order.

All reasonable methods to control erosion and sedimentation are to be taken during construction. The contractor should inspect the siltation devices immediately after each rainfall. Damaged devices should be repaired immediately, and additional devices installed if necessary. Silt should be removed from the fencing and related siltation devices when deposits are noticeable.

Should the erosion control measures fail, and sediment migrate beyond the limits of the control works, additional sedimentation facilities may need to be installed in the area of the migration and down gradient to contain the sediment.

8.0 WATER SERVICE

8.1 Existing Water Servicing

A 200mm diameter watermain currently exists along the property frontage on Athabaska Road and Essa Road. The site is currently serviced by a 25mm diameter water service connection to the residential home. If the existing service will not be utilized to service the new development, it will be decommissioned by turning off the main stop and cutting the service at the main.

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For this report, onsite pressures and flows have been confirmed to ensure there is sufficient capacity available for both domestic and firefighting conditions. We have utilized information obtained from the municipal hydrants in the vicinity of the subject site connected to the existing 200mm diameter watermain, including hydrant #39 and #1487. Table 8 illustrates the flow results of the testing conducted by Vipond on October 23rd, 2020.

**Table 8
Results of Hydrant Flow Tests**

Test #	Outlet Inside Dia. (in.)	Number of Outlets	Residual Reading (PSI)	Flow@ Residual (gal/min)
1	n/a	n/a	61 (static)	n/a
2	1.75	1	58	584
3	2.5	1	56	876
4	2.5	2	55	1582

Refer to Appendix B for the flow testing information obtained from Vipond.

8.2 Proposed Water Demands

The subject site is located within water servicing Zone 3S. As per the City of Barrie Water Supply Masterplan (WSP, July 2019) domestic water demand for the development are listed in Table 9 below:

**Table 9
Domestic Water Demand**

Population	Per Capita Flow (L/day)	Peaking Factors (based on MECP and City of Barrie Guidelines)		Flows (L/sec)	
		Peak Hour	Maximum Day	Peak Hour	Maximum Day
19	224	2.25	1.8	0.11	0.09

Fire demands for the proposed development were calculated in accordance with the Fire Underwriters Survey (FUS) as follows:

$$F = 220C(A)^{0.5}$$

Where,

- F = the required fire flow in litres per minute.
- C = coefficient related to the type of construction.
- A = total floor area of building (excluding basements)

**Table 10
Water Requirements for Fire Fighting**

Total Area (m ²)	Coefficient "C"	Required Flow (L/min)	Require Flow (L/sec)
880	1.0	6,526	108.77

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The building on the west side of the firewall was chosen because it is the worst-case scenario for total floor area and building exposure charges. The total area represents 2 floors for a single block of townhouse units (440m² x 2). Based on the guidance provided in the FUS and applying the relevant reductions and charges in flow for construction type (-25%), and exposure distances due to the proximity of the other proposed and existing buildings (+60%), the required fire flow for the development will be increased to 130.53 L/sec. Detailed FUS calculations are provided in Appendix B.

8.3 Proposed Water Servicing

Water servicing for the proposed development will be provided by the existing 200mm dia. watermain located within the Athabaska Road right-of-way. Each single unit will be serviced by a 25mm dia. water service from the existing 200mm dia. watermain.

Based on the provided flow test results, it can be concluded that the max day plus fire flow rate of 130.62 L/sec can be delivered to the proposed buildings from hydrant #39 with a residual pressure of 49.2 psi. Fire flow calculations are included in Appendix B. A servicing layout is provided on the drawings included in Appendix C.

9.0 SANITARY SERVICE

9.1 Existing Sanitary Servicing

A 300mm diameter sanitary sewer currently exists along Essa Road and carries flows west along Athabaska Road via a 300mm diameter sanitary sewer. Sewage is ultimately conveyed to the Holly Pumping Station (PS5) located on County Road 27. According to the information provided in the City of Barrie Wastewater Collection Master Plan (Cole Engineering, July 15th, 2019), the Holly Pumping Station (PS5) has a firm capacity of 200.9 L/sec and is currently operating around 81 L/sec, or 40% capacity.

Based on our review of the existing wastewater collection performance assessments completed by Cole Engineering in the Wastewater Collection Master Plan, under 2019 conditions, no surcharging of downstream sanitary sewers from the subject site were noted. Under wet weather flow conditions for a 25-year storm event, the immediate downstream reach of trunk sewer experienced a peak conduit depth ratio (d/D) of less than 0.5, and maintained a freeboard (distance between the ground surface and peak hydraulic grade line) of greater than 1.8m, indicating that there is no capacity issue and no surcharging conditions are predicted.

9.2 Proposed Sanitary Servicing

Contributing sanitary flows from the proposed development are calculated using the following City of Barrie design criteria as follows:

- A residential average sewage flow of 225 litres/capita/day
- A residential population density of 2.34 persons/unit
- An extraneous flow rate of 0.1 litres/sec/ha

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- A peaking factor based on Harmon's equation

The proposed development concept includes 8 townhouse units with a population of 19. Incorporating extraneous flows, the peak sewage flow generated by the proposed development is calculated to be approximately 0.23 L/sec. Detailed sanitary sewer design calculations are attached in Appendix B.

The immediate downstream reach of 300mm diameter sanitary sewer installed at 0.33% has a conveyance capacity of approximately 55L/sec. The proposed peak flow from the development represents 0.4% of the capacity of the immediate downstream reach of sewer and on this basis, it is expected available capacity exists within the existing infrastructure to support the development with no external sanitary upgrades anticipated.

The proposed development will be serviced by 8 – 100mm diameter gravity service laterals connecting to the existing 300mm diameter main located on Athabaska Road.

A detailed servicing drawing is provided in Appendix C.

10.0 SUMMARY AND CONCLUSIONS

The findings of this report are summarized as follows:

- Attenuation of peak post-development flowrates to below pre-development levels will be provided by utilizing rain gardens located in the rear yards of the townhomes.
- Quality control for the development will be provided by rain gardens to provide a minimum of 80% TSS removal and 50% removal of annual total phosphorus.
- Suitable measures can be implemented during construction to protect the adjacent properties and receiving storm sewers from migrating sediments.
- Water servicing will be provided via 8 – 25mm diameter services from the existing watermain on Athabaska Road. Existing hydrant #39 has the capacity to provide a max day plus fire flowrate of 130.62 L/s to the proposed development with a residual pressure of 49.2 psi.
- Sanitary servicing will be provided via 8 – 100mm diameter gravity services from the existing sanitary main on Athabaska Road.

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It is therefore recommended that:

- 1) This report and drawings be submitted to the City of Barrie for review and approval.
- 2) The construction mitigation measures outlined in this report are utilized as a guideline for construction mitigation measures for this site.

All of which is respectfully submitted,
PINESTONE ENGINEERING LTD.



Dwight Hordyk, EIT
Project Designer



Joe Voisin, P.Eng
Senior Engineer

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APPENDIX A

Geotechnical Investigation – Central Earth Engineering





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GEOTECHNICAL REPORT

Proposed Townhouse Development
829 Essa Road
Barrie, Ontario

Prepared For: 2591451 Ontario Inc.
Type of Document: Final Report
Reference Number: 18-1132A
Prepared By: Alexander Winkelmann, P.Eng
Date Issued: December 21, 2018

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Figure 1 – Site Location Plan

Figure 2A – Borehole Location Plan (Aerial)

Figure 2B – Borehole Location Plan (Proposed)

Figure 3 – Subsurface Cross-Section

Appendices

Appendix A – Borehole Logs

Appendix B – Grain Size Laboratory Data

Appendix C – Typical Details

1 Introduction

Central Earth Engineering Inc. (CEE) was retained by 2591451 Ontario Inc. to complete a geotechnical investigation and report for the property located at 829 Essa Road in Barrie, Ontario. The property is 60 metres long (east to west) and 25 metres wide (north to south). The site is bounded by Essa Road to the east, Athabaska Road to the south, and residential properties to the north and west. A site location plan is provided as Figure 1. The property currently contains a single family detached residential dwelling with a forested backyard.

CEE was provided with the following drawings/documents for review in preparation of this report:

- “*Conceptual Site Plan – 10 Units, 829 Essa Road, City of Barrie*” File 18-803, by Innovative Planning Solutions;
- “*829 Essa Road Street Townhomes*”, Drawings A-2 and A-3, by Patrick Markus Luckie, Architect; and
- A rendering of the initial design of the townhomes.

Based on our review of these drawings/documents, it is proposed to demolish the existing residential building and construct 10 connected townhouse units centred within the site. There is not expected to be any significant grade changes to the site to accommodate the development. Though the initial design concept shows the townhouses as slab-on-grade, it is possible that basements may be included in the final design concept.

Based on our correspondence, and review of the aforementioned information, the purpose of the geotechnical investigation was to assess the soil conditions at three (3) borehole locations spread throughout the site to support the development as described above. One monitoring well was installed for longer term groundwater monitoring. A borehole location plan is provided as Figure 2A on an aerial photograph of the site, and as Figure 2B on the conceptual site plan showing the approximate location of the townhouses.

2 Procedures and Methodology

Prior to the commencement of drilling activities, the locations of underground utilities including natural gas, electrical, telephone, water, etc. were marked out by public and private utility locating companies. The fieldwork for the drilling program was carried out on December 12, 2018. A total of three boreholes (Borehole 1 through 3) were advanced on site by Drilltech Drilling using a track-mounted drill rig. To advance the boreholes, continuous flight solid stem augers and standard soil sampling equipment was utilized. All samples were collected as per ASTM D1586 to assess the strength characteristics of the substrate.

The boreholes were advanced to depths ranging from 5.0 to 6.6 metres below existing grade (Elev. 96.3 to 93.6 metres). The horizontal locations were laid out in the field by CEE at the time of the drilling operations and were spread across the site to have representative information of the possible soil characteristics. Local elevation measurements of the ground surface of the borehole were measured based off the top nut of a fire hydrant located at the southeast corner of the property (with an assumed Elevation of 100.00 metres) using a laser level. GPS measurements measured with a handheld GPS unit and referenced to the NAD 83 geodetic datum.

The field staff examined, and classified characteristics of the soils encountered in the boreholes, including the presence of fill materials, made groundwater observations during and upon completion of the drilling, recorded observations of borehole construction, and processed the recovered samples. Soil sampling was conducted at

regular intervals for the full depth of the borehole. The boreholes were backfilled upon completion. All recovered soil samples were logged in the field, carefully packaged and transported to the laboratory for more detailed examination and classification. In the laboratory, the samples were classified as to their visual and textural characteristics. Stabilized groundwater levels were measured in the three installed monitoring wells on December 18, 2018.

3 Subsurface Conditions

3.1 General Overview

The detailed soil profiles encountered in the boreholes are indicated on the attached borehole logs provided in Appendix A. It should be noted that the conditions indicated on the borehole logs are for specific locations only and can vary between and beyond the borehole locations. The soil boundaries indicated on the borehole logs are inferred from non-continuous sampling. These boundaries are intended to reflect approximate transition zones and should not be interpreted as exact planes of geological change.

In addition, the descriptions provided in the borehole logs are inferred from a variety of factors, including: visual observations of the soil samples retrieved, laboratory testing, measurements prior to and after drilling, and the drilling process itself (speed of drilling, shaking/grinding of the augers, etc.). The passage of time also may result in changes in conditions interpreted to exist at locations where sampling was conducted.

3.2 Stratigraphy

All three boreholes encountered a nominal 150 to 250 mm thick layer of topsoil at surface. Below the nominal topsoil layer in Borehole 3, a layer of earth fill consisting of a silty sand, trace clay, trace gravel and trace rootlets/organics was encountered (possible service trench) and extended to 2.4 metres below existing grade (Elev. 97.3 metres). The earth fill was generally brown to moist. SPT "N" Values measured in the earth fill ranged between 3 to 5 blows per 300 mm of penetration, indicating a very loose to loose relative density.

Underlying the nominal topsoil layer in Boreholes 1 and 2 and underlying the earth fill layer at 2.4 metres below existing grade (Elev. 97.3 metres) in Borehole 3, a silty sand glacial till deposit with trace clay and trace gravel was encountered. The silty sand glacial till extended 2.3 to 3.8 metres below existing grade (Elev. 98.1 to 95.9 metres). One sample was selected to conduct grain size analysis of this deposit, with the results of this testing provided in Appendix B. The silty sand glacial till was generally brown and damp to moist. SPT "N" Values measured in the sandy silt deposit (disregarding the disturbed surface sample in Boreholes 1 and 2) ranged between 12 to greater than 50 blows per 300 mm of penetration, indicating a compact to very dense relative density.

Underlying the sandy silt glacial till in Borehole 1, a sand with some silt deposit was encountered at 3.2 metres below existing grade (Elev. 98.1 metres) and extended to beyond the vertical extent of investigation at 5.0 metres below existing grade (Elev. 96.3 metres). One sample was selected to conduct grain size analysis of this deposit, with the results of this testing provided in Appendix B. The sand deposit was generally brown and moist. SPT "N" Values measured in the sand deposit ranged between 39 to 66 blows per 300 mm of penetration, indicating a dense to very dense relative density.

Underlying the sandy silt glacial till in Boreholes 2 and 3, a clay and silt deposit with trace sand was encountered at 2.3 to 3.8 metres below existing grade (Elev. 97.9 to 95.9 metres) and extended to beyond the vertical extent of investigation at 5.0 to 6.6 metres below existing grade (Elev. 94.7 to 93.6 metres). One sample was selected to conduct grain size analysis of this deposit, with the results of this testing provided in Appendix B. The clay and silt deposit was generally brown to grey and moist. SPT "N" Values measured in the clay and silt deposit ranged between 21 to 89 blows per 300 mm of penetration, indicating a very stiff to hard consistency.

3.3 Ground Water and Infiltration Rate

Unstabilized ground water level measurements and cave measurements were taken upon completion of drilling in each borehole. These measurements provide a rough estimate of the possible excavation and temporary ground water control constructability considerations that may arise. In general, the boreholes showed no instance of unstabilized ground water levels (i.e. the boreholes were dry) and there was no appreciable caving of the sidewalls of the boreholes.

One (1) monitoring well was installed within Borehole 2. Ground water monitoring was conducted on December 18, 2018. At this time, no free water was encountered within the monitoring well. Based on this, the prevailing ground water table is expected to be deeper than approximately 6 metres below existing grade.

A grain size distribution curve of the near surface silty sand glacial till (Borehole 1, Sample 4) was developed by testing the soil sample in accordance with applicable LS standards in reference to ASTM D6913 (sieve analysis) and ASTM D7928 (sedimentation/hydrometer analysis). The result of the laboratory test and graphical representation of this grain size analysis is provided in Appendix B.

Determination of percolation rate are based on the "Ontario Ministry of Municipal Affairs and Housing (OMMAH), *Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions, 1997*". Based on this document, the estimated percolation rate of the sandy silt glacial till deposit encountered at surface is 30 mins/cm (20 mm/hour).

4 Engineering Design Parameters & Analysis

It is proposed to demolish the existing residential building and construct 10 connected townhouse units centred within the site. There is not expected to be any significant grade changes to the site to accommodate the development. Though the initial design concept shows the townhouses as slab-on-grade, it is possible that basements may be included in the final design concept.

For low-rise residential structures, all buildings must be designed in accordance with the requirements of Part 9 of the 2012 Ontario Building Code. Reference should be made to the following sections of the 2012 Ontario Building Code which stipulate the geotechnical design and construction requirements for residential structures:

- 9.3. Materials, Systems and Equipment
- 9.4. Structural Requirements
- 9.12. Excavation
- 9.13. Dampproofing, Waterproofing and Soil Gas Control
- 9.14. Drainage

9.15. Footings and Foundations

9.16. Floors-on-ground

4.1 Foundation Design

4.1.1 Foundations on Native Soil

Native soils suitable for the support of conventional spread and strip footing foundations were located as follows:

- Boreholes 1 and 2: 0.6 metres below existing grade (the surficial 0.6 metres is very loose, likely due to frost action over the years); and
- Borehole 3: 2.4 metres below existing grade (there is a 2.4-metre-thick layer of earth fill that was encountered in this borehole).

Spread or strip footing foundations set at the above stated elevations or lower bearing on the undisturbed native soil deposit can be designed using a geotechnical reaction at SLS of 100 kPa for a maximum of 25 mm of settlement. The factored geotechnical resistance at ULS is 150 kPa.

4.1.2 Foundations on Engineered Fill

Depending on the final depth of the foundations, determination if the earth fill is an isolated area due to previously excavated service trenches, and grade changes to the site, instead of extending foundations down to native soils in the area of Borehole 3 where a relatively thick layer of fill was encountered, the foundations can be set on engineered fill. Engineered fill must be placed under the full-time supervision of a geotechnical engineer as required in the 2012 Ontario Building Code. Prior to placement of the engineered fill all topsoil, organics, deleterious materials and the existing earth fill must be fully removed from within the excavation area, to expose the native silty sand glacial till deposit located at 2.4 metres below existing grade (in the area of Borehole 3). The exposed subgrade must be inspected by a qualified geotechnical staff member of CEE to ensure all unsuitable material was removed prior to commencement of engineered fill placement.

The engineered fill may consist of the excavated on-site soils provided they have a moisture content within 2% of optimum moisture content. Engineered fill must be compacted to a minimum of 98% SPMDD. The engineered fill must extend a minimum of 0.5 metres out from all sides of the foundations and extend at a 1H:1V slope down to the exposed subgrade. The engineered fill must be placed to be 0.3 metres higher than the proposed underside of foundation elevation. A sketch illustrating this dimensioning is provided within Appendix C.

Foundations placed on engineered fill as specified above can be designed using a geotechnical reaction at SLS of 100 kPa. The estimated settlement at this SLS bearing pressure is 25 mm. The factored geotechnical resistance at ULS is 150 kPa.

It is recommended that nominal reinforcing steel for stiffening of the basement foundations wall be provided to help mitigate minor cracking due to differential settlement. The reinforcing steel in the poured concrete foundation walls may consist of 2-15M bars continuous at the top of the foundation wall, and 2-15M bars continuous at the bottom of the foundation walls. Typically, these bars are placed 100 to 200 mm from the top or bottom of the foundation wall, respectively. A typical reinforcing steel detail for foundation walls placed on engineered fill is provided within Appendix C. The recommended nominal reinforcing steel should not be considered a structural

design. The need for different or additional reinforcement should be reviewed by a structural engineer to ensure the original structural design intent of the structure is maintained.

4.1.3 General Foundation Considerations

The foundation design parameters provided above are predicated on the assumption that the foundation subgrade surface is undisturbed, and that all deleterious, softened, disturbed and caved material is removed. The foundation excavation must be done in such a way that ground water is controlled to prevent any disturbance to the foundation base.

The foundation subgrade must be reviewed prior to concrete placement to ensure the above foundation design parameters are applicable, and to provide remedial recommendations if necessary. If the foundation excavation will be open for a prolonged period of time, the foundation subgrade should be protected with a skim coat of lean mix concrete (after inspection by the geotechnical engineer), to ensure that no deterioration due to weather effects occur. If foundation construction occurs in freezing conditions, the foundation subgrade must be protected from freezing.

Footings stepped from one level to another should be at a slope no steeper than 7 vertical to 10 horizontal. This concept should also be applied to excavations for new foundations in relation to existing footings or underground services, unless rigid shoring is provided. All footings and pile caps exposed to freezing conditions must be provided with a minimum of 1.2 metres of earth cover or equivalent insulation for frost protection.

4.2 Earth Pressures

Underground levels, basements, retaining walls, cantilevered shoring walls and shoring walls with a single level of earth anchors all must be designed to resist unbalanced lateral earth pressures imparted from the weight of adjacent soils. Lateral earth pressures are calculated using the following equation:

$$P = K[\gamma h + q]$$

where,

P	=	the horizontal pressure at depth, h (m)
K	=	the earth pressure coefficient (dimensionless)
h	=	depth below surface in metres
γ	=	the bulk unit weight of soil, (kN/m ³)
q	=	surcharge loading (kPa)

The above equation assumes that a drainage system is present which prevents the build up of any hydrostatic pressure behind the structure subjected to the unbalanced lateral earth pressures. If this is not the case, the equation must be revised to also incorporate the submerged unit weight of the soil multiplied by the earth pressure coefficient, in addition to the water pressure itself.

The values for use in the design of structures subjected to unbalanced lateral earth pressures at this site are as follows:

Soil Type	γ - Bulk Unit Weight (kN/m ³)	ϕ - Friction Angle (degrees)	Earth Pressure Coefficient (dimensionless)		
			K_a - Active	K_o - At-Rest	K_p - Passive
Earth Fill	19.0	28	0.36	0.53	2.77
Sandy Silt Glacial Till / Sand	21.0	34	0.28	0.44	3.54
Clay and Silt	20.0	32	0.31	0.47	3.25

The calculation of the earth pressure coefficients is based on Rankine theory, which provides a conservative estimate as no friction between the soil and the structure is accounted for. The earth pressure coefficients provided above are only applicable for flat ground surfaces beyond the structure and must be increased for sloping ground surfaces.

The earth pressure coefficients referenced within the above table are a function of the friction angle of the adjacent soil, and both the degree and direction of movement of the structure subjected to unbalanced lateral earth pressures. For structures that are restrained at the top (such as basement walls), the at-rest earth pressure coefficient will apply. For structures that allow for 0.1 to 1% of movement away from the soil, the full active earth pressure coefficient will apply. For structures that allow for 1 to 10% of movement into the soil, the full passive earth pressure coefficient will apply. The percentage movement is based on the height of the structure.

4.3 Floor Slab and Drainage

Where possible, the ground surface should be sloped on a positive grade away from the structure to promote surface water run-off and to reduce groundwater infiltration adjacent to foundations. To minimize infiltration of surface water, the upper 150 mm of backfill could comprise compacted relatively impervious soil material.

The existing proof-rolled earth fill and native soils are suitable for the support of a lightly supported unreinforced concrete slab on-grade, provided the upper topsoil layer is stripped and removed. The subgrade for the slab on grade must be assessed by the geotechnical engineer, prior to the placement of an aggregate base. If any soft or weak subgrade areas are identified, or if there are areas containing excessive amounts of deleterious/organic material, they must be locally sub-excavated and backfilled with approved clean earth fill or imported granular material and compacted to a minimum of 98% SPMDD.

All building floor slabs must be provided with a capillary moisture barrier and drainage layer. This is made by placing the concrete slab on a minimum 200 mm layer of 19 mm clear stone (OPSS.MUNI 1004) compacted by vibration to a dense state. The upper 50 mm of clear stone can be replaced with Granular 'A' (OPSS.MUNI 1010) for a working surface.

Perimeter drainage at the foundation level is not required, provided that the underside of concrete slab is at least 200 mm above the prevailing grade of the site and the surrounding surfaces slope away from the building at a gradient of at least 2 percent. If basements are constructed, a perimeter drainage system must be installed that will remove any ground water that infiltrates into the building backfill, to ensure that any water does not infiltrate into the basement. The perimeter drains must consist of minimum 100 mm diameter perforated pipes wrapped in filter socks, sufficiently covered on all sides by 19 mm clear stone. A typical detail for the basement drainage

provided in Appendix C. Perimeter drains should be directed to the sump underneath the basement floor in solid pipes so as not to surcharge the underfloor drainage layer with water.

5 Constructability Considerations

5.1 Excavations & Temporary Ground Water Control

Excavations must be carried out in accordance with the Occupational Health and Safety Act, Ontario Regulation 213/91 (as amended), Construction Projects, Part III - Excavations, Section 222 through 242.

Where workers must enter a trench or excavation the soil must be suitably sloped and/or braced in accordance with the OHSA. These regulations designate four (4) broad classifications of soils to stipulate appropriate measures for excavation safety. At this site, the native soils encountered near surface are considered Type 3 soils, which requires trench sidewalls to be constructed no steeper than 1 horizontal to 1 vertical from the base of the excavation.

Minimum support system requirements for steeper excavations are stipulated in Sections 235 through 238 and 241 of the OHSA and include provisions for timbering, shoring and moveable trench boxes. To reduce the potential for instability of the trench excavations, materials excavated from the service trenches and/or other fill materials or heavy equipment should not be placed near the crest of the trench excavations.

Based on the design concept for this site, excavations for servicing and foundations are not expected to extend into the prevailing ground water table. However, some ground water seepage may be encountered in the excavations, or water may pond at the base of the excavations during precipitation events. This seepage should diminish slowly and can be controlled by continuous pumping from a conventional sump pump arrangement at the base of the excavation.

It is important to note that soils encountered in the construction excavations may vary significantly across the site. Our preliminary soil classifications are based solely on the materials encountered in widely spaced explorations. The contractor should verify that similar conditions exist throughout the proposed area of excavation. If different subsurface conditions are encountered at the time of construction, we recommend that CEE be contacted immediately to evaluate the conditions encountered.

5.2 Quality Verification Services

On-site quality verification services are an integral part of the geotechnical design function, and for foundations and retaining walls, are required under the Ontario Building Code. Quality verification services are used to confirm that construction is being conducted in general conformance with the requirements as outlined in the drawings, reports and specifications prepared for the proposed development.

CEE can provide all the on-site quality verification services outlined below:

- The subgrade for shallow foundations may be field reviewed by the geotechnical engineer as required by the municipal regulating authority.

- Installation of retaining structures over 1.0 metres high and related backfilling operations must be field reviewed on a continuous basis by the geotechnical engineer.
- Part-time monitoring of the subgrade support capabilities, material quality, lift thickness, moisture content, degree of compaction, etc. is recommended for the following areas to ensure the recommendations within this report are followed and they perform adequately in the long-term:
 - o Slab-on-grades;
 - o Pavement structure (granulars and asphalt); and
 - o Bedding/backfilling of site servicing.
- Testing of the concrete (compressive strength, slump, air content, etc.) and testing of the asphalt (asphalt content and gradation) are recommended to ensure that the quality of the materials being brought to site meet the requirements of the project.

5.3 Site Work

The soils found at this site may become weakened when subjected to traffic, particularly when wet. If there is site work carried out during periods of wet weather, then it can be expected that the subgrade will be disturbed unless an adequate granular working surface is provided to protect the integrity of the subgrade soils from construction traffic. Subgrade preparation works cannot be adequately accomplished during wet weather and the project must be scheduled accordingly. The disturbance caused by the traffic can result in the removal of disturbed soil and use of granular fill material for site restoration or underfloor fill that is not intrinsic to the project requirements.

The most severe loading conditions on the subgrade may occur during construction. Consequently, special provisions such as end dumping and forward spreading of earth and aggregate fills, restricted construction lanes, and half-loads during paving and other work may be required, especially if construction is carried out during unfavourable weather.

If construction proceeds during freezing weather conditions, adequate temporary frost protection for the founding subgrade and concrete must be provided. The soil at this site is highly susceptible to frost damage. Consideration must be given to frost effects, such as heave or softening, on exposed soil surfaces in the context of this particular project development.

6 Limitations and Conclusion

6.1 Limitations

The investigation and comments are necessarily on-going as new information of underground conditions becomes available. More specific information with respect to the conditions between samples, or the lateral and vertical extent of materials may become apparent during excavation operations. The interpretation of the borehole information must, therefore, be validated during excavation operations. Consequently, during the future development of the property, conditions not observed during this investigation may become apparent. Should this occur, Central Earth Engineering Inc. should be contacted to assess the situation and additional testing and reporting may be required.

Central Earth Engineering Inc. should be retained for a general review of the final design drawings and specifications to verify that this report has been properly interpreted and implemented. If not accorded the privilege

of making this review, Central Earth Engineering Inc. will assume no responsibility for interpretation of the recommendations in the report. For example, it should be appreciated that modifications to bearing levels may be required if unforeseen subsoil conditions are revealed after the excavation is exposed to full view or if final design decisions differ from those assumed in this report.

The comments given in this report are intended only for the guidance of the design engineers. The number of boreholes required to determine the localized underground conditions between boreholes affecting construction costs, techniques, sequencing, equipment, scheduling, etc. could be greater than has been carried out for design purposes. Contractors bidding on or undertaking the works should, in this light, decide on their own investigations, as well as their own interpretations of the factual borehole results, so that they may draw their own conclusions as to how the subsurface conditions may affect them.

This report was prepared by Central Earth Engineering Inc. for the account of 2591451 Ontario Inc. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Central Earth Engineering Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this project

6.2 Conclusion

It is recognized that municipal/regional governing bodies, in their capacity as the planning and building authority under Provincial statutes, will make use of and rely upon this report, cognizant of the limitations thereof, both as are expressed and implied.

We trust this report is complete within our terms of reference, and the information presented is sufficient for your present purposes. If you have any questions, or when we may be of further assistance, please do not hesitate to contact our office.

Yours Truly,

Central Earth Engineering Inc.



Alexander Winkelmann, P.Eng.
President, Geotechnical Engineer

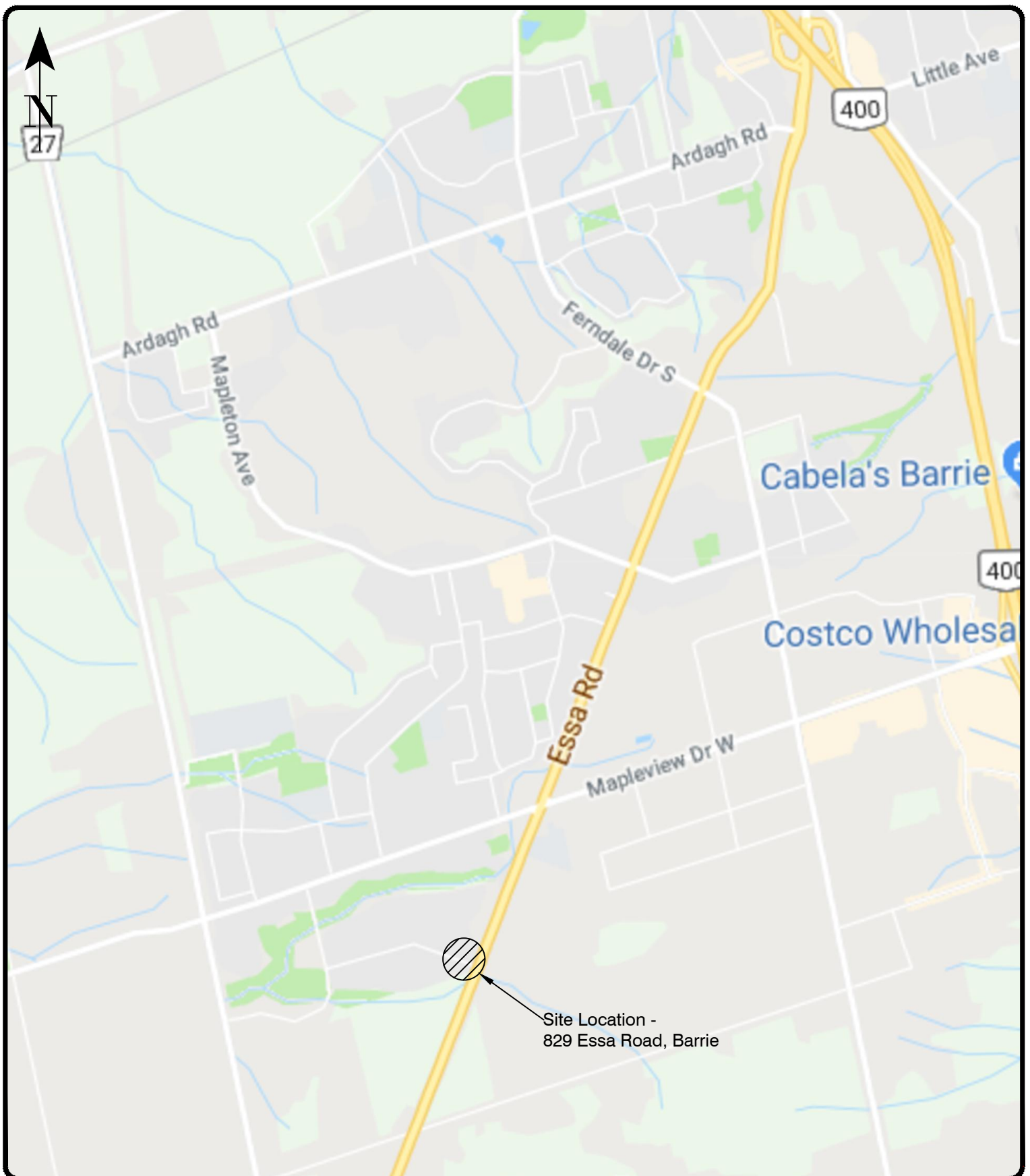


Figures –

SITE LOCATION PLAN

BOREHOLE LOCATION PLANS

SUBSURFACE CROSS-SECTION



Geotechnical Engineering and Construction
Materials Testing & Inspection

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Project:		829 Essa Road, Barrie	
Title:		SITE LOCATION PLAN	
Approved by:	A.W.	Date:	Dec 2018
Project No.:	18-1132A		
Drawn by:	E.P.	Scale:	1:25,000
Figure No.:	1		



Legend:

-  Approx. Borehole Location
-  Approx. Cross-Section Location

Reference:

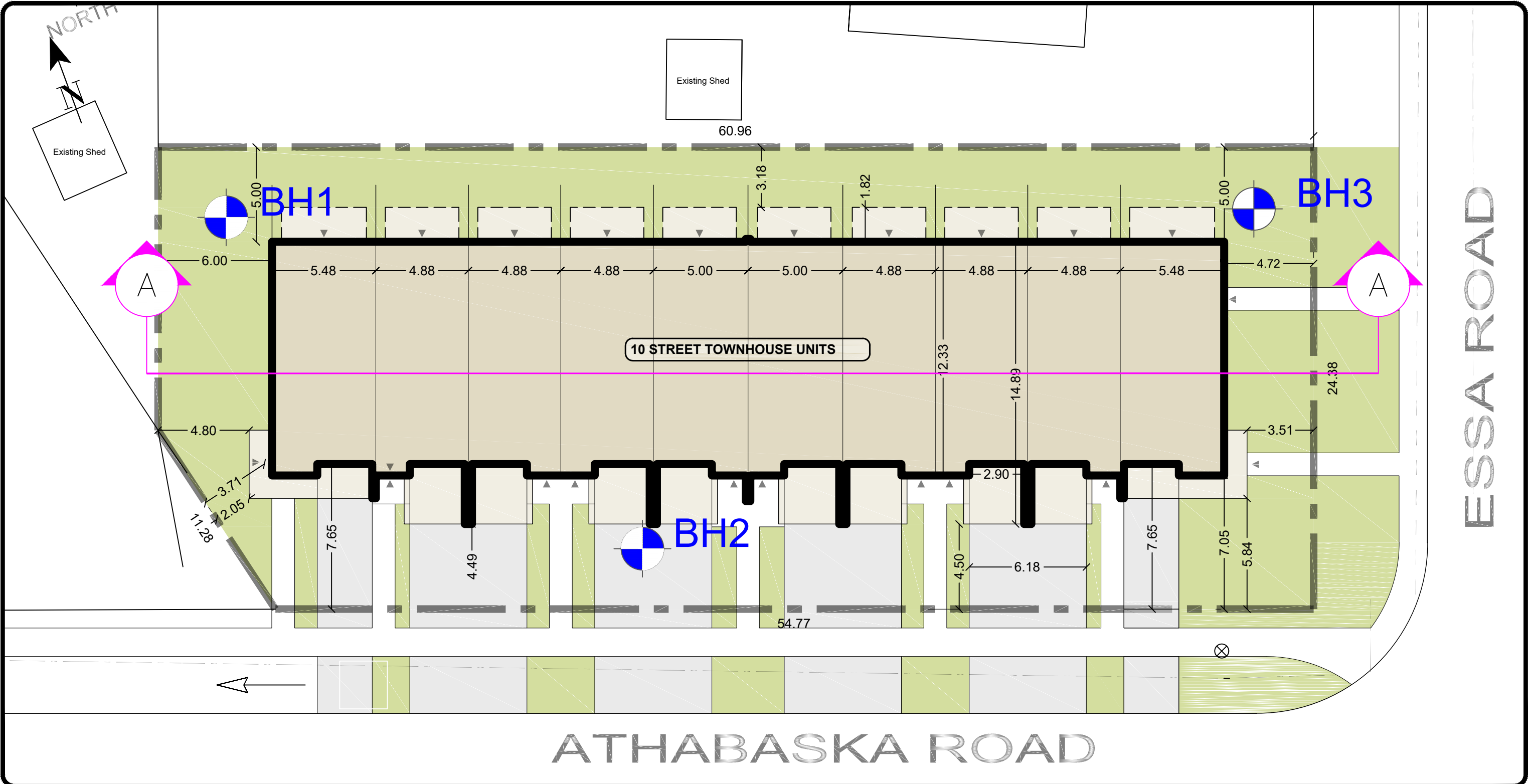
Simcoe County Maps, 2018



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Project:			829 Essa Road, Barrie		
Title:			BOREHOLE LOCATION PLAN (AERIAL)		
Approved by:		A.W.	Date:	Dec 2018	Project No.: 18-1132A
Drawn by:		E.P.	Scale:	1:200	Figure No.: 2A



Legend:

-  Approx. Borehole Location
-  Approx. Cross-Section Location

Reference:

Conceptual Site Plan by Innovative Planning Solutions, File 18-803, dated Oct. 29, 2018



Central Earth
ENGINEERING

*Geotechnical Engineering and Construction
Materials Testing & Inspection*

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Project: 829 Essa Road, Barrie		
Title: BOREHOLE LOCATION PLAN (PROPOSED)		
Approved by: A.W.	Date: Dec 2018	Project No.: 18-1132A
Drawn by: E.P.	Scale: 1:200	Figure No.: 2B

Appendix A – **BOREHOLE LOGS**

RECORD OF BOREHOLE No. 1



Project Number: **18-1132A**
 Project Client: **2591451 Ontario Inc.**
 Project Name: **829 Essa Road**
 Project Location: **Barrie, Ontario**
 Drilling Location: **Rear Yard, North West Corner of Site**

Drilling Method: **Solid Stem Augers** Drilling Machine: **Track Mount**
 Logged By: **RD** Northing: **4908453** Date Started: **2018-12-12**
 Reviewed By: **AW** Easting: **602105** Date Completed: **2018-12-12**

LITHOLOGY PROFILE		SOIL SAMPLING				DEPTH (m)	ELEVATION (m)	FIELD TESTING	LAB TESTING	Instrumentation Installation	COMMENTS & GRAIN SIZE DISTRIBUTION (%)
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value			Shear Strength Testing (kPa) × Other Test + Pocket Penetrometer ▲ Field Vane (Intact) △ Field Vane (Remolded) 40 80 120 160 Penetration Testing ○ SPT ● DCPT 10 20 30 40	△ Combustible Organic Vapour (ppm) ▲ Combustible Organic Vapour (%LEL) * Total Organic Vapour (ppm) 100 200 300 400 Atterberg Limits PL ——— LL ○ Water Content (%) 10 20 30 40		
Local 101.34m	SILTY SAND GLACIAL TILL, Trace Clay, Trace Gravel, Very Loose, Brown, Moist	SS	1	58	3	0	101	3	13		Topsoil = 150mm
	--- Compact, Light Brown ---	SS	2A 2B	100	15	0.5	100.5	15	7	21	
		SS	3	100	16	1	100	16	10		
	--- Dense, Brown ---	SS	4	100	37	1.5	99.5	37	8		2 64 26 8
3.2 98.1	SAND, Some Silt, Trace Gravel, Dense to Very Dense, Brown, Moist	SS	5A 5B	100	66	2.5	99	66	8		
	--- Wet ---	SS	6	100	46	3	98.5	46	13		3 77 (20)
		SS	7	83	39	3.5	98	39	18		
	End of Borehole @ 5.0m					4	97.5				
						4.5	97				
						5	96.5				

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Groundwater depth encountered on completion of drilling: **N/A**

Cave depth after auger removal: **None**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 : 50**
 Page: **1 of 1**

RECORD OF BOREHOLE No. 2



Project Number: **18-1132A**
 Project Client: **2591451 Ontario Inc.**
 Project Name: **829 Essa Road**
 Project Location: **Barrie, Ontario**
 Drilling Location: **Rear Yard, South Side of Site**

Drilling Method: **Solid Stem Augers** Drilling Machine: **Track Mount**
 Logged By: **RD** Northing: **4908436** Date Started: **2018-12-12**
 Reviewed By: **AW** Easting: **602120** Date Completed: **2018-12-12**

LITHOLOGY PROFILE		SOIL SAMPLING				DEPTH (m)	ELEVATION (m)	FIELD TESTING	LAB TESTING	Instrumentation Installation	COMMENTS & GRAIN SIZE DISTRIBUTION (%)				
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value			Shear Strength Testing (kPa)	Atterberg Limits		GR	SA	SI	CL	
								Other Test Pocket Penetrometer Field Vane (Intact) Field Vane (Remolded)	Combustible Organic Vapour (ppm) Combustible Organic Vapour (%LEL) Total Organic Vapour (ppm)						
							Penetration Testing SPT DCPT	Water Content (%) PL LL							
Local	100.16m														
	SILTY SAND CLACIAL TILL, Trace Clay, Trace Gravel, Very Loose, Brown, Moist	SS	1	100	3	0	100	3	14						Topsoil = 250mm
	--- Compact ---	SS	2	100	12	0.5	99.5	12	9						
		SS	3	100	20	1.5	98.5	20	8						
	2.3	97.9													
	CLAY AND SILT, Trace Sand, Very Stiff to Hard, Brown, Moist	SS	4	100	33	2.5	97.5	33	19						
		SS	5	100	24	3.5	96.5	24	19						
	--- Grey ---														
		SS	6	100	32	4.5	95.5	32	21						
		SS	7	100	21	5.5	94.5	21	25						
		SS	8	100	25	6.5	94	25	22						
		SS	9	100	26			26	25						
	End of Borehole @ 6.6m														

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 W: www.centralearth.com

Groundwater depth encountered on completion of drilling: **N/A**

Cave depth after auger removal: **None**

Groundwater depth observed on **Dec. 18/18** at a depth of: **Dry**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

Scale: **1 : 50**

Page: **1 of 1**

RECORD OF BOREHOLE No. 3



Project Number: **18-1132A**
 Project Client: **2591451 Ontario Inc.**
 Project Name: **829 Essa Road**
 Project Location: **Barrie, Ontario**
 Drilling Location: **Front Yard, North East Corner of Site**

Drilling Method: **Solid Stem Augers** Drilling Machine: **Track Mount**
 Logged By: **RD** Northing: **4908440** Date Started: **2018-12-12**
 Reviewed By: **AW** Easting: **602152** Date Completed: **2018-12-12**

LITHOLOGY PROFILE		SOIL SAMPLING				DEPTH (m)	ELEVATION (m)	FIELD TESTING		LAB TESTING		Instrumentation Installation	COMMENTS & GRAIN SIZE DISTRIBUTION (%)			
Lithology Plot	DESCRIPTION	Sample Type	Sample Number	Recovery (%)	SPT "N" Value			Shear Strength Testing (kPa)	Penetration Testing	Atterberg Limits	Water Content (%)		GR	SA	SI	CL
Local	99.74m															
	FILL: Silty Sand, Trace Clay, Trace Gravel, Trace Organics/Rootlets, Very Loose to Loose, Brown, Moist	SS	1	83	3	0	99.5	3			11					
		SS	2	100	5	0.5	99	5			11					
		SS	3	78	5	1	98.5	5			9					
2.4	--- Wet --- 97.3	SS	4A	67	50/6"	1.5	98				13					
	SILTY SAND GLACIAL TILL, Trace Clay, Trace Gravel, Very Dense, Brown, Damp	SS	4B			2	97.5	50/6"	5		5					
		SS	5	22	50/6"	2.5	97	50/6"	5		5					
		SS	6	67	89	3	96.5	89			13					
3.8	95.9	SS	7	22	65	3.5	96	65			21					
	CLAY AND SILT, Trace Sand, Hard, Grey, Moist					4	95.5									
	End of Borehole @ 5.0m					4.5	95									
						5										

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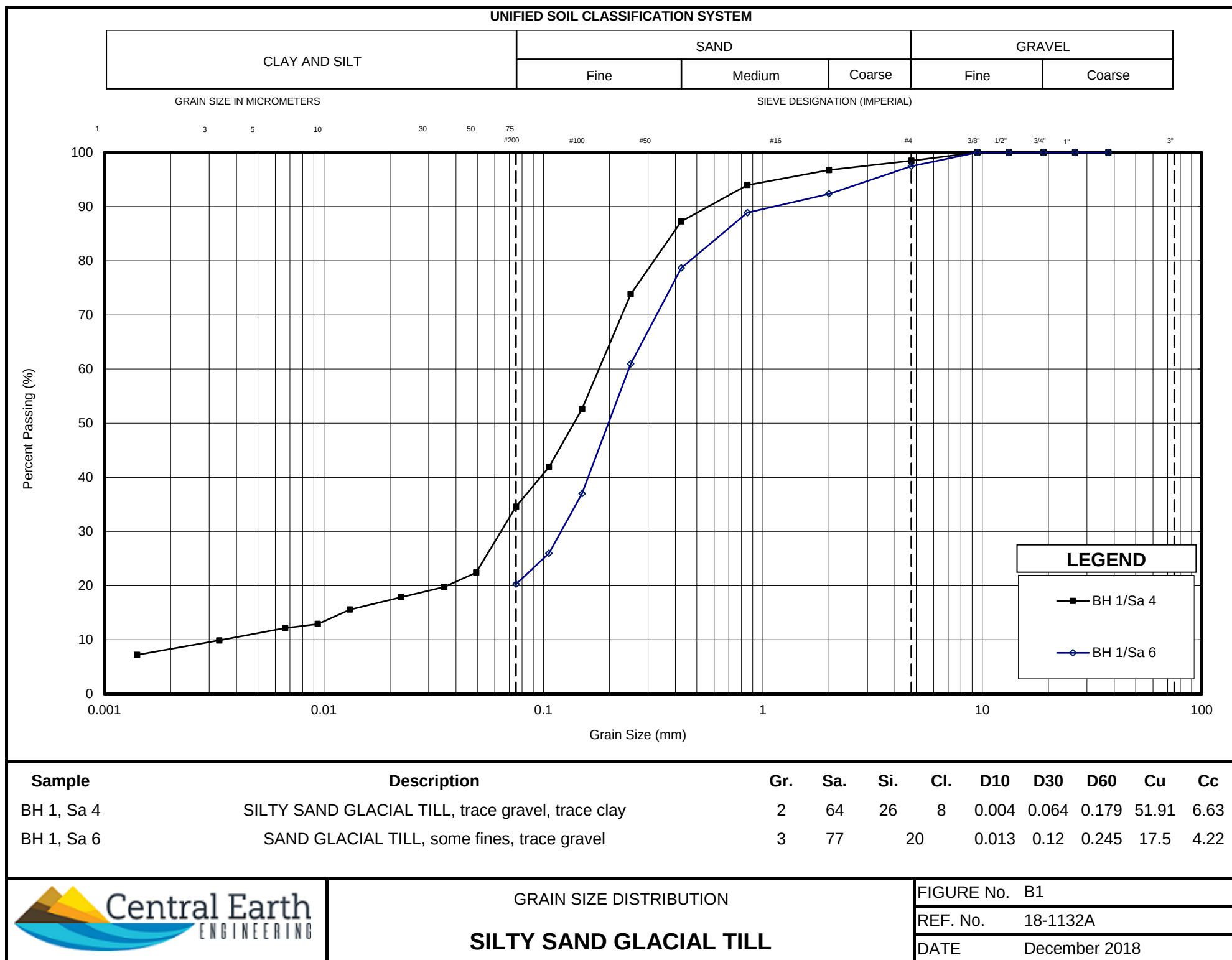
Groundwater depth encountered on completion of drilling: **N/A**

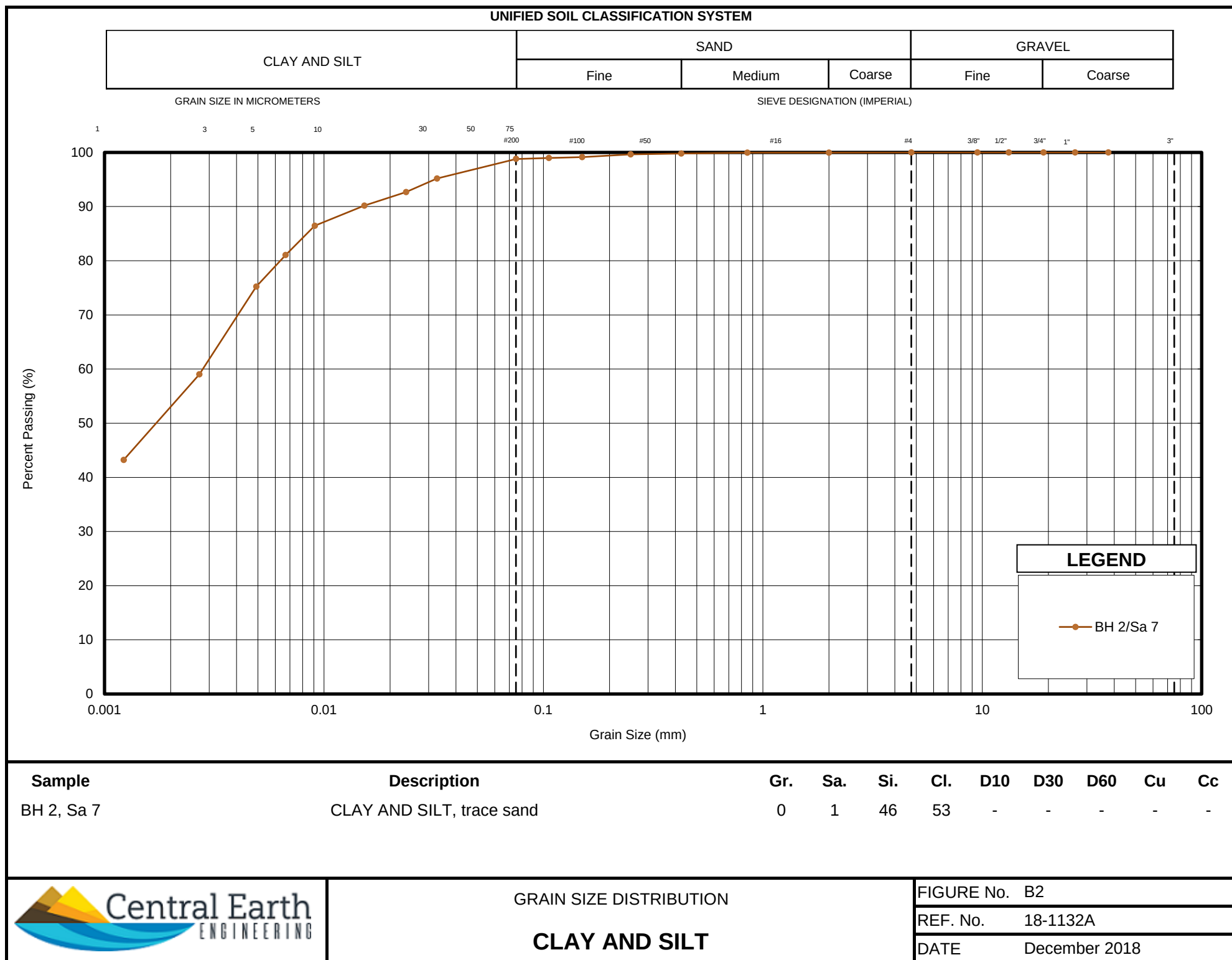
Cave depth after auger removal: **None**

Borehole details presented do not constitute a thorough understanding of all potential conditions present and require interpretative assistance from a qualified geotechnical engineer. Also, borehole information should be read in conjunction with the geotechnical report for which it was commissioned and the accompanying 'Explanation of Boring Log'.

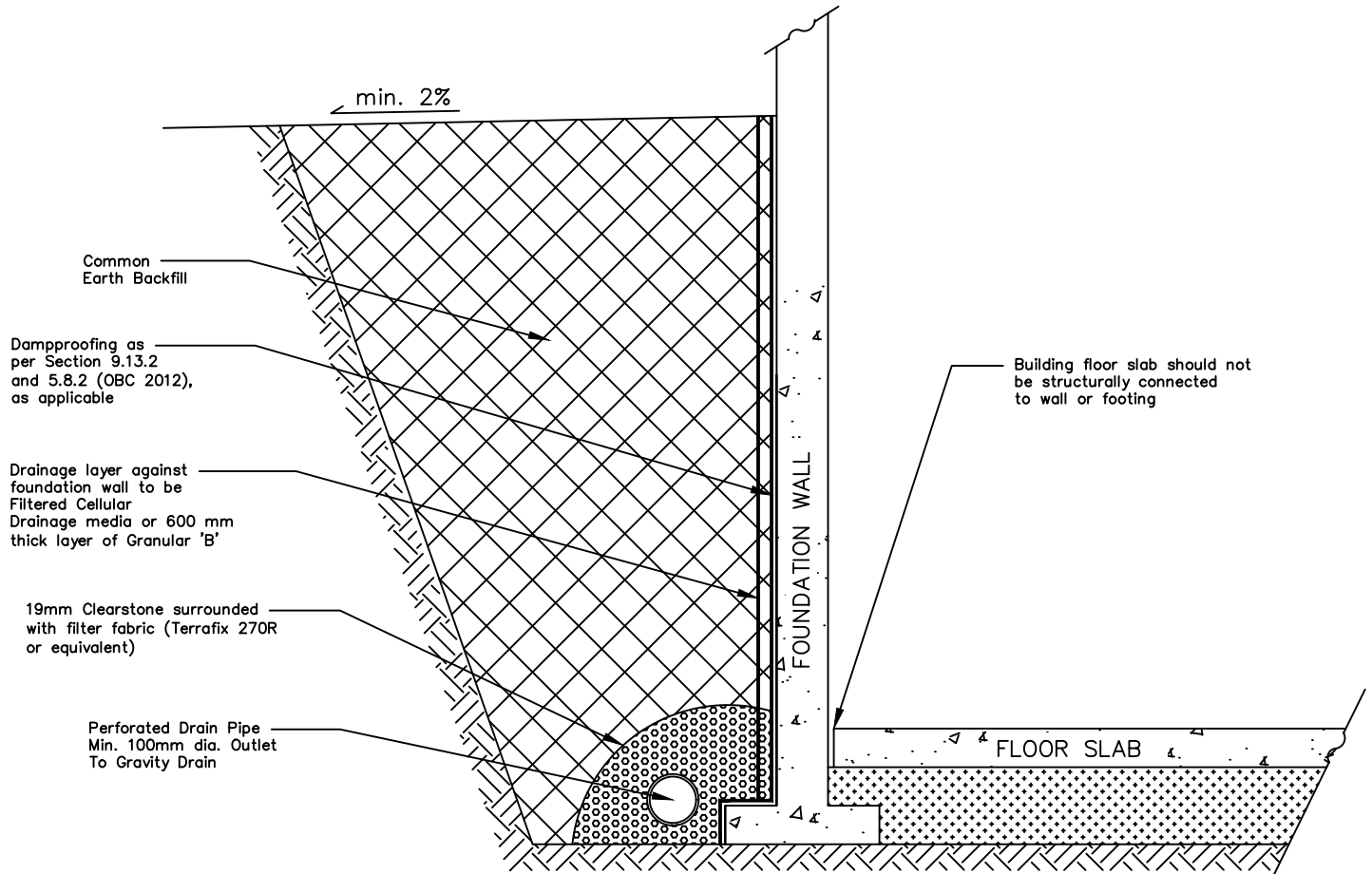
Scale: **1 : 50**
 Page: **1 of 1**

Appendix B –
GRAIN SIZE LABORATORY RESULTS





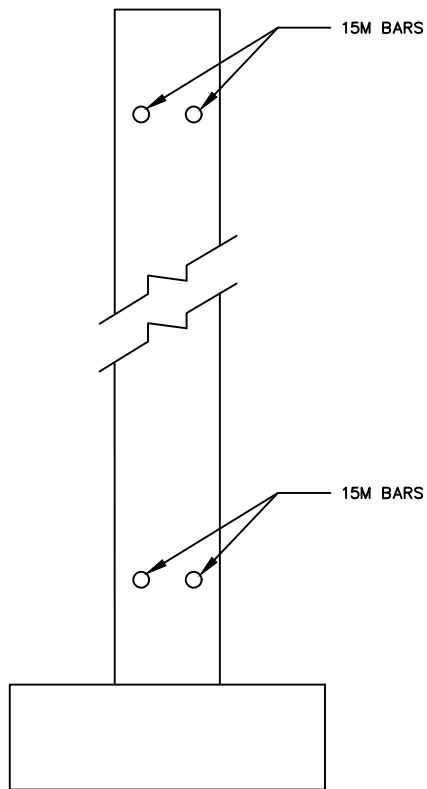
Appendix C – **TYPICAL DETAILS**



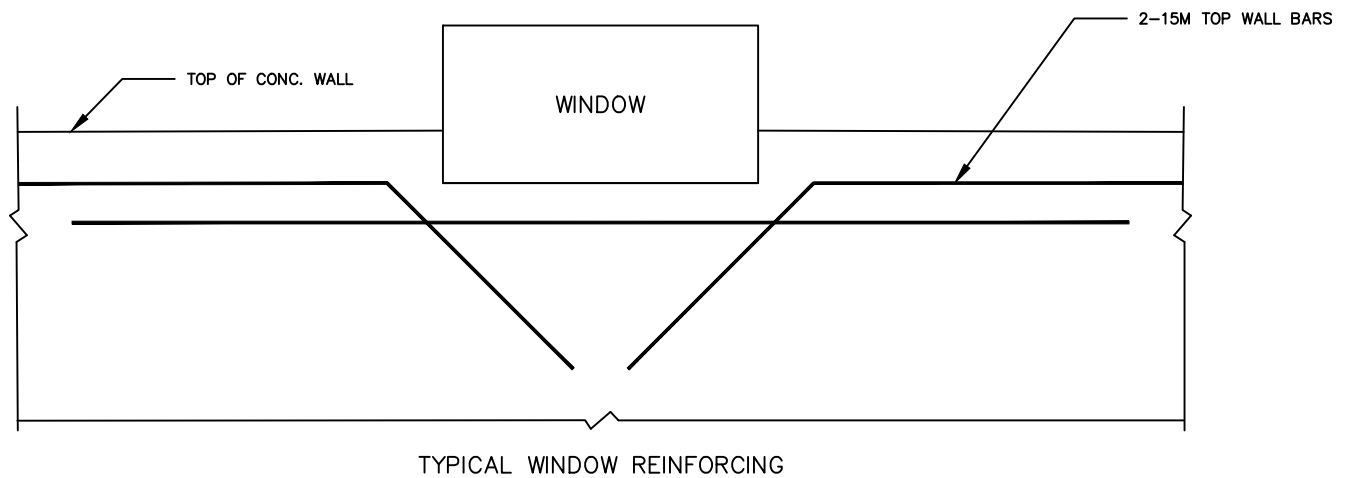
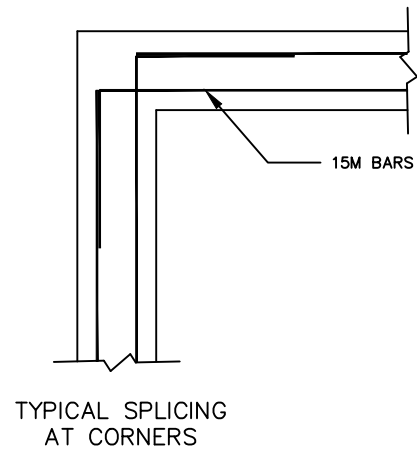
Geotechnical Engineering and Construction
Materials Testing & Inspection

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BASEMENT FOUNDATION WALL DRAINAGE TYPICAL DETAIL



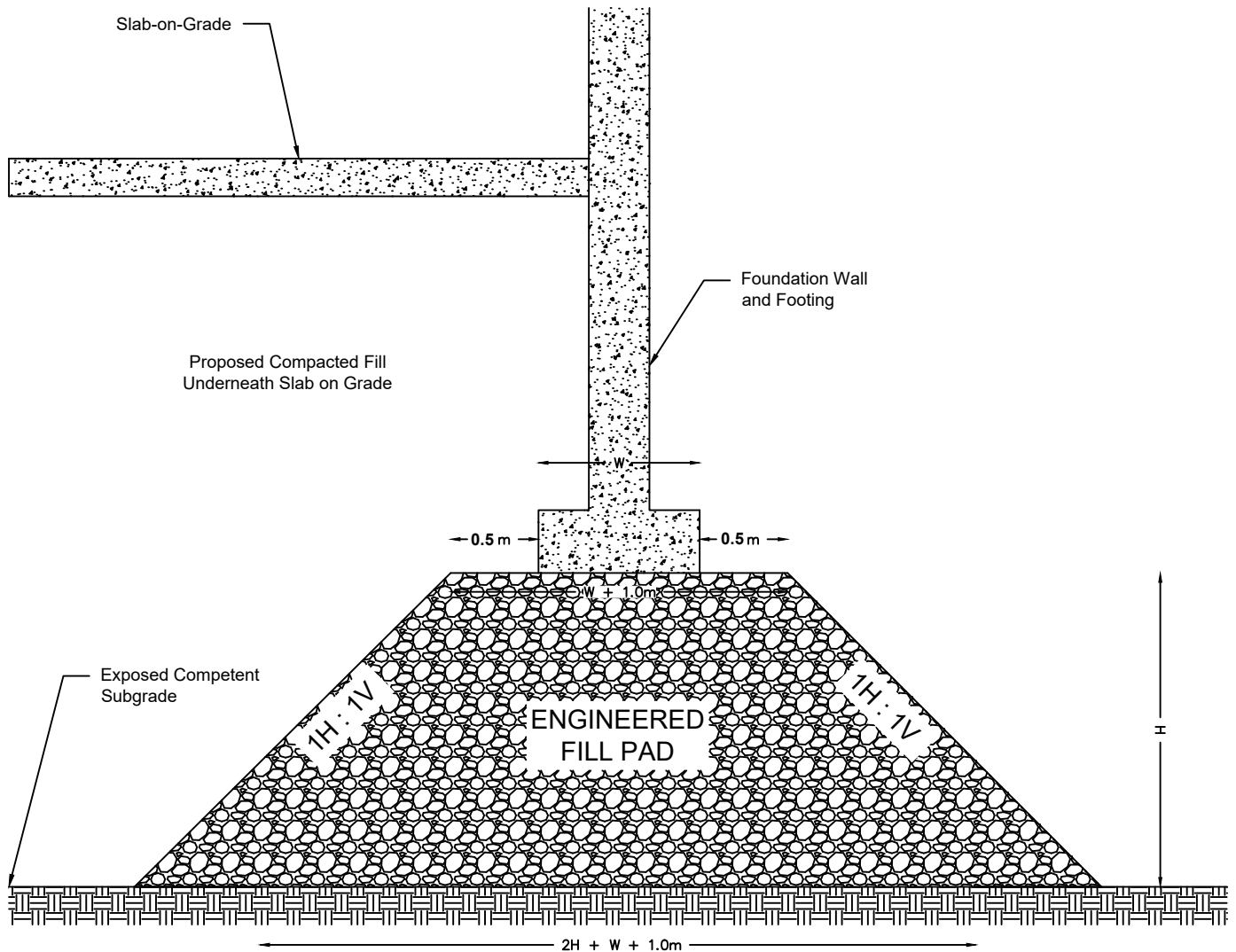
TYPICAL REINFORCED
WALL



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FOUNDATION WALL REINFORCING ON ENGINEERED FILL TYPICAL DETAIL



Design Chart 1.08: Hydrologic Soil Groups (Continued)**- Based on Soil Texture**

<u>Sands, Sandy Loams and Gravels</u>	
- overlying sand, gravel or limestone bedrock, very well drained	A
- ditto, imperfectly drained	AB
- shallow, overlying Precambrian bedrock or clay subsoil	B
<u>Medium to Coarse Loams</u>	
- overlying sand, gravel or limestone, well drained	AB
- shallow, overlying Precambrian bedrock or clay subsoil	B
<u>Medium Textured Loams</u>	
- shallow, overlying limestone bedrock	B
- overlying medium textured subsoil	BC
<u>Silt Loams, Some Loams</u>	
- with good internal drainage	BC
- with slow internal drainage and good external drainage	C
<u>Clays, Clay Loams, Silty Clay Loams</u>	
- with good internal drainage	C
- with imperfect or poor external drainage	C
- with slow internal drainage and good external drainage	D

Source: U.S. Department of Agriculture (1972)

**RESIDENTIAL TOWNHOUSE DEVELOPMENT
829 ESSA ROAD – CITY OF BARRIE
STORM WATER MANAGEMENT & FUNCTIONAL SERVICING REPORT**

APPENDIX B

Design Calculations

3.2.1 Service Area

The drainage system shall be designed to accommodate all upstream drainage areas plus any external area tributary to the system for the existing, interim and ultimate development conditions, as determined by the delineation of appropriate topographic mapping and the preparation of drainage plans.

3.2.2 Design Flow

Storm sewer systems with a drainage area ≤ 50 ha shall be designed to convey the 1:5 year (minimum) design storm using the Rational Method and the City's IDF regression equation for rainfall intensity unless otherwise approved or directed by the City. Storm sewer systems with a drainage area > 50 ha shall be designed using an approved computer program and verified with the Rational Method. The storm sewer design shall be based on the larger of the two flows calculated using the computer model and the Rational Method. Under no circumstances shall the storm system be designed in a surcharged condition.

The design of the storm sewers shall be computed using the City of Barrie's Storm Sewer Design Sheet as provided in **Appendix A**.

All storm sewers shall be designed according to the Rational Formula where:

$$Q = \frac{(C)(i)(A)}{360}$$

where,

- Q = the design flow in (m³/s)
- C = the site specific runoff coefficient
- A = the drainage area (ha)
- i = rainfall intensity (mm/hr)

The rainfall intensity shall be calculated in accordance with the following table and equation:

Table 3.1: Barrie WPCC IDF Curve Parameters - Adjusted to Account for Climate Change

Parameter	Return Period					
	2-Yr	5-Yr	10-Yr	25-Yr	50-Yr	100-Yr
A	678.085	853.608	975.865	1146.275	1236.152	1426.408
B	4.699	4.699	4.699	4.922	4.699	5.273
C	0.781	0.766	0.760	0.757	0.751	0.759

Rainfall Intensity, I (mm/hr) = $A/(t+B)^C$, where t is time duration in minutes

Parameters based on rain gauge data for the period 1979 – 2003 for the Barrie WPCC Station #6110557

Based on a review of the literature, the IDF intensity values for Barrie WPCC Station were increased by 15% before calculating a, b, c values to account for climate change

$$i = \frac{A}{(t_d + B)^C}$$

where,

- i = the rainfall intensity (mm/hr)
- t_d = the storm duration (minutes)
- A, B, C = a function of the local intensity-duration data.

The storm duration is set to the time of concentration (i.e. the sewer inlet time plus the time of travel in the pipe or channel) for the total cumulative drainage area to the node of interest. The maximum inlet time for the first pipe of a storm sewer system is 10 minutes.

The runoff coefficient shall be calculated in accordance with the following table:

Table 3.2: Runoff Coefficients (Rational C) (5-yr to 10-yr) Based on Hydrologic Soil Group

Land Use	Runoff Coefficient "C"		
	A-AB	B-BC	C-D
Cultivated Land, 0 - 5% grade	0.22	0.35	0.55
Cultivated Land, 5 - 10% grade	0.30	0.45	0.60
Cultivated Land, 10 - 30% grade	0.40	0.65	0.70
Pasture Land, 0 - 5% grade	0.10	0.28	0.40
Pasture Land, 5 - 10% grade	0.15	0.35	0.45
Pasture Land, 10 - 30% grade	0.22	0.40	0.55
Woodlot or Cutover, 0 - 5% grade	0.08	0.25	0.35
Woodlot or Cutover, 5 - 10% grade	0.12	0.30	0.42
Woodlot or Cutover, 10 - 30% grade	0.18	0.35	0.52
Lakes and Wetlands	0.05	0.05	0.05
Impervious Area (i.e., buildings, roads, parking lots, etc.)	0.95	0.95	0.95
Gravel (not to be used for proposed parking or storage areas)	0.40	0.50	0.60
Residential – Single Family	0.30	0.40	0.50
Residential – Multiple (i.e., semi, townhouse, apartment)	0.50	0.60	0.70
Industrial – light	0.55	0.65	0.75
Industrial – heavy	0.65	0.75	0.85
Commercial	0.60	0.70	0.80
Unimproved Areas	0.10	0.20	0.30
Lawn, < 2% grade	0.05	0.11	0.17
Lawn, 2 - 7% grade	0.10	0.16	0.22
Lawn, > 7% grade	0.15	0.25	0.35

Adapted from Design Chart 1.07, Ontario Ministry of Transportation, "MTO Drainage Management Manual," MTO. (1997)

An approximation of the runoff coefficient can be calculated based on the following relationship with:

$$c = (0.7)(TIMP) + 0.2$$

where,

c = the runoff coefficient

TIMP = total impervious fraction (dimensionless)

The runoff coefficient shall be adjusted for return period events greater than the 10-yr storm per the following table:



Table 3.3: Runoff Coefficient Adjustment for 25-yr to 100-yr Storms

Return Period	Runoff Coefficient "C"
25 years	$C_{25} = 1.1 \cdot C_5$
50 years	$C_{50} = 1.2 \cdot C_5$
100 years	$C_{100} = 1.25 \cdot C_5$

Adapted from Design Chart 1.07, Ontario Ministry of Transportation,
"MTO Drainage Management Manual," MTO, (1997).

Note: When applying the runoff coefficient adjustment, the maximum c-value should not exceed 1.0.

Given that the direct connection of foundation drains to the storm sewer is not permitted, a detailed HGL analysis is typically not required unless deemed otherwise by the City due to special circumstances. Refer to **Section 7.3** for details regarding HGL analysis requirements.

The calculation of total percent impervious (TIMP) values for modeling shall be in accordance with **Section 7.2.5** (**Table 7.6**).

3.2.3 Pipe Capacity and Size

The storm sewer capacity shall be calculated using the Manning's equation assuming the pipe is flowing full as follows:

$$Q = \left[\frac{1}{n} \right] A(R)^{\frac{2}{3}} (S)^{\frac{1}{2}}$$

where,

- Q = the pipe capacity (m³/s)
- n = the Manning roughness value
- R = the hydraulic radius (m)
- S = the sewer pipe slope (m/m).

A maximum inlet time of 10 minutes shall be used for the first pipe of a storm sewer system.

The velocity of flow in the storm sewer (assuming pipe flowing full) shall be calculated as follows:

$$v = \left[\frac{Q}{A} \right]$$

where,

- Q = flow in the pipe when flowing full (m³/s)
- A = cross sectional area of the pipe (m²)

The appropriate roughness coefficients shall be used as identified in **Table 3.4**.

The minimum size for a storm sewer (within a street) shall be 300 mm in diameter. No decrease of pipe size from a larger size upstream to a smaller size downstream shall be allowed regardless of the increase in grade.

3.2.4 Roughness Coefficients

The following roughness coefficients shall be used for hydraulic calculations of storm sewers:

829 Essa Road Townhomes
MODIFIED RATIONAL METHOD - PRE-DEVELOPMENT CATCHMENT 101
City of Barrie

Project Number: 18-11409B
Date: December 21, 2020
Design By: JHV
File: Z:\Project Documents\11409B 829 Essa Road Townhomes\DEC 2020 REVISION\FSR & SWMMRM SWM Calculations.xls

IDF Curve Parameters				Intensity (mm/hr)
Storm Event	A	B	C	
2 year	678.085	4.699	0.7810	83.11
5 year	853.608	4.699	0.7660	108.92
10 year	975.865	4.699	0.7600	126.55
25 year	1146.275	4.922	0.7570	148.15
50 year	1236.152	4.699	0.7510	164.22
100 year	1426.408	5.273	0.7590	180.15

Tc = 10.0 minutes

$$i = \frac{A}{(t_c + B)^C}$$

I = average rainfall intensity (mm/hr)
A,B,C, = the IDF equation coefficients (dimensionless)
T_c = time of concentration (min)

Runoff Coefficients	
Land Use	"C"
Unimproved Area	0.20
Pasture Land	0.28
Woodlot	0.30
Lakes / Swamps	0.05
Impervious Area	0.95
Building Area	0.95
Gravel	0.50
Lawn	0.16
X	0.00
Y	0.00
Z	0.00

Time of Concentration Calculations:

Catchment Parameters			
Catchment ID	=	101	ha
Catchment Area	=	0.1742	
Flow Length	=	35	m
Slope	=	0.03	m/m
Weighted Runoff Coefficient	=	0.39	

Pre-Development Runoff Coefficients:

Catchment	Total Area (m²)	Unimproved Area (m²)	Pasture Area (m²)	Woodlot Area (m²)	Lakes/Swamps Area (m²)	Impervious Area (m²)	Building Area (m²)	Gravel Area (m²)	Lawn Area (m²)	X Area (m²)	Y Area (m²)	Z Area (m²)	Weighted C
101	1,742			513		415			814				0.39

Runoff Coefficient Adjustment for 25-100 yr Storm Events:

Catchment	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
101	0.39	0.39	0.39	0.43	0.47	0.49

Pre-Development Peak Flow Rates:

Catchment	2-Year (m³/s)	5-Year (m³/s)	10-Year (m³/s)	25-Year (m³/s)	50-Year (m³/s)	100-Year (m³/s)
101	0.016	0.021	0.024	0.031	0.037	0.042

Notes:
1) Runoff coefficients from City of Barrie Storm Drainage and SWM Policies and Design Guidelines Table 3.2
2) Runoff coefficients for events greater than the 10 year storm have been adjusted as per City Policies and Guidelines Table 3.3

829 Essa Road Townhomes

MODIFIED RATIONAL METHOD - POST DEVELOPMENT CATCHMENT 201

City of Barrie

Project Number: 18-11409B

Date: December 21, 2020

Design By: JHV

File: Z:\Project Documents\11409B 829 Essa Road Townhomes\DEC 2020 REVISION\FSR & SWMMRM SWM Calculations.xls

IDF Curve Parameters				Intensity (mm/hr)
Storm Event	A	B	C	
2 year	678.085	4.699	0.7810	83.11
5 year	853.608	4.699	0.7660	108.92
10 year	975.865	4.699	0.7600	126.55
25 year	1146.275	4.922	0.7570	148.15
50 year	1236.152	4.699	0.7510	164.22
100 year	1426.408	5.273	0.7590	180.15

Tc = 10.0 minutes

i =

A

(tc + B)^c

I = average rainfall intensity (mm/hr)

A,B,C, = the IDF equation coefficients (dimensionless)

T_c = time of concentration (min)

Runoff Coefficients	
Land Use	"C"
Unimproved Area	0.20
Pasture Land	0.28
Woodlot	0.30
Lakes / Swamps	0.05
Impervious Area	0.95
Building Area	0.95
Gravel	0.50
Lawn	0.16
X	0.00
Y	0.00
Z	0.00

Time of Concentration Calculations:

Catchment Parameters		
Catchment ID	= 201	ha
Catchment Area	= 0.0755	
Flow Length	= 10	m
Slope	= 0.02	m/m
Weighted Runoff Coefficient	= 0.69	

Post-Development Runoff Coefficients:

Catchment	Total Area (m²)	Unimproved Area (m²)	Pasture Area (m²)	Woodlot Area (m²)	Lakes/Swamps Area (m²)	Impervious Area (m²)	Building Area (m²)	Gravel Area (m²)	Lawn Area (m²)	X Area (m²)	Y Area (m²)	Z Area (m²)	Weighted C
201	755					135	370		250				0.69

Runoff Coefficient Adjustment for 25-100 yr Storm Events:

Catchment	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
201	0.69	0.69	0.69	0.76	0.83	0.86

Post-Development Peak Flow Rates:

Catchment	2-Year (m³/s)	5-Year (m³/s)	10-Year (m³/s)	25-Year (m³/s)	50-Year (m³/s)	100-Year (m³/s)
201	0.012	0.016	0.018	0.024	0.028	0.033

Notes:

1) Runoff coefficients from City of Barrie Storm Drainage and SWM Policies and Design Guidelines Table 3.2

2) Runoff coefficients for events greater than the 10 year storm have been adjusted as per City Policies and Guidelines Table 3.3

829 Essa Road Townhomes

MODIFIED RATIONAL METHOD - POST DEVELOPMENT CATCHMENT 202

City of Barrie

Project Number: 18-11409B

Date: December 21, 2020

Design By: JHV

File: Z:\Project Documents\11409B 829 Essa Road Townhomes\DEC 2020 REVISION\FSR & SWM\MRM SWM Calculations.xls

IDF Curve Parameters				Intensity (mm/hr)
Storm Event	A	B	C	
2 year	678.085	4.699	0.7810	83.11
5 year	853.608	4.699	0.7660	108.92
10 year	975.865	4.699	0.7600	126.55
25 year	1146.275	4.922	0.7570	148.15
50 year	1236.152	4.699	0.7510	164.22
100 year	1426.408	5.273	0.7590	180.15

Tc = 10.0 minutes

i =

A

(tc + B)^c

I = average rainfall intensity (mm/hr)

A,B,C, = the IDF equation coefficients (dimensionless)

T_c = time of concentration (min)

Runoff Coefficients	
Land Use	"C"
Unimproved Area	0.20
Pasture Land	0.28
Woodlot	0.30
Lakes / Swamps	0.05
Impervious Area	0.95
Building Area	0.95
Gravel	0.50
Lawn	0.16
X	0.00
Y	0.00
Z	0.00

Time of Concentration Calculations:

Catchment Parameters		
Catchment ID	= 202	ha
Catchment Area	= 0.0987	
Flow Length	= 30	m
Slope	= 0.02	m/m
Weighted Runoff Coefficient	= 0.44	

Post-Development Runoff Coefficients:

Catchment	Total Area (m ²)	Unimproved Area (m ²)	Pasture Area (m ²)	Woodlot Area (m ²)	Lakes/Swamps Area (m ²)	Impervious Area (m ²)	Building Area (m ²)	Gravel Area (m ²)	Lawn Area (m ²)	X Area (m ²)	Y Area (m ²)	Z Area (m ²)	Weighted C
202	987						350		637				0.44

Runoff Coefficient Adjustment for 25-100 yr Storm Events:

Catchment	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
202	0.44	0.44	0.44	0.48	0.53	0.55

Post-Development Peak Flow Rates:

Catchment	2-Year (m ³ /s)	5-Year (m ³ /s)	10-Year (m ³ /s)	25-Year (m ³ /s)	50-Year (m ³ /s)	100-Year (m ³ /s)
202	0.010	0.013	0.015	0.020	0.024	0.027

Notes:

1) Runoff coefficients from City of Barrie Storm Drainage and SWM Policies and Design Guidelines Table 3.2

2) Runoff coefficients for events greater than the 10 year storm have been adjusted as per City Policies and Guidelines Table 3.3

829 Essa Road Townhomes

MODIFIED RATIONAL METHOD - PRE TO POST DEVELOPMENT FLOW SUMMARY - NO INFILTRATION

City of Barrie

Project Number:

18-11409B

Date:

December 21, 2020

Design By:

JHV

File:

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Storm Event	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)
Pre-Development Catchment 101	0.016	0.021	0.024	0.031	0.037	0.042
Total Pre-Development Flow	0.016	0.021	0.024	0.031	0.037	0.042
Post Development Catchment 201 (uncontrolled)	0.012	0.016	0.018	0.024	0.028	0.033
Post Development Catchment 202 (no infiltration)	0.010	0.013	0.015	0.020	0.024	0.027
Total Post Development Flow	0.022	0.029	0.034	0.043	0.052	0.060
Total Difference from Post to Pre	0.006	0.008	0.010	0.012	0.015	0.017

829 Essa Road Townhomes

MODIFIED RATIONAL METHOD - POST DEVELOPMENT CATCHMENT 202 WITH RAIN GARDENS

City of Barrie

Project Number: 18-11409B

Date: December 21, 2020

Design By: JHV

File: Z:\Project Documents\11409B 829 Essa Road Townhomes\DEC 2020 REVISION\FSR & SWM\MRM SWM Calculations.xls

IDF Curve Parameters				Intensity (mm/hr)
Storm Event	A	B	C	
2 year	678.085	4.699	0.7810	83.11
5 year	853.608	4.699	0.7660	108.92
10 year	975.865	4.699	0.7600	126.55
25 year	1146.275	4.922	0.7570	148.15
50 year	1236.152	4.699	0.7510	164.22
100 year	1426.408	5.273	0.7590	180.15

Tc = 10.0 minutes

i =

A

(tc + B)^c

I = average rainfall intensity (mm/hr)

A,B,C, = the IDF equation coefficients (dimensionless)

T_c = time of concentration (min)

Runoff Coefficients	
Land Use	"C"
Unimproved Area	0.20
Pasture Land	0.28
Woodlot	0.30
Lakes / Swamps	0.05
Impervious Area	0.95
Building Area	0.95
Gravel	0.50
Lawn	0.16
X	0.00
Y	0.00
Z	0.00

Time of Concentration Calculations:

Catchment Parameters		
Catchment ID	= 202	
Catchment Area	= 0.0637	ha
Flow Length	= 10	m
Slope	= 0.25	m/m
Weighted Runoff Coefficient	= 0.16	

Post-Development Runoff Coefficients:

Catchment	Total Area (m²)	Unimproved Area (m²)	Pasture Area (m²)	Woodlot Area (m²)	Lakes/Swamps Area (m²)	Impervious Area (m²)	Building Area (m²)	Gravel Area (m²)	Lawn Area (m²)	X Area (m²)	Y Area (m²)	Z Area (m²)	Weighted C
202	637								637				0.16

Runoff Coefficient Adjustment for 25-100 yr Storm Events:

Catchment	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year
202	0.16	0.16	0.16	0.18	0.19	0.20

Post-Development Peak Flow Rates:

Catchment	2-Year (m³/s)	5-Year (m³/s)	10-Year (m³/s)	25-Year (m³/s)	50-Year (m³/s)	100-Year (m³/s)
202	0.002	0.003	0.004	0.005	0.006	0.006

Notes:

1) Runoff coefficients from City of Barrie Storm Drainage and SWM Policies and Design Guidelines Table 3.2

2) Runoff coefficients for events greater than the 10 year storm have been adjusted as per City Policies and Guidelines Table 3.3

829 Essa Road Townhomes

MODIFIED RATIONAL METHOD - PRE TO POST DEVELOPMENT FLOW SUMMARY - INFILTRATION (LIDs)

City of Barrie

Project Number:

18-11409B

Date:

December 21, 2020

Design By:

JHV

File:

Z:\Project Documents\11409B 829 Essa Road Townhomes\DEC 2020 REVISION\FSR & SWMMRM SWM Calculations.xls



Storm Event	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)	(m ³ /s)
Pre-Development Catchment 101	0.016	0.021	0.024	0.031	0.037	0.042
Total Pre-Development Flow	0.016	0.021	0.024	0.031	0.037	0.042
Post Development Catchment 201 (uncontrolled)	0.012	0.016	0.018	0.024	0.028	0.033
Post Development Catchment 202 (roof top infiltration)	0.002	0.003	0.004	0.005	0.006	0.006
Total Post Development Flow	0.014	0.019	0.022	0.028	0.034	0.039
Total Difference from Post to Pre	-0.001	-0.002	-0.002	-0.003	-0.003	-0.004

Rain Garden Design Calculations

The following is a list of parameters and design criteria for one rear-yard rain garden:

- The capacity to store runoff from a 54m² rooftop catchment area for each end unit and a 38m² catchment area for each middle unit.
- A rainfall depth of 20mm - According to the MECP Stormwater Management Planning and Design Manual (2003), the target storage volume should be 20mm over the rooftop area since 90% of all daily rainfall depths are less than this amount.
- A runoff coefficient of 0.95 for impervious area (Table 3.2, MECP Manual).
- A percolation rate of 20mm/hr for native soil (as per the geotechnical report provided by Central Earth Engineering).
- A recommended drawdown time of 48 hours per the LID Manual.
- A soil porosity of 0.4 for the recommended use of 50mm diameter clear stone as the storage media.
- An estimated subsurface storage media depth of 0.7m and surface storage depth of 0.2m.
- A longevity factor of 0.5 (as per Table 4.12, MECP Manual), considering the loss of permeability and compaction of the stone reservoir over time.

END UNIT RAIN GARDEN

$$\text{Runoff Volume} = C * i * A$$

where:

C	= surface runoff coefficient
i	= intensity, mm/hr.
A	= runoff catchment area, m ²

$$\text{Runoff Volume} = (0.95)(0.02m)(54m^2) = 1.026m^3 \sim 1.03m^3$$

Cell depth – A deep reservoir on a highly permeable soil can cause soil compaction and loss of permeability from the mass of overlying stone and stored water. Therefore, an additional calculation should be conducted to determine the maximum allowable rain garden cell depth. As per the Low Impact Development Stormwater Management Planning and Design Guide (2010):

$$d_{max} = P * \frac{\left(T - \frac{d_p}{P}\right)}{n * 1000}$$

where:

d_{max}	= maximum rain garden cell depth, m
P	= percolation rate of native soil, mm/hr.
T	= drawdown time, hr
n	= porosity of native soil
d_p	= maximum surface ponding depth, mm

$$d_{max} = 20 * \frac{\left(48 - \frac{200}{20}\right)}{0.4 * 1000} = 1.9m$$

Therefore, the rain garden does not require an underdrain since the chosen total design depth of the rain garden cell does not exceed the maximum allowable limit.

To calculate the bottom area of the rain garden, using Equation 4.3 in the MECP Manual,

$$A = \frac{V}{PnTf} = \frac{V}{dnf}$$

where:

A	= rain garden footprint, m^2
V	= required storage volume, m^3
d	= rain garden design depth, m
f	= longevity factor

$$A = \frac{(1.03m^3)}{(0.7)(0.4)(0.5)} = 7.36m^2$$

A single rain garden will have a stone reservoir depth of 0.7m and a minimum bottom area of $8.0m^2$ ($8m \times 1.0m$) to contain and infiltrate the runoff.

Additional storage of runoff will be provided on the surface of the rain garden as it is designed with a shallow depression of 0.2m.

MIDDLE UNIT RAIN GARDEN

$$Runoff Volume = C * i * A$$

$$Runoff Volume = (0.95)(0.02m)(38m^2) = 0.722m^3 \sim 0.72m^3$$

$$A = \frac{V}{PnTf} = \frac{V}{dnf}$$

$$A = \frac{(0.72m^3)}{(0.7)(0.4)(0.5)} = 5.15m^2$$

A single rain garden will have a stone reservoir depth of 0.7m and a minimum bottom area of $5.5m^2$ ($5.5m \times 1.0m$) to contain and infiltrate the runoff.

Additional storage of runoff will be provided on the surface of the rain garden as it is designed with a shallow depression of 0.2m.

Development Export Summary

Development :829 Essa Road

Updated : Sept 2014

Pre-Development Phosphorus Export

DEVELOPMENT : 829 Essa Road			
Landuse	Area (ha)	P coeff (kg/ha)	Pload (kg/yr)
Natural Heritage			
Forest	0.05	0.06	
Low Intensity Residential	0.12	0.13	0.02
Natural Heritage Land use Class Total :		0.17	0.02
Development Total :		0.17	0.02

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Updated : Sept 2014

Cropland Site Sediment & Phosphorus Pre-Development Export

DEVELOPMENT : 829 Essa Road			
COLOUR KEY : Site Specific Input Constant / Lookup Calculation			
SubArea :			
Slope Area (ha)	R (rainfall / runoff for Lake Simcoe)		
Surface Slope Gradient (%)	K (soil erodability factor)		
Length of Slope (m)	NN (determined by slope)		
Cropt Type Factor)	LS (slope length gradient factor)		
Tillage Type Factor	C (crop management factor)		
	P (prevention + capture)		
	Soil Loss (kg/year)		
	Phosphorus export (kg/ha/yr)		
	Phosphorus load (kg/yr)		

PRE Developed Area (ha) :

Phosphorus export (kg/ha/yr) :

Phosphorus load (kg/yr) :

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Post-Development Phosphorus Export

DEVELOPMENT : 829 Essa Road

Landuse	Area (ha)	P coeff (kg/ha)	Pload (kg/yr)
Natural Heritage			
Low Intensity Residential	0.17	0.13	0.02
Natural Heritage Land use Class Total :		0.17	0.02
Development Total :		0.17	0.02

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Updated : Sept 2014

Cropland Site Sediment & Phosphorus Post-Development Export

DEVELOPMENT : 829 Essa Road

COLOUR KEY :	Site Specific Input	Constant / Lookup	Calculation
SubArea :			

Slope Area (ha)	R (rainfall / runoff for Lake Simcoe)
Surface Slope Gradient (%)	K (soil erodability factor)
Length of Slope (m)	NN (determined by slope)
Cropt Type Factor)	LS (slope length gradient factor)
Tillage Type Factor	C (crop management factor)
	P (prevention + capture)
	Soil Loss (kg/year)
	Phosphorus export (kg/ha/yr)
	Phosphorus load (kg/yr)

PRE Developed Area (ha) :
 Phosphorus export (kg/ha/yr) :
 Phosphorus load (kg/yr) :

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Post Dev BMP

Area (ha)	Treated Area %	P coefficient	P coefficient	P Load Reduction (kg/yr)	Rationale
Best Management Practices (BMP) Applied (and Rationale)					
Low Intensity Residential					
0.10	100	0.13	100 %	0.01	Catchment 202: Rain Gardens
Bioretention Systems					
Low Intensity Residential					
0.08		0.13			
User Entry					

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Development Area P and BMP Summary

Total PreDevelopment Area (ha):	0.17
PreDevelopment Area excluding Wetlands (ha):	0.17
Total PostDevelopment Area (ha):	0.17
Total Area treated by BMP's (ha):	0.10
Treated Area total:	0.18
Total PreDevelopment Load (kg/yr):	0.02
Total PostDevelopment Load (kg/yr):	0.02
Total P Load Reduction with BMP's (kg/yr):	0.01
Minimum P Load Reduction Required:	0.00
Total PostDevelopment Load with BMP's (kg/yr)	0.01
Conclusion :	Net increase in P load, additional reduction is required.

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Post Dev Construction

Sediment Control Measures - Filtration Controls

Storm Drain Inlet Protection

Sediment Control Measures - Perimeter Controls

Sediment/Silt Fence

Vehicle Tracking Control/Mud Mat

4.5 Bioretention

4.5.1 Overview

Description

As a stormwater filter and infiltration practice, bioretention temporarily stores, treats and infiltrates runoff. Depending on native soil infiltration rate and physical constraints, the system may be designed without an underdrain for full infiltration, with an underdrain for partial infiltration, or with an impermeable liner and underdrain for filtration only, which can also be referred to as a biofilter. The primary component of a bioretention practice is the filter bed which is a mixture of sand, fines and organic material. Other important elements of bioretention include a mulch ground cover and plants adapted to the conditions of a stormwater practice. Pretreatment, such as a settling forebay, vegetated filter strip, or stone diaphragm, often precedes the bioretention to remove particles that would otherwise clog the filter bed. Bioretention is designed to capture small storm events or the water quality storage requirement. An overflow or bypass is necessary to pass large storm event flows.

Bioretention can be adapted to fit into many different development contexts and provides a convenient area for snow storage and treatment. In a low density development, it might have a soft edge and gentle slopes, while a high density application might have a hard edge with vertical sides. A number of common forms of bioretention design are illustrated in Figure 4.5.1.

Common Concerns

Bioretention is a popular LID practice as it can meet local stormwater requirements while using space that would be landscaped anyway. However, there are some common concerns that can be addressed during design. These include:

- *Risk of Groundwater Contamination:* Most pollutants in urban runoff are well retained by infiltration practices and soils and therefore, have a low to moderate potential for groundwater contamination (Pitt *et al.*, 1999). Chloride and sodium from de-icing salts applied to roads and parking areas during winter are not well attenuated in soil and can easily travel to shallow groundwater. Infiltration of de-icing salt constituents is also known to increase the mobility of certain heavy metals in soil (e.g., lead, copper and cadmium), thereby raising the potential for elevated concentrations in underlying groundwater (Amrhein *et al.*, 1992; Bauske and Goetz, 1993). However, very few studies that have sampled groundwater below infiltration facilities or roadside ditches receiving de-icing salt laden runoff have found concentrations of heavy metals that exceed drinking water standards (e.g., Howard and Beck, 1993; Granato *et al.*, 1995). To minimize risk of groundwater contamination the following management approaches are recommended (Pitt *et al.*, 1999; TRCA, 2009b):
 - stormwater infiltration practices should not receive runoff from high traffic areas where large amounts of de-icing salts are applied (e.g., busy

- highways), nor from pollution hot spots (e.g., source areas where land uses or activities have the potential to generate highly contaminated runoff such as vehicle fuelling, servicing or demolition areas, outdoor storage or handling areas for hazardous materials and some heavy industry sites);
 - prioritize infiltration of runoff from source areas that are comparatively less contaminated such as roofs, low traffic roads and parking areas; and,
 - apply sedimentation pretreatment practices (e.g., oil and grit separators) before infiltration of road or parking area runoff.
- *Risk of Soil Contamination:* Available evidence from monitoring studies indicates that small distributed stormwater infiltration practices do not contaminate underlying soils, even after more than 10 years of operation (TRCA, 2008).
- *Performance in Winter Conditions and Spring Snowmelt:* Performance studies show that bioretention effectively captures and treats runoff during winter months with average daily temperatures in the -5 to 10 °C range (Traver, 2005; UNHSC, 2005, Roseen *et al.*, 2009). Frost penetration of filter media varied from zero to 17 cm in studies at the University of New Hampshire (Roseen, 2007). Year round monitoring of a bioswale in the Greater Toronto Area showed the facility continued to function during winter, with temperatures in the filter bed remaining above zero at a depth of 50 cm below the surface (TRCA, 2008b). While bioretention frequently accepts runoff containing high chloride concentrations, the dissolved chloride will pass through to the groundwater without treatment. Cold climate adaptation for bioretention designs include extending the filter bed and underdrain pipe below the frost line, oversizing the underdrain to reduce the freezing potential, and selecting salt-tolerant vegetation. Some bioretention design variants, such as stormwater planters and curb extensions, are new to cold climates and have not been monitored in winter conditions. Stormwater planters that are wholly above ground should be given special consideration, as the underdrain and other conveyance structures will be more susceptible to freezing.
- *Vegetation Maintenance:* Vegetation maintenance requirements are similar to those of other landscaped areas. The landscaping design should account for the expected level of maintenance. Formal landscape designs will require more maintenance than naturalized landscaping designs. Bioretention in higher density urban areas will need frequent routine maintenance to remove trash, check for clogging, and maintain vegetation.
- *Standing Water and Mosquitoes:* The maximum allowable surface ponding time is 24 hours after the storm event, which is less than the time required for one mosquito breeding cycle. Maximum ponding depth will be between 150-250 millimetres at the end of a storm, but most water is stored in voids within soil and gravel layers. In high density urban landscapes, it may be desirable to have a shorter ponding time.

Figure 4.5.1 Forms of bioretention

Bioretention Cells can be used in development types with large landscaping areas, parks, parking lot islands, or any areas without tight space constraints. They will have side slopes of 2:1 or shallower. Often, they take inflow as sheet flow, but in some cases, such as parking lots, they may be surrounded by curbs and have concentrated inflow.



Left – York University (Source: TRCA); Right – Riverwood Park, Mississauga, Ontario (Source: CVC)

Rain gardens capture roof, lawn and driveway runoff from low to medium density residential lots in a shallow depression in the front, side, or rear yard of the home depending on the development's drainage pattern. These can be simple gardens constructed by the homeowner as a retrofit, or they can be professionally designed into a residential development and may have an underdrain connected to the main storm drain pipe.



Left and Right - front yard rain gardens that takes runoff from the residential lot and street (Source: City of Maplewood, Minnesota)

Stormwater planters (or foundation planters) are typically used in ultra urban areas adjacent to buildings and in plazas. They differ from traditional landscaping beds by receiving runoff from other surfaces.



(Source: City of Portland, BES)

Extended tree pits (also known as parallel bioretention) are located within the road right of way and take advantage of the landscaped space between the sidewalk and street. They can be designed to take runoff from the sidewalk or street. They are typically designed to be offline, that is when they are full the stormwater will bypass the practice and flow to the downstream street inlet.



Source: left – City of Portland, BES; right – CVC.

Curb extensions are, like extended tree pits, installed in the road right-of-way and can also act as a traffic calming device. In place of an otherwise raised concrete surface, the area is constructed as a depression with vegetation and used for stormwater treatment.



Source: City of Portland, BES

- **On Private Property:** If bioretention practices are installed on private lots, property owners or managers will need to be educated on their routine maintenance needs, understand the long-term maintenance plan, and may be subject to a legally binding maintenance agreement. An incentive program such as a storm sewer user fee based on the area of impervious cover on a property that is directly connected to a storm sewer (*i.e.*, does not first drain to a pervious area or LID practice) could be used to encourage property owners or managers to maintain existing practices. Alternatively, bioretention areas could be located in an expanded road right-of-way or “stormwater easement” so that municipal staff can access the facility in the event it fails to function properly.
- **Foundations and Seepage:** Bioretention facilities should be set back at least 4 metres from building foundations. Stormwater planters located near building foundations will need to have an impermeable liner under the bioretention media or the foundation will need to be waterproofed.

- *Roadway Stability:* Design standards on roadway drainage should be consulted. It may be necessary to provide a barrier to keep water from saturating the road's sub-base.
- *Pedestrian Traffic:* Many bioretention applications are located in areas of high foot traffic. Designers should consider methods to prevent pedestrian traffic through the facility, such as shrub placement, curbing, and protective railings.

Physical Suitability and Constraints

Some of the key constraints and design mitigation strategies for bioretention include:

- *Wellhead Protection:* Facilities receiving road or parking lot runoff should not be located within two (2) year time-of-travel wellhead protection areas.
- *Available Space:* Designers should reserve open areas of about 10 to 20% of the size of the contributing drainage area. These are areas that would be typically set aside for landscaping. More space is required for designs with soft and shallow side slopes than those with hard, vertical edges.
- *Site Topography:* Bioretention is best applied when contributing slopes are between 1 to 5%. Ideally, the proposed treatment area will be located in a natural depression to minimize excavation. The surface of the filter bed should be flat to allow flow to spread out and not concentrate in one area of the practice. However, for linear bioretention practices, such as those along roadways, the longitudinal slope must be considered. A stepped multi-cell design can be used when a flat surface cannot be maintained along the length of a linear bioretention.
- *Available Head:* If an underdrain is used, then 1 to 1.5 metres elevation difference is needed between the inflow point and the downstream storm drain invert. This is generally not a constraint due to the standard depth of storm drains. For bioretention without an underdrain, the design will only require enough elevation difference to move large event flows through the overflow or bypass without generating a backflow or flooding problem.
- *Water Table:* Bioretention should be separated from the seasonally high water table by a minimum of one (1) metre to ensure groundwater does not intersect the filter bed, as this could lead to groundwater contamination or practice failure.
- *Soils:* Bioretention can be located over any soil type, but hydrologic soil group A and B soils are best for achieving water balance benefits. Facilities should be located in portions of the site with the highest native soil infiltration rates. Where infiltration rates are less than 15 mm/hr (hydraulic conductivity less than 1×10^{-6} cm/s) an underdrain is required. Native soil infiltration rate at the proposed facility location and depth should be confirmed through measurement of hydraulic

conductivity under field saturated conditions using the methods described in Appendix C.

- *Drainage Area and Runoff Volume:* Bioretention cells work best for smaller drainage areas, as flow distribution over the filter bed is easier to achieve. Typical drainage areas are between 100 m² to 0.5 hectares. The maximum recommended drainage area to one bioretention facility is approximately 0.8 hectares (Davis *et al.*, 2009). Ideally, bioretention should be used as a source control for small drainage areas and not as an end of pipe control. Typical ratios of impervious drainage area to bioretention cell area range from 5:1 to 15:1.
- *Pollution Hot Spot Runoff:* To protect groundwater from possible contamination, source areas where land uses or human activities have the potential to generate highly contaminated runoff (e.g., vehicle fueling, servicing and demolition areas, outdoor storage and handling areas for hazardous materials and some heavy industry sites) should not be treated by bioretention facilities designed for full or partial infiltration. Facilities designed with an impermeable liner (filtration only facilities) can be used to treat runoff from pollution hot spots.
- *Proximity to Underground Utilities:* Designers should consult local utility design guidance for the horizontal and vertical clearances required between storm drains, ditches, and surface water bodies. It is feasible for on-site utilities to cross linear bioretention; however, this may require design of special protection for the utility. For road right-of-way applications, care should be taken to provide utility-specific horizontal and vertical offsets. However, conflicts with water and sewer laterals (house connections) may be unavoidable. If so, revisit the off-sets with the utility company, and sequence construction to avoid impacts to services.
- *Overhead Wires:* Designers should also check whether maximum future tree canopy height in the bioretention area will not interfere with existing overhead phone and power lines.
- *Setbacks from Buildings:* If an impermeable liner is used, no setback is needed. If not, a four (4) metre setback from buildings should be applied.

Typical Performance

Bioretention is suited to meet both water quality and water balance objectives. It may also be used in a treatment train with traditional detention practices that meet the regional event peak discharge requirements. The ability of bioretention to meet the stormwater management objectives is shown in Table 4.5.1.

Table 4.5.1 Ability of bioretention to meet SWM objectives

BMP	Water Balance Benefit	Water Quality Improvement	Stream Channel Erosion Control Benefits
Bioretention with no underdrain	Yes	Yes – size for water quality storage requirement	Partial – based on available storage volume and infiltration rates
Bioretention with underdrain	Partial – based on available storage volume beneath the underdrain and soil infiltration rate	Yes – size for water quality storage requirement	Partial – based on available storage volume beneath the underdrain and soil infiltration rate
Bioretention with underdrain and impermeable liner	Partial – some volume reduction through evapotranspiration	Yes – size for water quality storage requirement	Partial – some volume reduction through evapotranspiration

Water Balance

Bioretention has been shown to reduce runoff volume through evapotranspiration and infiltration of runoff. The research can be classified into bioretention applications that include underdrains and those that do not (and therefore rely on full infiltration into underlying soils). Aside from the underdrain, many other factors can impact the water balance such as the native soil infiltration rate, rainfall patterns, and sizing criteria. Table 4.5.2 presents the runoff reduction results from various bioretention studies, each with their own set of environmental contexts and design factors influencing the results.

Table 4.5.2 Volumetric runoff reduction¹ achieved by bioretention

LID Practice	Location	% Runoff Reduction ¹	Reference
Bioretention without underdrain	Connecticut	99%	Dietz and Clausen (2005)
	Pennsylvania	80%	Ermilio (2005)
	Pennsylvania	70%	Emerson and Traver (2004)
Bioretention with underdrain	North Carolina	40 to 60%	Smith and Hunt (2007)
	North Carolina	33 to 50%	Hunt and Lord (2006)
	Maryland and North Carolina	20 to 50%	Li <i>et al.</i> (2009)
Runoff Reduction Estimate²		85% without underdrain 45% with underdrain	

Notes:

1. Runoff reduction estimates are based on differences in runoff volume between the practice and a conventional impervious surface over the period of monitoring.
2. This estimate is provided only for the purpose of initial screening of LID practices suitable for achieving stormwater management objectives and targets. Performance of individual facilities will vary depending on site specific contexts and facility design parameters and should be estimated as part of the design process and submitted with other documentation for review by the approval authority.

Water Quality - Pollutant Removal Capacity

Performance results from both laboratory and field studies indicate that bioretention systems have the potential to be one of the most effective BMPs for pollutant removal (TRCA, 2009b). Bioretention provides effective removal for many pollutants as a result of sedimentation, filtering, soil adsorption, microbial processes and plant uptake. It is also important to note that there is a relationship between the water balance and water quality functions. If a bioretention cell infiltrates and evaporates 100% of the runoff from a site, then there is essentially no pollution leaving the site in surface runoff.

Furthermore, treatment of infiltrated runoff continues to occur as it moves through the native soil. Table 4.5.3 summarizes pollutant removal results from some recent performance studies.

Table 4.5.3 Pollutant removal efficiencies¹ for bioretention (in percent)

Reference	Location	Lead	Copper	Zinc	TSS ²	TP ³	TKN ⁴	PAH ⁵	Bacteria ⁶
Dietz and Clausen (2005)	Haddam, Connecticut	NT	NT	NT	NT	-111	31	NT	NT
Hunt <i>et al.</i> (2006)	Greensboro, North Carolina	81	99	98	-170	-240	-5	NT	NT
Hunt <i>et al.</i> (2006)	Chapel Hill, North Carolina	NT	NT	NT	NT	65	45	NT	NT
Davis, (2007)	College Park, Maryland	88	83	54	59	79	NT	NT	NT
Davis, (2007)	College Park, Maryland	84	77	69	54	77	NT	NT	NT
Muthanna <i>et al.</i> (2007)	Trondheim, Norway	99	89	96	100	NT	NT	NT	NT
Hunt <i>et al.</i> (2008) ⁷	Charlotte, North Carolina	31	54	77	60	31	44	NT	71
Roseen <i>et al.</i> (2009) ⁷	Durham, New Hampshire	NT	NT	95	86	0	NT	NT	NT
Roseen <i>et al.</i> (2009) ⁷	Durham, New Hampshire	NT	NT	80	86	27	NT	NT	NT
Dibiasi <i>et al.</i> (2009)	College Park, Maryland	NT	NT	NT	NT	NT	NT	87	NT

Notes:

NT = not tested

1. Pollutant removal efficiency refers to the pollutant load reduction from the inflow to the outflow (from an underdrain) of the practice, over the period of monitoring unless otherwise noted. Negative values represent net increases in load between the inflow and outflow.
2. Total suspended solids (TSS)
3. Total phosphorus (TP)
4. Total Kjeldahl nitrogen (TKN)
5. Polycyclic aromatic hydrocarbons (PAH)
6. Measured as *E.coli* coliform units (CFU) per 100 mL
7. Values represent efficiency ratios based on differences in average event mean concentrations between the inflow and outflow (from an underdrain) of the practice, over the period of monitoring.

Excellent pollutant removal rates have been observed through field studies for total suspended solids (Roseen *et al.*, 2009), polycyclic aromatic hydrocarbons (TRCA, 2008b; Dibiasi *et al.*, 2009), and metals (Davis *et al.*, 2003; Hunt *et al.*, 2006; Roseen *et al.*, 2009).

al., 2006; Davis, 2007; TRCA, 2008b). Good removal rates for metals have even been observed in bioretention facilities receiving snow melt that contains de-icing salt constituents (Muthanna *et al.*, 2007).

Field investigations of nutrient removal by bioretention facilities have produced more variable results (TRCA, 2009b). Some facilities have been observed to increase total phosphorus in infiltrated water (Dietz and Clausen, 2005; Hunt *et al.*, 2006; TRCA, 2008b). These findings have been attributed to leaching from filter media soil mixtures which contained high phosphorus content. To avoid phosphorus export, the phosphorus content (*i.e.*, Phosphorus Index) of the filter media soil mixture should be examined prior to installation and kept between 10 to 30 ppm (Hunt and Lord, 2006). While moderate reductions in total nitrogen and ammonia nitrogen have been observed in laboratory studies (Davis *et al.*, 2001) and field studies (Dietz and Clausen, 2005), nitrate nitrogen has consistently been observed to be low.

Little data exists on the ability of bioretention to reduce bacteria concentrations, but preliminary laboratory and field study results report good removal rates for fecal coliform bacteria (Rusciano and Obropta, 2005; Hunt *et al.*, 2008; TRCA, 2008b).

Several site-specific conditions and design factors can greatly increase or decrease the median removal rates (Table 4.5.4).

Table 4.5.4 Factors that influence bioretention pollutant removal rates

Factors That Reduce Removal Rates	Factors That Enhance Removal Rates
Filter bed less than 500 mm deep	Filter bed deeper than 750 mm
Filter media P-Index values ≥ 30 ppm ¹	Filter media P-Index values < 30 ppm ¹
Oversized underdrain system	Properly sized (or no) underdrain system
No pretreatment provided	Pretreatment provided
Single bioretention cell	Multiple bioretention cells, including forebay
Parsely landscaped with ground cover only	Densely landscaped with trees, shrubs and ground cover
Filter media comprised predominantly of sand	Filter media comprised of a mixture of sand, fines and organic matter
Filter surface left uncovered or covered with stone	Filter surface covered with mulch and vegetation

Notes:

1. P-index values refers to phosphorus soil test index values in parts per million (ppm). See www.omafra.gov.on.ca for information on soil testing and a list of accredited soil laboratories.

Stream Channel Erosion Control

The feasibility of storing the channel erosion control volume within bioretention areas will be dependent on the size of the drainage area and available space. It may prove infeasible due to the large footprint needed to maintain the recommended maximum ponding depth of 200 mm. Meeting the channel erosion control requirement through bioretention is most feasible in the regions of the Greater Toronto Area with A and B

soils. In these situations, the reduction in runoff volume through infiltration and evapotranspiration may be sufficient. It is important to note that the bioretention practice will infiltrate runoff throughout the course of the storm; so the actual capacity of the bioretention cell to capture runoff from the drainage area will be larger than its designed storage volume.

Other Benefits

The benefits of bioretention reach beyond the specific stormwater management goals to other social and environmental benefits, including:

Reduced thermal aquatic impacts: Bioretention and other filtration and infiltration practices benefit aquatic life by reducing thermal impacts on receiving waters from urban runoff (Jones and Hunt, 2009). Unlike detention ponds, bioretention does not raise water temperature and can help maintain baseflows through infiltration.

Snow Storage: Bioretention areas can be used for snow storage and snow melt treatment from the contributing drainage area during winter, especially those located adjacent to parking lots and roadways. To function as snow storage, bioretention must include an overflow for snow melt in excess of the designed ponding depth. Additionally, the plant material must be salt-tolerant, perennial and tolerant of periodic inundation.

Reduced Urban Heat Island: Bioretention is able to reduce the local urban heat island by introducing soils and vegetation into urban areas, such as parking lots. Vegetation absorbs less solar radiation than hard urban surfaces. Also, the water vapor emitted by plant material also cools ambient temperatures.

4.5.2 Design Template

Applications

Bioretention can be used wherever water can be conveyed to a landscaped area. Facilities have been installed at commercial, institutional, and residential sites in spaces that are traditionally pervious and landscaped. Bioretention facilities are installed close to the impervious area that generates the runoff. Typical locations are in and around parking lots, in traffic islands and near building roof leaders. Bioretention planters, extended tree pits, and curb extension are able to fit into ultra-urban development contexts. Typical locations for each bioretention design variant are illustrated in Figure 4.5.2.

Figure 4.5.2 Example applications of bioretention

Bioretention Cells	
Landscaped islands in parking lots: Parking islands can be used to both improve parking lot aesthetics and treat lot runoff. The parking lot grading is designed for sheet flow towards linear landscaping areas between rows of spaces. A curb-less edge or curb cuts are used to convey water into the depressed landscaped area. (Source: CWP)	
Parking lot edges: Small parking lots can be graded so that flows reach a curb-less edge or curb cut before reaching catchbasins or inlets. The turf at the edge of the parking lot is used as filter strip pretreatment and the depression for bioretention is located in the pervious area adjacent to the parking lot. (Source: CWP).	
Rights-of-way, traffic islands, and medians: Landscaped or unused space within the right-of-way can be turned into bioretention for treating road runoff. The road cross section can be designed to slope towards the center median or traffic islands rather than the outer edge. A linear configuration can be used to receive sheet flow from the roadway or a grass channel or pipe may convey flows to the bioretention. (Source: Seattle Public Utilities)	
Roundabouts, cul-de-sacs, and entrance loops: The road cross section is designed to slope towards the center island. A curb-less edge or curb cuts are used. (Source: CWP)	

Pervious areas between buildings and sidewalks:

Landscaping around buildings and between buildings and sidewalks can be turned into multi-functional spaces with bioretention. Roof leaders, sidewalks and other impervious areas around the building can be directed to these practices. Densely vegetated practices can also provide some urban heat island cooling to the site. (Source: CWP)



Courtyards: Runoff collected in a storm drain system or roof leaders can be directed to bioretention in courtyards. (Source: City of Portland, BES)



Rain Garden

Rain gardens capture roof, lawn, and driveway runoff from lots in a shallow depression. These can be simple gardens constructed as a retrofit, or professionally designed and may have an underdrain. They are designed to capture runoff from small drainage areas, typically less than 1000 square metres.



Left – Single family home rain garden (Source: City of Maplewood, MN); Right – commercial development rain garden (Source: City of Burnsville, MN).

Stormwater Planters

Stormwater planters generally receive runoff from adjacent rooftop downspouts. They can also be used to establish a pervious area within the hardscape of a plaza, courtyard, pedestrian zone, or streetscape. While they treat a very small drainage area, a significant portion of rooftop and plaza runoff may be captured and treated this way.



Source: Left – City of Portland, BES; Right – CWP

Extended Tree Pits

These facilities are installed in the sidewalk area where tree pits are typically found. Instead of using only the small square pit area, a row of pits is utilized as an enlarged planting area. Stormwater from the roadway is diverted into the expanded tree pit using curb cuts or trench drains. If large mature canopy trees are desired, then additional soil volume should be provided in the tree pit.



Sources: Left - City of Portland, BES; Right - Tavella Design Group, Bridgeport, CT.

Stormwater Curb Extensions

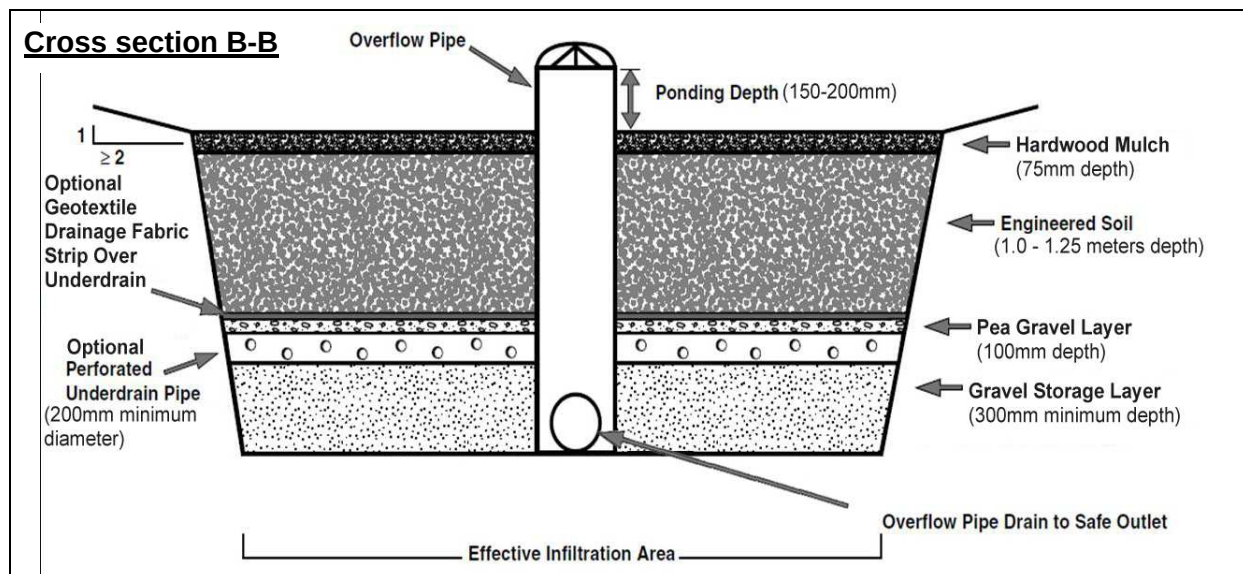
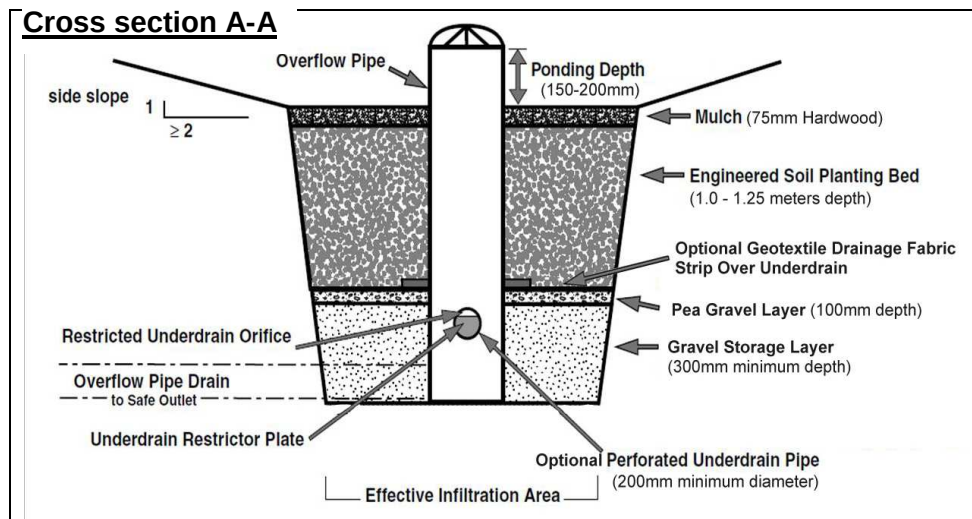
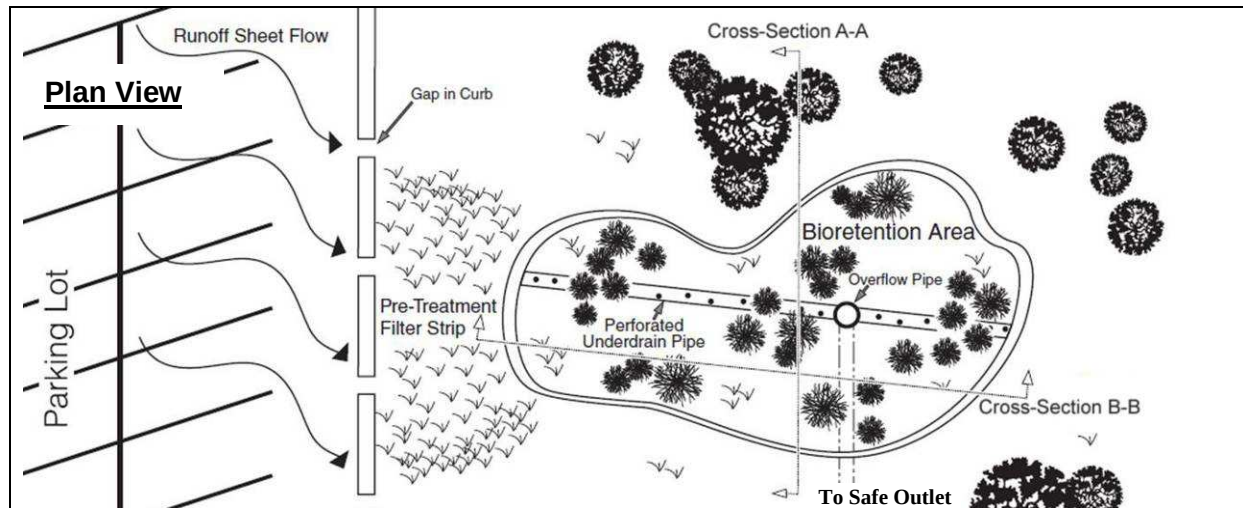
Similar to extended tree pits, these practices are also installed in the public right-of-way. However, curb extensions are typically traffic calming and street parking control device. In its adaptation to a stormwater BMP, the otherwise raised concrete is constructed as a depressed vegetation area and used for stormwater treatment. These practices work well as retrofits to residential neighborhoods.



Source: Left – City of Portland, BES; Middle and Right – CWP

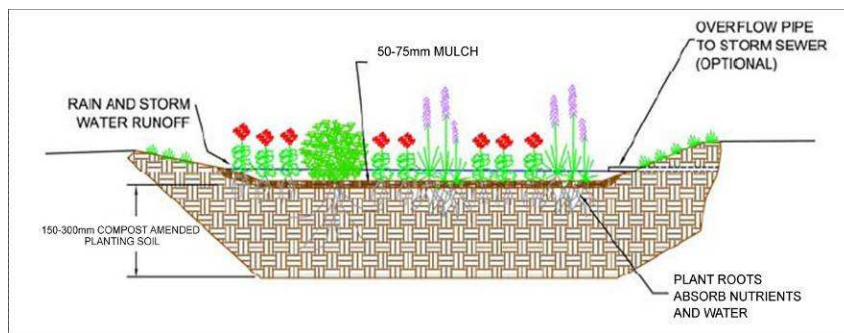
Typical Details

Figure 4.5.3 Plan view and cross sections of a typical bioretention cell



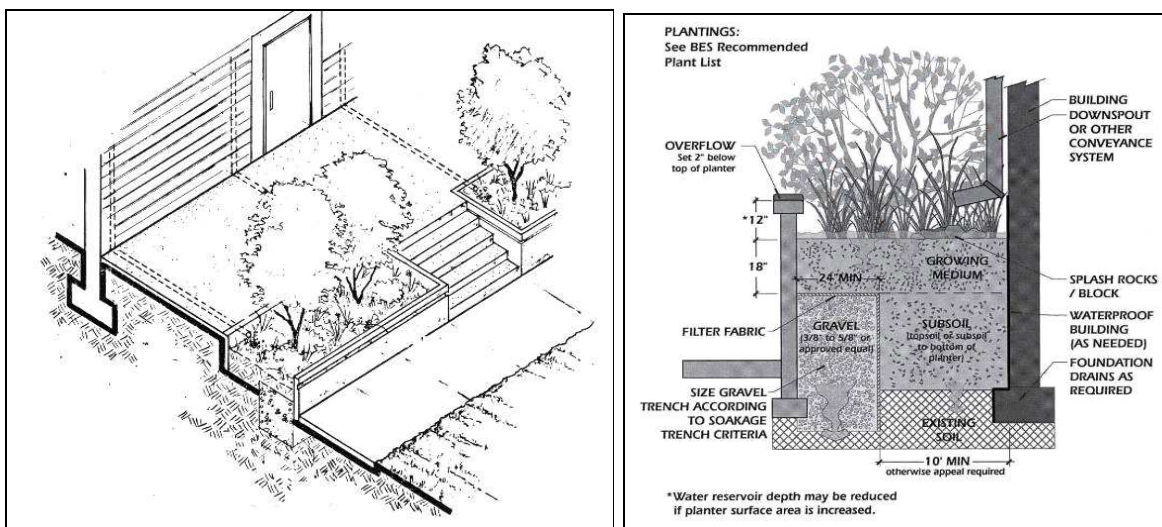
Source: adapted from Wisconsin Department of Natural Resources bioretention details

Figure 4.5.4 Rain garden cross section



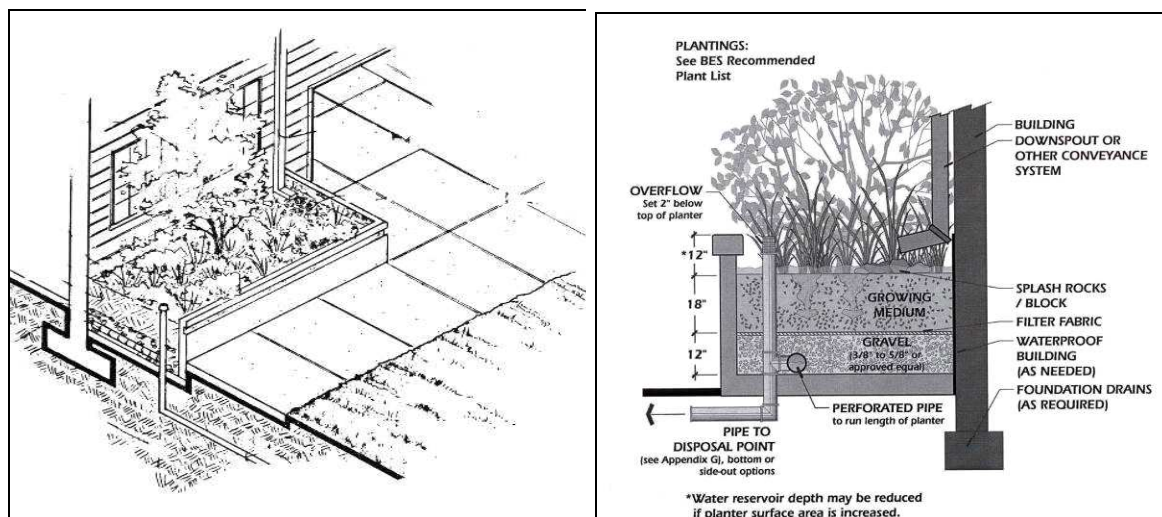
Source: MDE, 2000

Figure 4.5.5 Infiltrating stormwater planter box



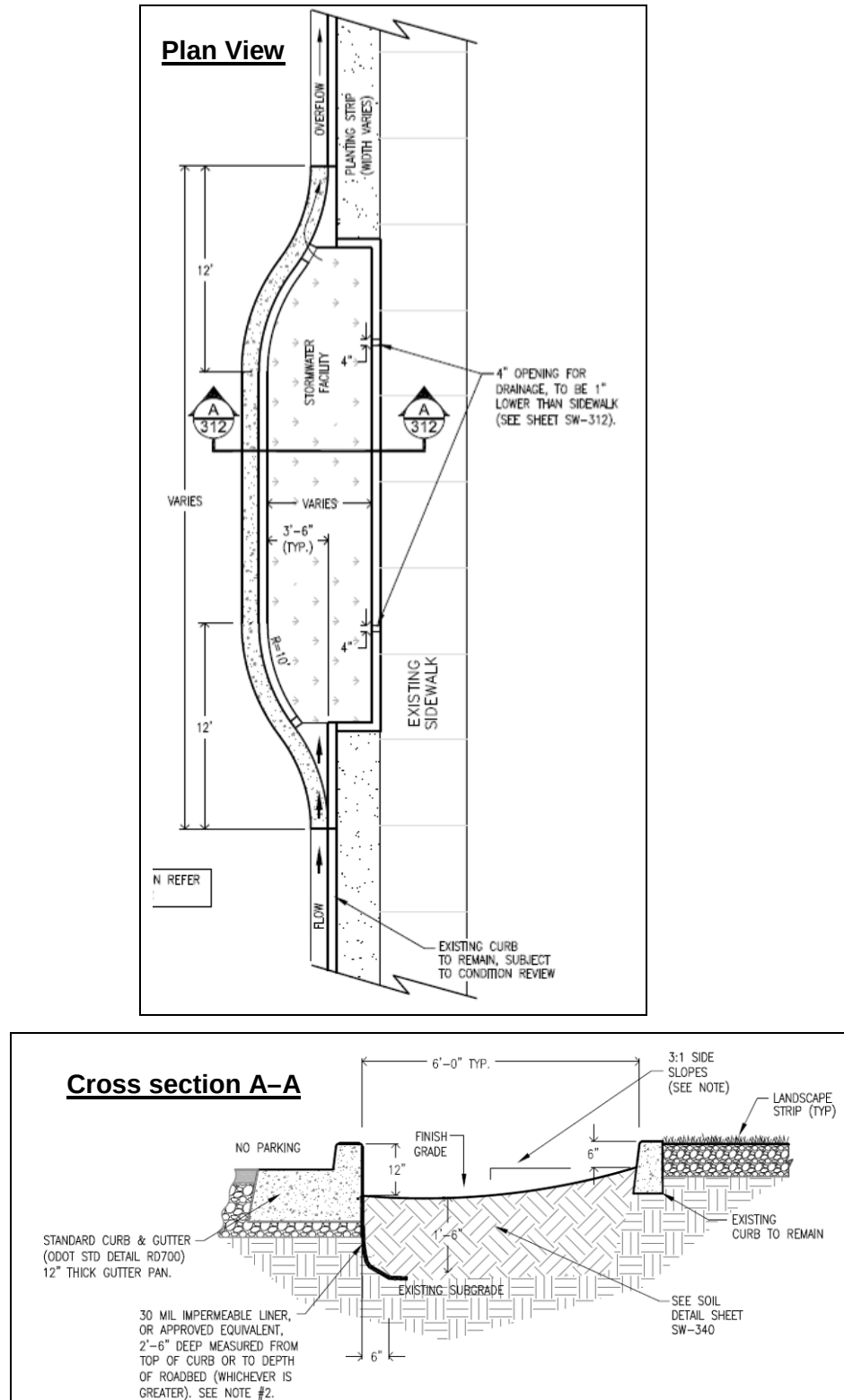
Source: City of Portland, 2004

Figure 4.5.6 Stormwater planter box biofilter (filtration only)



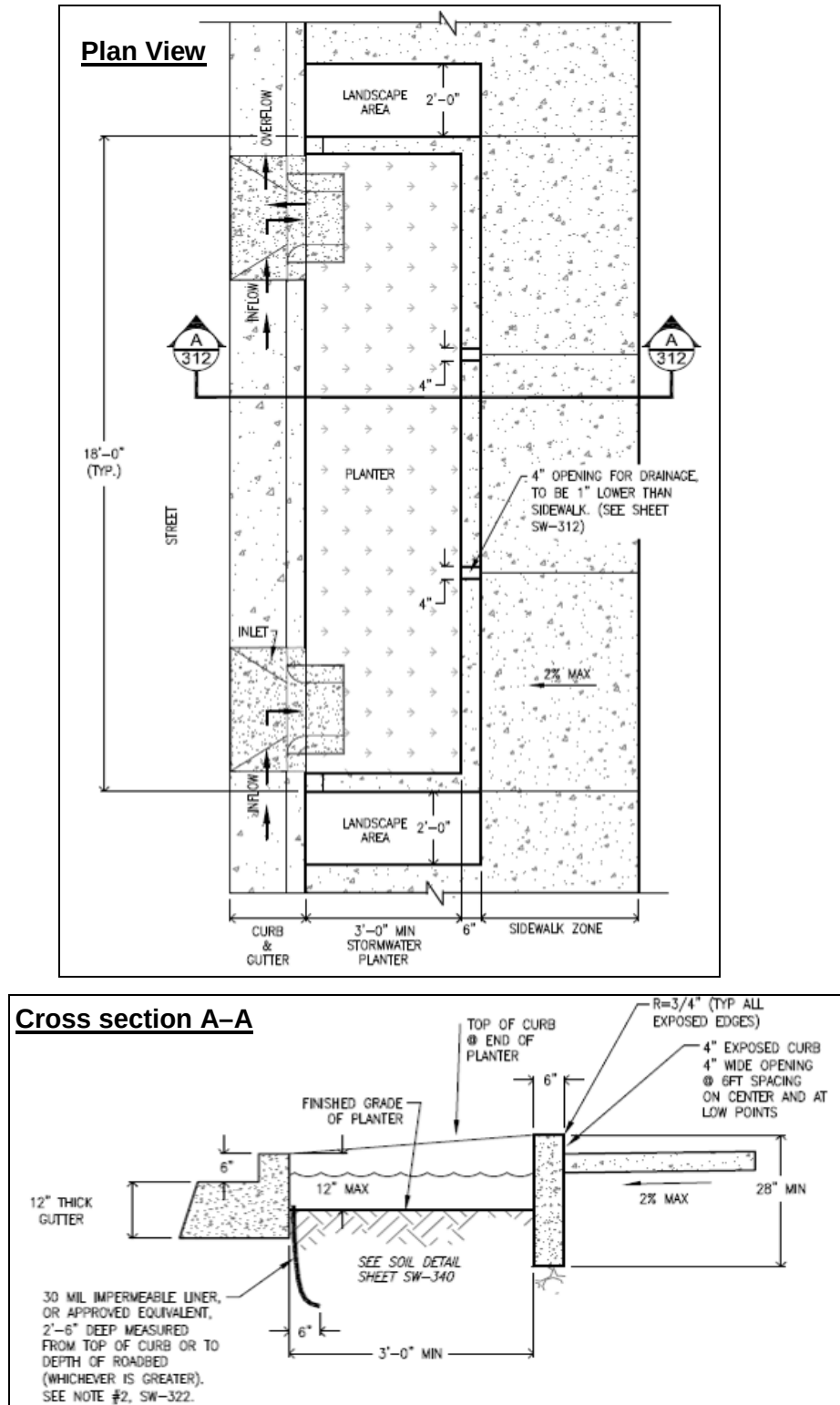
Source: City of Portland, 2004

Figure 4.5.7 Plan view and cross section of a stormwater curb extension



Source: City of Portland, 2004

Figure 4.5.8 Plan view and cross section of an extended tree pit



Source: City of Portland, 2004

Design Guidance

Geometry and Site Layout

There are several key geometry and site layout factors to take into account:

- The minimum footprint of the filter bed area is based on the drainage area. Typical drainage areas to bioretention are between 100 m² to 0.5 hectares. The maximum recommended drainage area to one bioretention facility is approximately 0.8 hectares (Davis *et al.*, 2009). Footprints far in excess of the calculated area are not desirable, as the bioretention plants may not receive adequate water. Undersized bioretention may result in early failure and more frequent overflows.
- If multiple small bioretention practices are planned, such as in landscaped islands of a parking lot or between residential lots, then the sizing and spacing of these need to be considered early in the site planning.
- The geometric design of bioretention will be dictated by other elements of the landscape such as buildings, sidewalks, utility corridors, retaining walls, etc. Bioretention can be configured to fit into many locations and shapes. However, cells that are narrow or have narrow sections may concentrate flow as it spreads throughout the cell and result in erosion.
- The filter bed surface should be level to encourage stormwater to spread out evenly over the surface. Ponding in one location of the bioretention will result in increased sedimentation and clogging at the ponding location and uneven watering of the vegetation.

Pretreatment

Pretreatment prevents premature clogging of bioretention facilities by capturing coarse sediment particles before they reach the filter bed. In some cases, where the drainage areas produce little sediment, such as rooftops, bioretention can function effectively without pretreatment (Heasom *et al.* 2006). A two-cell design that incorporates a forebay is recommended for bioretention with the available space and high sediment load drainage areas. Several pretreatment measures are feasible, depending on the method of conveyance and the drainage area:

- *Two-cell design (channel flow)*: Forebay ponding volume should account for 25% of the water quality storage requirement and be designed with a 2:1 length to width ratio. This pre-treatment device is the most effective and can be designed for easy sediment-removal.
- *Vegetated filter strip (sheet flow)*: Should ideally be a minimum of three (3) metres in width. However, space constraints at some bioretention sites prohibit this width. If smaller strips are used, more frequent maintenance of the filter bed can be anticipated. See Section 4.6 for additional detail about vegetated filter strips.

- *Gravel diaphragm (sheet flow)*: A small trench filled with pea gravel, which is perpendicular to the flow path between the edge of the pavement and the bioretention practice will promote settling out of sediment. It also acts as a level spreader, maintaining sheet flow into the facility. If the contributing drainage area is steep, then larger stone should be used in the diaphragm. A drop of 50-150 mm into the gravel diaphragm can be used to dissipate energy and promote settling.
- *Rip rap and/or dense vegetation (channel flow)*: These energy dissipation techniques are acceptable pretreatment on small bioretention cells with a drainage area of less than 100 square metres.
- *Gutter screens*: Screens are appropriate for pretreatment of runoff from roof leaders.

Conveyance and Overflow

Bioretention can be designed to be inline or offline from the drainage system (Figure 4.5.9). Inline bioretention accepts all of the flow from a drainage area and conveys larger event flows through an overflow outlet. Overflow structures need to be sized to safely convey larger storm events out of the bioretention cell. The invert of the overflow should be placed at the maximum water surface elevation of the bioretention area, which is typically 150-250 millimetres above the surface of the filter bed. The overflow capture device should be scaled to the application – this may be a landscaped grade outlet, stand pipe with trash guard, or a transportation-type yard inlet.

Offline bioretention practices use flow splitters or bypass channels that only allow the required water quality storage volume to enter the facility. This may be achieved with a pipe, weir, or curb opening sized for the target flow, but in conjunction, create a bypass channel so that higher flows do not pass over the surface of the filter bed. Using a weir or curb opening minimizes clogging and reduces the maintenance frequency.

Figure 4.5.9 Examples of inline and offline bioretention



Source: Left – CWP; Right – Low Impact Development Center

The inflow conveyance may take one of the following forms (Figure 4.5.10):

- downspouts to a forebay or stone energy dissipater;
- sheet flow off of a depressed curb;
- One or more curb cuts;
- covered drains that convey flows across sidewalks from the curb or downspouts;
- grates or trench drains that capture runoff from the sidewalk or plaza.

Figure 4.5.10 Examples of inlets to bioretention practices



Clockwise from upper left: pipe with riprap (Source: NC Stormwater Manual); trench drain through curb walk (Source: Biohabitats), Curb and gutter inlet structure to bioretention in highway median; curb cut or depressed curb to parking lot bioretention

Whatever the design, flows should enter the bioretention in a safe and non-erosive manner. Using a river rock channel within large bioretention cells can help evenly distribute flows throughout the filter bed while avoiding erosion of the mulch layer.

All conveyance structures should be designed to prevent clogging by trash or organic matter. In high-litter areas, trash racks at the inlet are a possible solution. A trash rack installed in the pretreatment cell can limit the area requiring frequent clean-out.

Artistic Design Elements

Bioretention gives stormwater engineers and urban landscape architects the chance to merge their creative efforts. Functional stormwater treatment can be combined with art when incoming stormwater cascades over waterfalls, turns water wheels, swishes through chutes, or rings rain chimes.

Monitoring Wells

A capped vertical stand pipe consisting of an anchored 100 to 150 millimetre diameter perforated pipe with a lockable cap installed to the bottom of the facility is recommended for monitoring the length of time required to fully drain the facility between storms.

Gravel Storage Layer

- *Depth:* Should be a minimum of 300 mm deep and sized to provide the required storage volume. Granular material should be 50 mm diameter clear stone.
- *Pea gravel choking layer:* A 100 mm deep layer of pea gravel (3 to 10 mm diameter clear stone) should be placed on top of the coarse gravel storage layer as a choking layer separating it from the overlying filter media bed.

Filter Media

- *Composition:* The recommended bioretention filter media soil mixture is:

Component	Percent by Weight
Sand (2.0 to 0.050 mm dia.)	85 to 88 %
Fines (< 0.050 mm dia.)	8 to 12 %
Organic matter	3 to 5 %

To ensure a consistent and homogeneous bed, filter media should come pre-mixed from an approved vendor. The filter media soil mixture should have the following properties:

- o The recommended Phosphorus soil test (P- index) value is between 10 to 30 ppm (Hunt and Lord, 2006). Visit the Ontario Ministry of Agriculture, Food, and Rural Affairs website (www.omafra.gov.on.ca) for information on soil testing and a list of accredited soil laboratories.
- o Soils with cationic exchange capacity (CEC) exceeding 10 milliequivalents per 100 grams (meq/100 g) are preferred for pollutant removal (Hunt and Lord, 2006).
- o The mixture should be free of stones, stumps, roots, or other similar objects larger than 50 mm.
- o For optimal plant growth, the recommended pH is between 5.5 to 7.5. Lime can be used to raise the pH, or iron sulphate plus sulphur can be used to lower the pH. The lime and iron sulphate need to be uniformly mixed into the soil (Low Impact Development Center, 2003a).
- o The media should have an infiltration rate of greater than 25 mm/hr.

One adaptation is to design the media as a sand filter with organic content only at the top. Leaf compost tilled into the top layers will provide organic content for the plants. If grass is the only vegetation, the ratio of compost may be reduced (Hirschman, 2008; Smith and Hunt, 2007).

- *Depth:* The recommended filter bed depth is between 1 and 1.25 metres. However, in constrained applications, pollutant removal benefits may be

achieved in filter beds as shallow as 500 millimetres. (Davis *et al.*, 2009; and Hunt *et al.*, 2006). If trees are included in the bioretention design, then the filter bed depth must be at least 1 metre and have soil volume to accommodate the root structure of mature trees. A minimum of 12 cubic metres of shared root space is recommended for healthy canopy trees. Use perennials, shrubs or grasses instead of trees when landscaping shallower filter beds.

- **Mulch:** A 75 millimetre layer of mulch on the surface of the filter bed enhances plant survival, suppresses weed growth, and pre-treats runoff before it reaches the filter bed. Shredded hardwood bark mulch makes a very good surface cover, as it retains a significant amount of nitrogen and typically will not float away. The mulch layer also plays a key role in the removal of heavy metals, sediment, and nutrients (Davis *et al.*, 2001; Davis *et al.*, 2003; Davis *et al.*, 2006; Dietz and Clausen, 2006; Hunt, 2003; and Hsieh and Davis, 2005).

Underdrain

- Only needed where native soil infiltration rate is less than 15 mm/hr (hydraulic conductivity of less than 1×10^{-6} cm/s).
- Should consist of a perforated pipe embedded in the coarse gravel storage layer at least 100 mm above the bottom of the gravel storage layer.
- HDPE or equivalent material perforated pipes with smooth interior walls should be used. Pipes should be over-sized to accommodate freezing conditions. A minimum 200 mm diameter underdrain is recommended for this reason (MPCA, 2005). Underdrains should be capped on the upstream end(s).
- A strip of geotextile filter fabric placed between the filter media and pea gravel choking layer over the perforated pipe is optional to help prevent fine soil particles from entering the underdrain. Table 4.5.7 provides further detail regarding geotextile specifications.
- A vertical standpipe connected to the underdrain can be used as a cleanout and monitoring well.

Landscaping

Landscaping is critical to the function and appearance of bioretention and will determine the level of maintenance. Some of the factors that will drive landscaping choices are listed below:

- Bioretention cells can be formal gardens or naturalized landscaping.
- Where possible, a combination of native trees, shrubs, and perennial herbaceous materials should be used.
- A planting mix with evergreen and woody plants will provide appealing textures and colors year round, but they may not be appropriate for snow storage areas.
- In areas where less maintenance will be provided and where trash accumulation in shrubbery or herbaceous plants is a concern, consider a “turf and trees” landscaping model.
- If trees are to be used, or the bioretention is located in a shaded location, then ensure that the chosen herbaceous plants are shade tolerant.
- Spaces for herbaceous flowering plants can be included. This may be attractive at a community entrance location or in a residential rain garden.

- Snow storage areas in bioretention should be vegetated with salt-tolerant, herbaceous perennials. Tree and shrub locations cannot conflict with plowing and piling of snow into storage areas.
- Snow melt from roads, parking lots, driveways, or sidewalks will have high chloride levels, so designers should only select salt-tolerant species.
- “Wet footed” plants, such as wetland forbs, should be planted near the center, whereas upland species are better for the edges of the bioretention area.

A complete list of landscape design considerations and a list of plants suitable for bioretention is provided in Appendix B.

Other Details

In urban settings, the trash load and pedestrian traffic call for special consideration. Consider using the following adaptations:

- To protect the vegetation and prevent soil compaction, fencing (low, wrought iron fences), low walls, bollards and chains, curbs, and constructed walkways can be incorporated. These will also serve as a protective barrier to pedestrians from the sometimes steep drop off from the pavement to the depressed bioretention practice.
- Trash racks can be installed between the pre-treatment cell and the main filter bed. This will allow trash to be collected from one location.
- A trash rack can be placed across curb cuts. While this trash rack may clog occasionally, it keeps trash in the gutter to be picked up by street sweeping equipment.
- For maintenance access, a pre-treatment area can be placed above ground or a manhole or grate cover directly over the pre-treatment area can be used.
- Educational signage can be incorporated into the designs.
- Landscaping stone, river rock, or boulders can be used to protect structures or discourage traffic through the practice.
- Log or stone check dams can be used to slow flow and catch litter.

Other Design Resources

Many stormwater manuals provide useful design guidance for bioretention, including:

- City of Toronto’s Design Guidelines for ‘Greening’ Surface Parking Lots include guidelines for the use of biofilters to treat runoff from parking lots.
http://www.toronto.ca/planning/urbdesign/greening_parking_lots.htm
- Lake County, OH Bioretention Guidance Manual
<http://www2.lakecountyohio.org/smd/Forms.htm>
- Portland, OR Stormwater Management Manual
<http://www.portlandonline.com/bes/index.cfm?c=dfbcc>
- Stormwater Source Control Design Guidelines 2005, Greater Vancouver Regional District http://www.gvrd.bc.ca/sewerage/stormwater_reports.htm

- Urban Watershed Forestry Manual Part 2: Conserving and Planting Trees at Development Sites <http://www.cwp.org/forestry/index.htm>
- Wisconsin Stormwater Management Technical Standards <http://www.dnr.state.wi.us/runoff/stormwater/techstds.htm>

BMP Sizing

The depth of a bioretention cell designed for full infiltration (i.e., no underdrain) is dependent on the native soil infiltration rate, porosity (void space ratio) of the filter bed and gravel storage layer media (i.e., aggregate material used in the stone reservoir) and the targeted time period to achieve complete drainage between storm events.

Assuming a void space ratio of 0.4 for both the filter bed and gravel storage layer media, the maximum allowable depth of the cell can be calculated using the following equation:

$$d_{c \max} = i * (t_s - d_p / i) / V_r$$

Where:

- $d_{c \max}$ = Maximum bioretention cell depth (mm)
- i = Infiltration rate for native soils (mm/hr)
- V_r = Void space ratio for filter bed and gravel storage layer (assume 0.4)
- t_s = Time to drain (design for 48 hour time to drain is recommended)
- d_p = Maximum surface ponding depth (mm)

For designs that include an underdrain, the filter media bed should be 1 to 1.25 metres in depth. The following equation can be used to determine the maximum depth of the stone reservoir below the invert of the underdrain pipe:

$$d_{r \max} = i * t_s / V_r$$

Where:

- $d_{r \max}$ = Maximum depth of stone reservoir below the underdrain pipe

The value for native soil infiltration rate (i) used in the above equations should be the design infiltration rate that incorporates a safety correction factor based on the ratio of the mean value at the proposed bottom elevation of the practice to the mean value in the least permeable soil horizon within 1.5 metres of the proposed bottom elevation (see Appendix C, Table C2).

For designs with no underdrain that are located on less permeable soils, a minimum filter bed depth of 0.5 metres is recommended to ensure water quality benefits will be achieved. For designs with filter bed depths less than 1 metre, a maximum surface ponding depth of 85 to 100 mm is recommended.

Once the depth of the bioretention cell is determined the water quality volume, computed using the methods in the relevant CVC and TRCA stormwater management criteria documents (CVC, 2010; TRCA, 2010), can be used to determine the footprint needed using the following equation:

$$A_f = WQV / (d_c * V_r)$$

Where:

- A_f = Footprint surface area (m^2)
- WQV = Water quality volume (m^3)
- d_c = Bioretention cell depth (m)
- V_r = Void space ratio for filter bed and gravel storage layer (assume 0.4)

The ratio of impervious drainage area to footprint surface area of the practice should be between 5:1 and 15:1 to limit the rate of accumulation of fine sediments and thereby prevent clogging.

Design Specifications

Table 4.5.5 Bioretention specifications

Material	Specification	Quantity
Filter Media Composition	Filter Soil Mixtures to contain: ▪ 85 to 88% sand ▪ 8 to 12% soil fines ▪ 3 to 5% organic matter in form of leaf compost Other Criteria: ▪ Phosphorus soil test (P-Index) value 10 to 30 ppm ▪ Cationic exchange capacity (CEC) greater than 10 meq/100 g ▪ pH between 5.5 to 7.5	Recommended depth is between 1.0 and 1.25 metres. Alternative depths may be appropriate in constrained applications. Volumetric computation based on surface area and depth used in design computations.
Mulch Layer	Shredded hardwood bark mulch	A 75 mm layer on the surface of the filter bed.
Geotextile	Material specifications should conform to Ontario Provincial Standard Specification (OPSS) 1860 for Class II geotextile fabrics. Should be woven monofilament or non-woven needle punched fabrics. Woven slit film and non-woven heat bonded fabrics should not be used as they are prone to clogging. Primary considerations are: - Suitable apparent opening size (AOS) for non-woven fabrics, or percent open area (POA) for woven fabrics, to maintain water flow even with sediment and microbial film build-up; - Texture (i.e., grain size distribution) of the overlying native soil, filter media soil or aggregate material; and - Permeability of the native soil. The following geotextile fabric selection criteria are suggested (adapted from AASHTO, 2002; Smith, 2006; and U.S. Dept. of Defense, 2004): <u>Apparent Opening Size (AOS; max. average roll value) or Percent Open Area (POA)</u>	Strip over the perforated pipe underdrain (if present) between the filter media bed and gravel storage layer (stone reservoir)

Material	Specification	Quantity
	For fine grained soils with more than 85% of particles smaller than 0.075 mm (passing a No. 200 sieve): AOS $\leq$ 0.3 mm (non-woven fabrics) For fine grained soils with 50 to 85% of particles smaller than 0.075 mm (passing a No. 200 sieve): AOS $\leq$ 0.3 mm (non-woven fabrics) POA $\geq$ 4% (woven fabrics) For coarser grained soils with 5 to 50% of particles smaller than 0.075 mm (passing a No. 200 sieve): AOS $\leq$ 0.6 mm (non-woven fabrics) POA $\geq$ 4% (woven fabrics) For coarse grained soils with less than 5% of particles smaller than 0.075 mm (passing a No. 200 sieve): AOS $\leq$ 0.6 mm (non-woven fabrics) POA $\geq$ 10% (woven fabrics) <u>Hydraulic Conductivity (k, in cm/sec)</u> k (fabric) > k (soil) <u>Permittivity (in sec⁻¹)</u> Where, Permittivity = k (fabric)/thickness (fabric): For fine grained soils with more than 50% of particles smaller than 0.075 mm (passing a No. 200 sieve), Permittivity should be 0.1 sec⁻¹ For coarser grained soils with 15 to 50% of particles smaller than 0.075 mm (passing a No. 200 sieve), Permittivity should be 0.2 sec⁻¹. For coarse grained soil with less than 15% of particles smaller than 0.075 mm (passing a No. 200 sieve), Permittivity should be 0.5 sec⁻¹.	
Gravel	Washed 50 mm diameter clear stone should be used to surround the underdrain and for the gravel storage layer Washed 3 to 10 mm diameter clear stone should be used for pea gravel choking layer.	Volume based on dimensions, assuming a void space ratio of 0.4.
Underdrain	Perforated HDPE or equivalent, minimum 100 mm diameter, 200 mm recommended.	- Perforated pipe for length of cell. - Non-perforated pipe as needed to connect with storm drain system. - One or more caps. - T's for underdrain configuration.

Construction Considerations

Ideally, bioretention sites should remain outside the limit of disturbance until construction of the bioretention begins to prevent soil compaction by heavy equipment. Bioretention locations should not be used as the site of sediment basins during construction, as the concentration of fines will prevent post-construction infiltration. They should also not be used for storing materials. To prevent sediment from clogging the surface of a bioretention cell, stormwater should be diverted away from the bioretention site until the drainage area is fully stabilized. Due to the locations of many bioretention practices in the road right-of-way or tight urban spaces, considerations of traffic control and utility conflicts must be part of the plans and inspections.

The following is a typical construction sequence to properly install a bioretention practice. The steps may be modified to reflect different bioretention applications or expected site conditions.

1. Bioretention areas should be fully protected by silt fence or construction fencing to prevent compaction by construction traffic and equipment.
2. Installation may only begin after entire contributing drainage area has been either stabilized or flows have been safely routed around the area. The designer should check the boundaries of the contributing drainage area to ensure it conforms to original design.
3. The pretreatment forebay should be excavated first and sealed until full construction is completed.
4. Excavators or backhoes working adjacent to the proposed bioretention area should excavate the cell to the appropriate design depth.
5. It may be necessary to rip the bottom soils to promote greater infiltration or excavate any sediment that may have built up during construction.
6. There are three options at this step depending on the design:
 - a. No infiltration: Place an impermeable liner on the bed of the bioretention area with 150 mm overlap on sides. Lay the perforated underdrain pipe, Pack 50 mm diameter clear stone to 75 mm above top of underdrain, an optional 75 mm choking coarse of pea gravel, and then lay the non-woven geotextile drainage fabric over the stone and underdrain.
 - b. Partial infiltration: Place desired depth of stone for the infiltration volume on bed and then lay the perforated underdrain pipe over it. Pack 50 mm diameter clear stone to 75 mm above the top of the underdrain, an optional 75 mm choking coarse of pea gravel and then lay the non-woven geotextile drainage fabric over the stone and underdrain.
 - c. Full infiltration: Stone can be placed to provide added stormwater volume storage or the bioretention media can be added directly to the bottom of the excavation.
7. Bioretention filter media should be obtained premixed from a vendor. Apply in 300 mm lifts until desired top elevation of bioretention area is achieved. Thoroughly wet each lift before adding the next and wait until water has drained through the soil before adding the next lift. Wait a few days to check for settlement, and add additional media as needed.

8. Prepare planting holes for any trees and shrubs, install vegetation, and water accordingly. Install any temporary irrigation.
9. Plant landscaping materials as shown in the landscaping plan, and water them weekly in the first two months.
10. Lay down surface cover in accordance with the design (mulch, riverstone, or turf).
11. Conduct final construction inspection, checking inlet, pretreatment cell, bioretention cell and outlet elevations.

Construction Inspection

Common construction pitfalls can be avoided by careful construction supervision that focuses on the following aspects:

Erosion and Sediment Control

- Bioretention locations should be blocked from construction traffic and should not be used for erosion and sediment control.
- Proper erosion and sediment controls should be in place for the drainage area.

Materials

- Gravel for the underdrain should be clean and washed; no fines should be present in the material.
- Underdrain pipe material should be perforated and of the correct size.
- A cap should be placed on the upstream (but not the downstream) end of the underdrain.
- Filter media should be tested to confirm that it meets specifications.
- Mulch composition should be correct.

Elevations

Elevations of the following items should be checked for accuracy:

- Depth of the gravel and invert of the underdrain
- Inverts for inflow and outflow points
- Filter depth after media is placed
- Ponding depth provided between the surface of the filter bed and the overflow structure
- Mulch depth

Landscaping and Stabilization

- Correct vegetation should be planted.
- Pretreatment area should be stabilized.
- Drainage area should be stabilized prior to directing water to the bioretention.

The following items should be checked after the first rainfall event, and adjustments should be made as necessary:

- Outfall protection/energy dissipation at concentrated inflow should be stable.
- Flow should not concentrate and should spread evenly over the filter bed.

- Ponded water at the surface of the bioretention facility should drain within 24 hours of the end of the storm event. The filter media bed should fully drain within a maximum period of 72 hours.
- Excessive sediment accumulation should not be present.

4.5.3 Maintenance and Construction Costs

Inspection and Maintenance

Bioretention requires routine inspection and maintenance of the landscaping as well as periodic inspection for less frequent maintenance needs or remedial maintenance. Generally, routine maintenance will be the same as for any other landscaped area, weeding, pruning, and litter removal. Routine operation and maintenance tasks are key to public acceptance of highly visible bioretention units.

Periodic inspections after major storm events will determine whether corrective action is necessary to address gradual deterioration or abnormal conditions. For the first two years following construction the facility should be inspected at least quarterly and after every major storm event (> 25 mm). Subsequently, inspections should be conducted in the spring and fall of each year and after major storm events.

While maintenance can be performed by landscaping contractors who are already providing similar landscape maintenance services on the property, they will need some additional training on bioretention needs. This training should focus on elevation differences needed for ponding, mulching requirements, acceptability of ponding after a rainstorm, and fertilizer requirements. The planting plan should be kept for maintenance records and used to help maintenance staff identify which plants are weeds or invasive.

Aside from homeowner initiated rain garden projects, legally binding maintenance agreements are a necessity for bioretention facilities on private property. Agreements should specify the property owner's responsibilities and the municipality's right to enter the property for inspection or corrective action. Agreements must require regular inspection and maintenance and should refer to an inspection checklist. The construction contract should include a care and replacement warranty to ensure vegetation is properly established and survives during the first growing season following construction.

The expected lifespan of infiltration practices is not well understood, however, it can be expected that it will vary depending on pretreatment practice maintenance frequency, and the sediment texture and load coming from the catchment.

Routine Maintenance and Operation

Routine inspection and maintenance activities as shown in Table 4.5.6 are necessary for the continued operation of bioretention areas.

Table 4.5.6 Suggested routine inspection and maintenance activities for bioretention

Activity	Schedule
Inspect for vegetation density (at least 80% coverage), damage by foot or vehicular traffic, channelization, accumulation of debris, trash and sediment, and structural damage to pretreatment devices.	After every major storm event (>25 mm), quarterly for the first two years, and twice annually thereafter.
Regular watering may be required during the first two years until vegetation is established;	As needed for first two years of operation.
Remove trash and debris from pretreatment devices, the bioretention area surface and inlet and outlets.	At least twice annually. More frequently if desired for aesthetic reasons.
- Remove accumulated sediment from pretreatment devices, inlets and outlets; - Trim trees and shrubs; - Replace dead vegetation, remove invasive growth; - Repair eroded or sparsely vegetated areas; - Remove accumulated sediment on the bioretention area surface when dry and exceeds 25 mm depth (PDEP, 2006); - If gullies are observed along the surface, regrading and revegetating may be required.	Annually or as needed

Annual Inspection and Maintenance

The annual spring cleaning should consist of an inspection and corrective maintenance tasks described in Table 4.5.7

Table 4.5.7 Suggested inspection items and corrective actions for bioretention

Inspection Item	Corrective Actions
Vegetation health, diversity and density	- Remove dead and diseased plants. - Add reinforcement planting to maintain desired vegetation density. - Prune woody matter. - Check soil pH for specific vegetation. - Add mulch to maintain 75 mm layer.
Sediment build up and clogging at inlets	- Remove sand that may accumulate at the inlets or on the filter bed surface following snow melt. - Examine drainage area for bare soil and stabilize. Apply erosion control such as silt fence until the area is stabilized. - Check that pretreatment is properly functioning. For example, inspect grass filter strips for erosion or gullies. Reseed as necessary.
Ponding for more than 48 hours	- Check underdrain for clogging and flush out. - Apply core aeration or deep tilling - Mix amendments into the soil - Remove the top 75 mm of bioretention soil - Replace bioretention soil

Installation and Operation Costs

Due to the wide range in bioretention types and designs, the costs can vary widely. Rain gardens can be very economical if constructed by the homeowner. The costs for a simple rain garden excavated by a homeowner would only include the plants, mulch, and, if necessary, soil amendments. On the other end of the spectrum, stormwater

planters will cost much more per square meter because of the concrete sidewalks, underdrain structure, and professional design costs. The materials used in the construction of bioretention are typical of construction and landscaping projects.

In a study by the Center for Watershed Protection to estimate and compare construction costs for various stormwater BMPs, the median base construction cost for bioretention was estimated to be \$62,765 (2006 USD) per impervious hectare treated with estimates ranging from \$49,175 to \$103,165 (CWP, 2007b). These estimates do not include design and engineering costs, which could range from 5 to 40% of the base construction cost (CWP, 2007b).

4.5.4 References

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FLOW TEST RESULTS



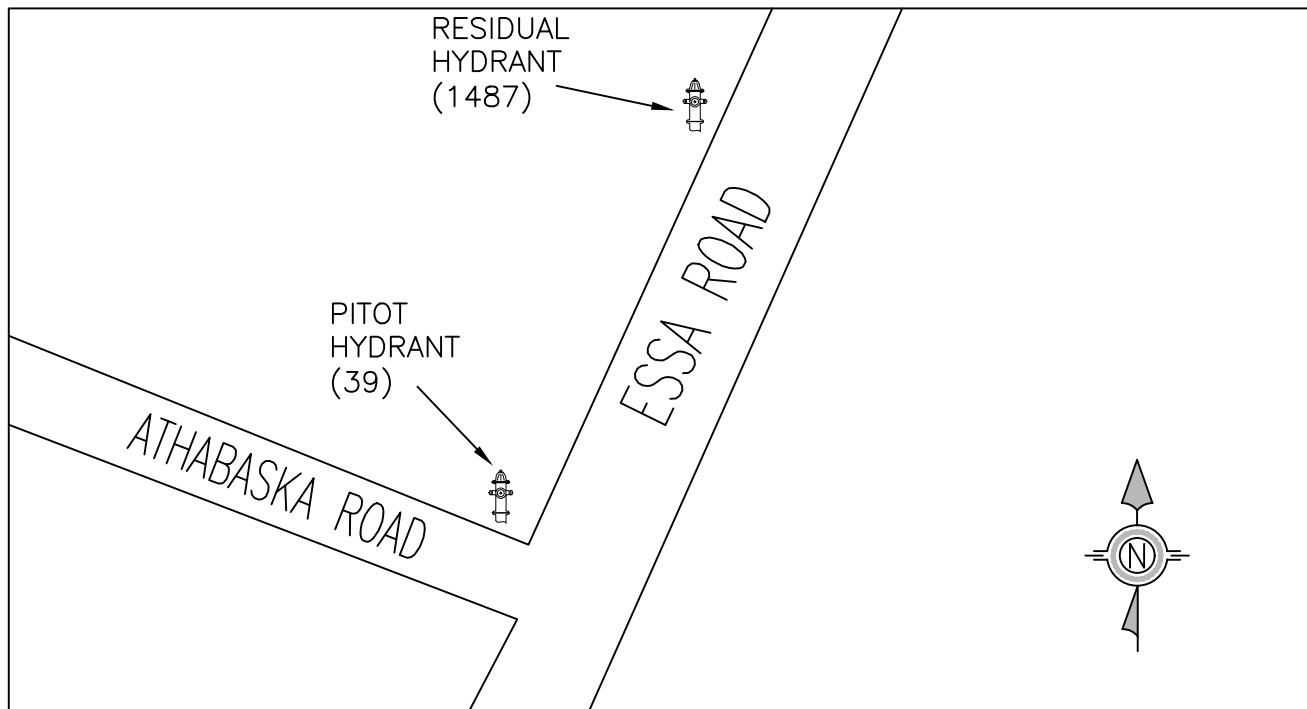
DATE : FRIDAY OCTOBER 23, 2020 TIME : 1:30 PM

LOCATION : ATHABASKA ROAD & ESSA ROAD

BARRIE

ONTARIO

TEST BY : VIPOND FIRE PROTECTION AND LOCAL PUC



STATIC PRESSURE : 61 PSI

UNDERGROUND TYPE & SIZE: 12" PVC

TEST NO.	NO. OF NOZZLES	NOZZLE DIAMETER (INCHES)	DISCHARGE CO-EFFICIENT	RESIDUAL PRESSURE (PSI)	PITOT PRESSURE (PSI)	DISCHARGE (U.S.GPM)
1	1	1-3/4	0.995	58	43	584
2	1	2-1/2	0.9	56	27	876
3	2	2-1/2	0.9	55	22	1582



ATHABASKA ROAD & ESSA ROAD

BARRIE

ONTARIO

BY : GUS A.

OFFICE : BARRIE

TEST BY : VIPOND & PUC

DATE : OCTOBER 23, 2020

STATIC:

61 PSI

RESIDUAL:

TEST#1 58 PSI

TEST#2 56 PSI

TEST#3 55 PSI

FLOW:

@ 584 GPM

@ 876 GPM

@ 1582 GPM

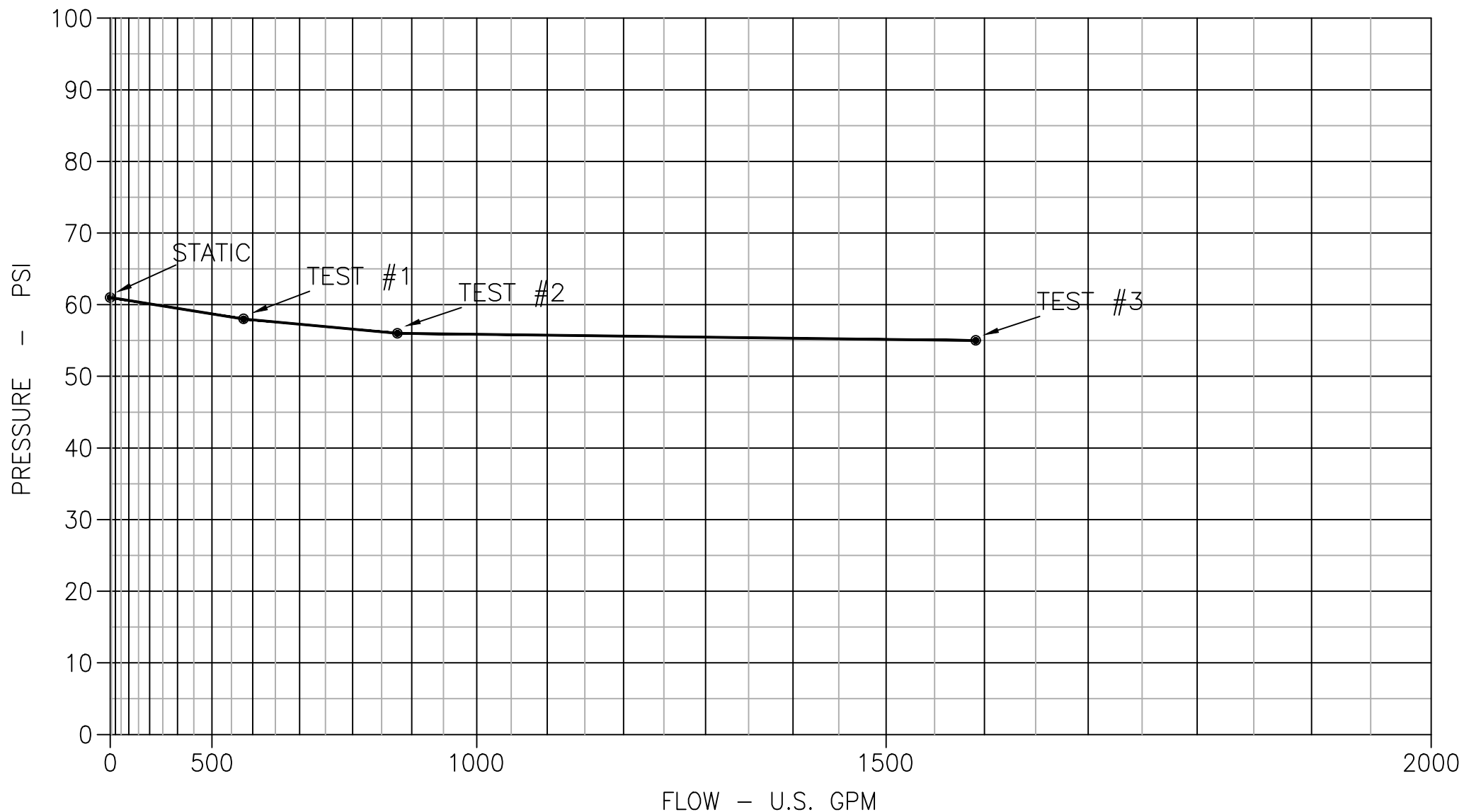


Table 3-5 Maximum Day Factor and Average Daily Demand Per Capita Per Zone

	ZONE 1	ZONE 2N	ZONE 3N	ZONE 2S	ZONE 3S
Maximum Day Factor from 2015 to 2017 ¹	2.30	1.66	2.41	2.32	1.63
Maximum Day Factor for Established Population	2.30	1.80	2.41	2.32	1.80
Maximum Day Factor for Future Population ²					
2021	1.90	1.80	2.00	1.75	1.80
2026	1.80	1.80	2.00	1.75	1.80
2031	1.80	1.75	2.00	1.75	1.80
2036	1.80	1.75	2.00	1.65	1.80
2041	1.80	1.75	2.00	1.65	1.80
2071	1.75	1.75	1.90	1.65	1.75
Average Daily Demand Per Capita (Lpcd) for Established and Future Population ³	174	162	210	129	224

¹ Based on the maximum day factor of each zone found in Table 3-2, Table 3-3 and Table 3-4.

² The maximum day factors for the future population were based on Table 3-1: Peaking factors of *Design Guidelines For-Drinking Water Systems, 2008* from MECP, according to the total equivalent population of each projected year.

³ The average daily demand each zone was calculated based on the average daily demand per capita found in Table 3-2, Table 3-3 and Table 3-4.

The combined population projections, average day demand and maximum day demand projections to ultimate buildout are summarized in Table 3-6. The demands associated with growth in the Salem and Hewitts Secondary Plan Areas have been included in Zones 2S and 3S.

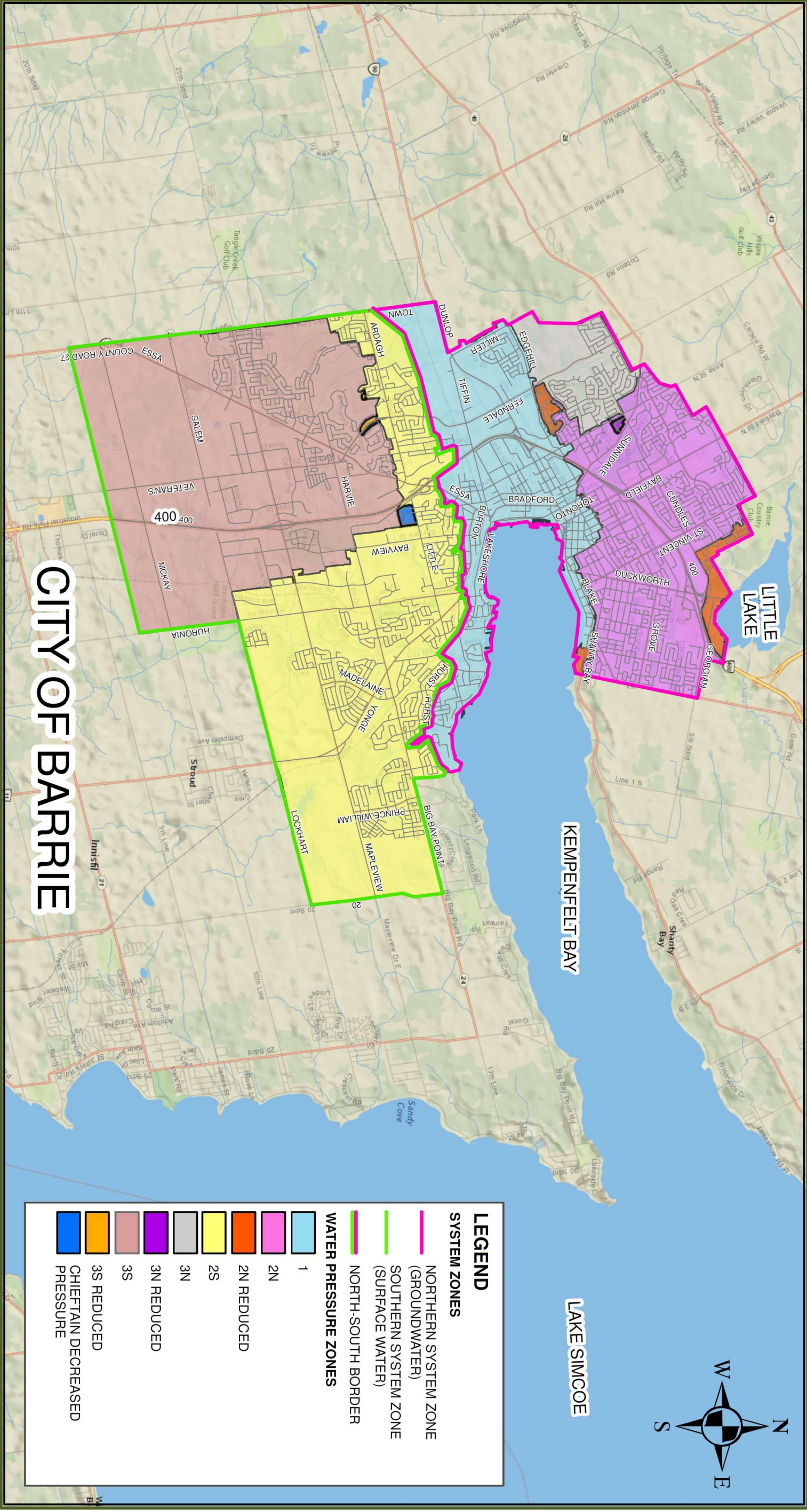


Table 3-1: Peaking Factors

POPULATION	MINIMUM RATE FACTOR (MINIMUM HOUR)	MAXIMUM DAY FACTOR	PEAK RATE FACTOR (PEAK HOUR)
500 - 1 000	0.40	2.75	4.13
1 001 - 2 000	0.45	2.50	3.75
2 001 - 3 000	0.45	2.25	3.38
3 001 - 10 000	0.50	2.00	3.00
10 001 - 25 000	0.60	1.90	2.85
25 001 - 50 000	0.65	1.80	2.70
50 001 - 75 000	0.65	1.75	2.62
75 001 -150 000	0.70	1.65	2.48
greater than 150 000	0.80	1.50	2.25

3.4.3 Commercial and Institutional Water Demands

Institutional and commercial flows should be determined by using historical records, where available. Where no records are available, the values in Table 3.2 should be used. For other commercial and tourist-commercial areas, an allowance of 28 m³/(ha·d) [3000 USgal/(acre·d)] average flow should be used in the absence of reliable flow data.

When using the above unit demands, maximum day and peak rate factors should be developed. For establishments in operation for only a portion of the day such as schools and shopping plazas, the water usage should also be factored accordingly. For instance, with schools operating for 8 hours per day, the water use rate would be at an average rate of 70 L/(student·day) [19 USgal/(student·day)] x 24/8 or 210 L/student (55 USgal/student) over the 8-hour period of operation. The water use will drop to a residual amount during the remainder of the day. Schools generally do not exhibit large maximum day to average day ratios and a factor of 1.5 will generally cover this variation. For estimation of *peak demand* rates, an assessment of the water-using fixtures is generally necessary and a fixture-unit approach should be used.

Fire Flow Calculations – Fire Underwriters Survey 1999

Building Area = 440 m² @ 2 floors = 880 m²

(The building on the west side of the firewall was chosen because it is the worst-case scenario for total floor area and building exposure charges.)

Fire demands for the proposed development were calculated in accordance with the Fire Underwriters Survey (FUS) as follows:

$$F = 220C(A)^{0.5}$$

Where,

F =	the required fire flow in litres per minute.
C =	coefficient related to the type of construction.
A =	total floor area of building (excluding basements) calculated as per FUS

C = 1.0 for ordinary construction

$$F = 220 \times 1.0 \times (880)^{0.5}$$

$$F = 6,526 \text{ L/min}$$

$$F = 108.77 \text{ L/sec}$$

Reductions:

Reduction for low hazard occupancy (-25%)

Fire Flow = 81.58 L/sec

Exposure charge for proposed building to the north (+15%)

Exposure charge for proposed building to the south (+5%)

Exposure charge for existing building to the east (+25%)

Exposure charge for proposed building to the west (+15%)

Total charge = +60% or 48.95 L/sec on the 81.58 L/sec fire flow

Required fire flow as per FUS 1999 calculation = 130.53 L/sec

829 ESSA ROAD TOWNHOUSE DEVELOPMENT
FIRE FLOW ANALYSIS
 Barrie, Ontario



Project Number: 18-11409B
 Date: December 18, 2020
 Design By: DPH
 File: Z:\Project Documents\11409B 829 Essa Road Townhomes\DEC 2020 REVISION\FSR & SWM\Site Fire Flow Analysis.xls

CALCULATION OF RESIDUAL PRESSURE AT BUILDING

1. Boundary Conditions (Based on Fire Flow Test Results):			
	Metric	Imperial	
P0 - Starting Pressure	42.90 m	61 psi	
P1 - Pressure at Q1	38.68 m	55 psi	
Q1 - From Fire Flow Test	5989 L/min	1582 U.S. gal/min	
Q2 - Required Flow	7838 L/min	2071 U.S. gal/min	From FUS
P-loss 1	4.22 m	6 psi	
P-loss 2	6.95 m	10 psi	
P2 - Residual Pressure	35.95 m	51 psi	Extrapolated from Fire Flow Test Results

2. Friction Losses Through Water Service:			
<i>Hazen-Williams Equation</i>	Metric	Imperial	
C _{hw} = Pipe Friction Factor	140	140	
k = conversion factor	10.675	4.727	
n = constant	1.852	1.852	
m = constant	4.8704	4.8704	
Q = Flow	0.130633 m ³ /s	2071 U.S. gal/min	
d = Pipe Diameter	150 mm	5.91 in	
p = Loss/Length	0.2688 m/m	0.1165 psi/ft	
Length	0 m	0 ft	
Loss	0.00 m	0.0 psi	
	0 kPa		

3. Friction Losses Through Apurtenances:						
Apurtenances	Number	K	Velocity	Head Loss	Total Loss	
			<i>m/s</i>	<i>m</i>	<i>m</i>	<i>psi</i>
150mm Tee (run)	0	0.400	7.392	1.114	0.000	0.000
150mm Tee (branch)	0	1.000	7.392	2.785	0.000	0.000
Valve - 150mm dia.	0	0.17	7.392	0.473	0.000	0.000
Elbow - 150mm dia.	0	0.35	7.392	0.975	0.000	0.000
Total Minor Losses					0.000	0.000

4. Elevation - Elevational differences from existing hydrant to proposed building:			
		Metric	Imperial
Elevation at Boundary (i.e. Residual Hydrant):	303.36 m		995 ft
Elevation at Proposed Building:	304.7 m		1000 ft
Elevation Difference = Loss/Gain	1.34 m		1.9 psi

ANALYSIS SUMMARY

Total Losses	1.340 m		
	13.15 kPa	1.9 psi	

Residual Pressure after Losses	34.61 m		
	340 kPa	49.2 psi	PASS

<i>Allowable Residual Pressure</i>	<i>140 kPa</i>	<i>20.3 psi</i>	
------------------------------------	----------------	-----------------	--

- street townhouses ≤ 47 units / hectare @ 2.34 ppu
- 'walk-up' apartments 26 - 53 units / hectare @ 2.34 ppu
- *Typical zoning designations for Medium Density Development may include RM2 and RM2-TH. It should be noted that zoning designations are subject to change and it is the responsibility of the designer to ensure that the most current zoning by-law's and designations are being used.*

- High Density – Apartment dwellings 54 - 300 units / hectare @ 1.67ppu
- *Typical zoning designations for High Density Development may include RA1 and RA2. It should be noted that zoning designations are subject to change and it is the responsibility of the designer to ensure that the most current zoning by-law's and designations are being used.*

The method of determining design population for residential flows will depend upon the particular stage that is being considered and the appropriate detail required for the desired level of design accuracy. Reference shall be made to current zoning policies.

Development Details Basis

When the details regarding the proposed uses on individual lots are known or can be assumed with reasonable certainty, a more detailed approach to design population and associated design flow estimation is required. This approach involves the determination of individual design flows for the various areas in the tributary area which will contribute to an individual sewer reach. The actual number of units shall be used and the design population estimated using the following people per unit (ppu) values.

Low Density	3.13 ppu
Medium Density	2.34 ppu
High Density	1.67 ppu

3.3.1.2 Average Daily Flow

Average daily domestic flow = 225 L/day/person (excluding extraneous flows).

3.3.1.3 Peak Flow

Peak domestic flow is to be calculated using the following formula:

$$Q_p = \frac{P \times q \times M}{86.4} + I \times A$$

- where
- Q_p = Peak residential sanitary sewage flow, including peak extraneous flows (L/s)
 - P = Design population in thousands (see Section 3.3.1.1)
 - q = Average daily domestic flow per capita (L/day/person) (see Section 3.3.1.2)
 - M = Peaking factor (see Section 3.3.1.3)
 - I = Peak extraneous flow (L/s/ha) (see Section 3.3.4)
 - A = Tributary area (ha) (see Section 3.2.2)

The peaking factor, M , is to be calculated using the Harmon Formula:

$$M = 1 + \frac{14}{4 + P^{0.5}}$$

8.2.2 Evaluation of Existing Pumping Stations

The capacity of existing pump stations to convey future flows was also evaluated for the various planning horizons. For this analysis, peak modelled flows corresponding to the 25-year design event were compared against the firm capacity of each station. **Table 8.3** presents the results. The station firm capacity represents the current available capacity at each station while the 2017 Existing, 2021, 2026, 2031, 2036 and 2041 values represent the required capacity in the time period. It is noted that the Huronia PS has already been decommissioned and the Lockhart PS is planned to be decommissioned. In addition, the Holly PS is currently being expanded and the station firm capacity is being increased. The values shown in **Table 8.3** correspond to the post-construction firm capacity.

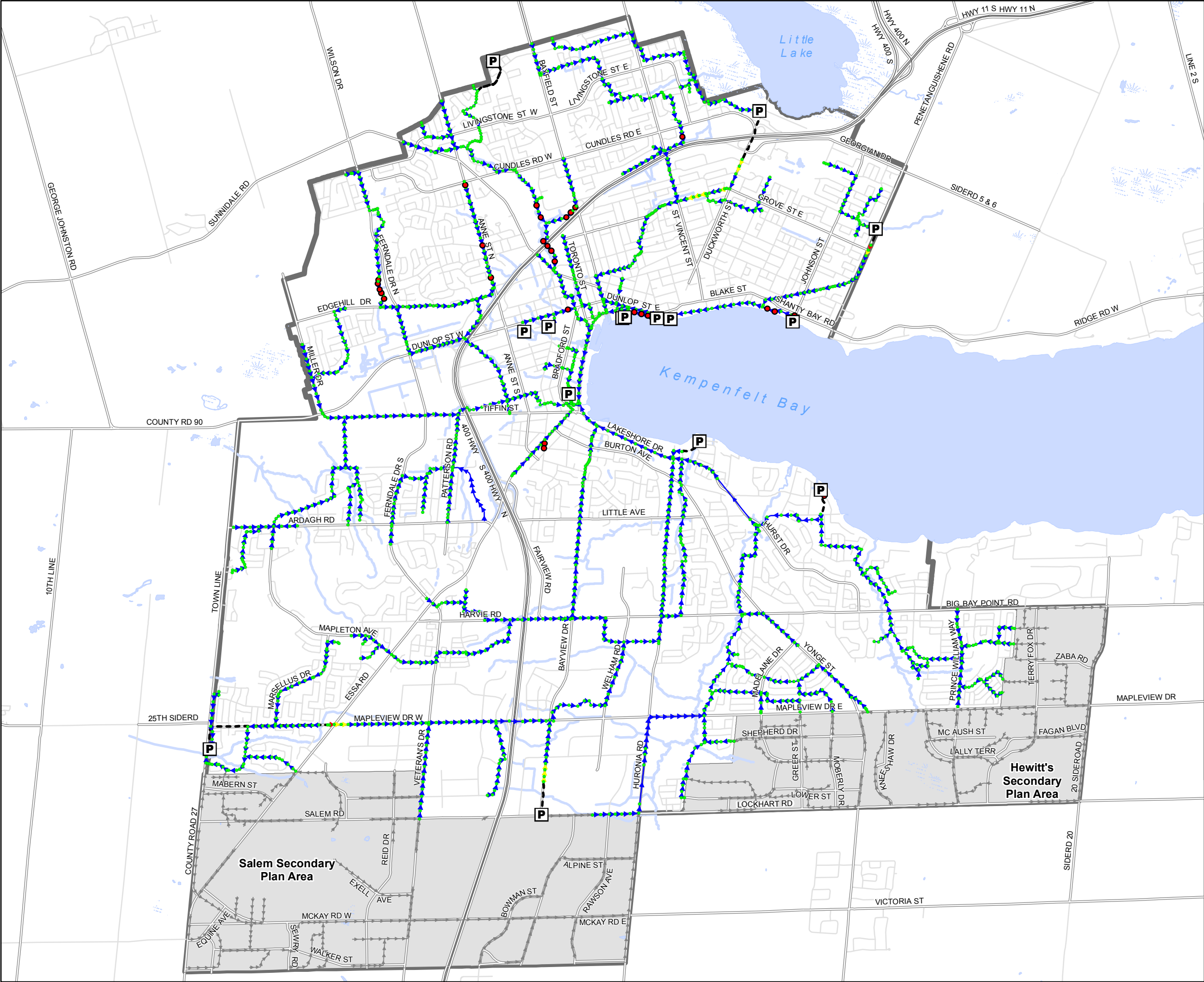
Table 8.3 Pumping Station Capacity Assessment (2017-2041)

Modelled Pumping Station	Station Firm Capacity (L/s)	2017 Existing Peak Flow Entering Station (L/s)	2021 Peak Flow Entering Station (L/s)	2026 Peak Flow Entering Station (L/s)	2031 Peak Flow Entering Station (L/s)	2036 Peak Flow Entering Station (L/s)	2041 Peak Flow Entering Station (L/s)
Barrie WwTF (PS10)	2,205	1,736	1,910	1,963	2,057	2,061	2,109
Grove Street (PS1)	158	53	52	54	54	57	63
Minets Point (PS2)	61.3	22.1	22.2	22.3	22.6	22.9	23.6
Huronia (PS3)	91	Decommissioned					
Lockhart (PS4)	51	5	Planned to Be Decommissioned				
Holly (PS5)	200.9	67	81	95	108	120	130
Little Lake (PS6)	110	37	37	40	40	41	42
Tynedale (PS12)	11	10	15	15	16	16	16
Mooregate (PS18)	21	8	8.5	8.5	8.7	8.9	9.1

It is noted that the influent flow to PS10 does not include any recycle streams. For PS10, the Wastewater Treatment Master Plan contains a detailed examination of future needs.

The results presented above indicated that all SPSs, with the exception of Tynedale SPS (PS12), have sufficient firm capacity to convey peak flows associated with the 25-year design event through to the year 2041. PS12 will require a pump capacity increase in the planning phase 2021.

Figure 6-2
City of Barrie
Existing Wastewater
Collection System Performance
Peak Dry Weather Flow
Conditions



- P** Pumping Stations
- Secondary Plan Boundaries
- City of Barrie Limits
- Forcemain
- Future Annexed Lands Sewers
- Freeboard (m)**
 - ≤ 1.8m
 - > 1.8m
- Peak Conduit Depth Ratio (d/D)**
 - < 0.5
 - 0.5 - 0.7
 - > 0.7

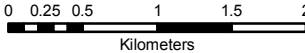
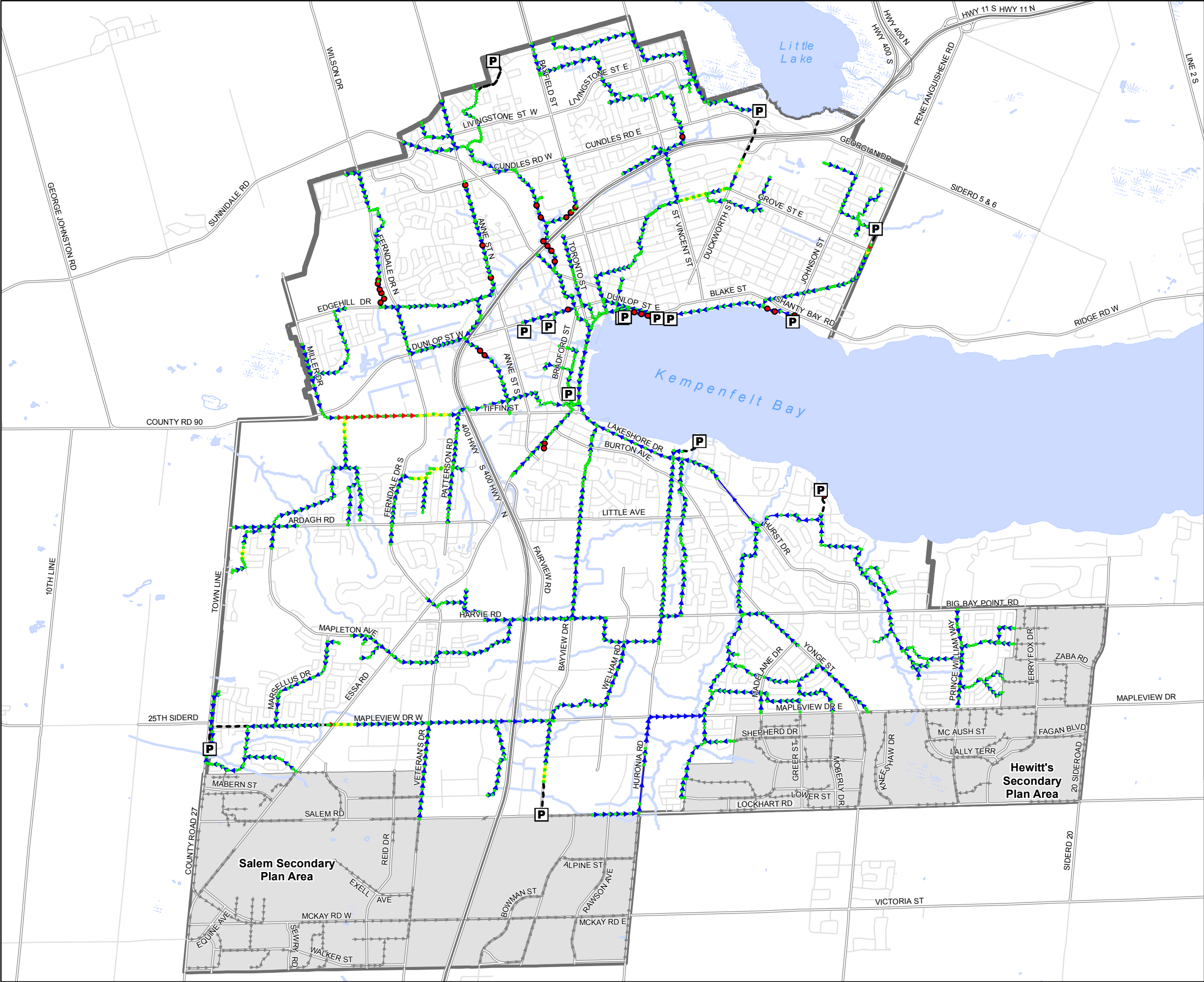


Figure 6-3
City of Barrie
Existing Wastewater
Collection System Performance
Wet Weather Flow Conditions
(2 Year Storm Event)



- P** Pumping Stations
- Secondary Plan Boundaries
- Forcemain
- Future Annexed Lands Sewers
- City of Barrie Limits**
- Freeboard (m)**
 - $\leq 1.8\text{m}$
 - $> 1.8\text{m}$
- Peak Conduit Depth Ratio (d/D)**
 - < 0.5
 - $0.5 - 0.7$
 - > 0.7

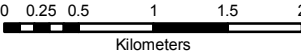
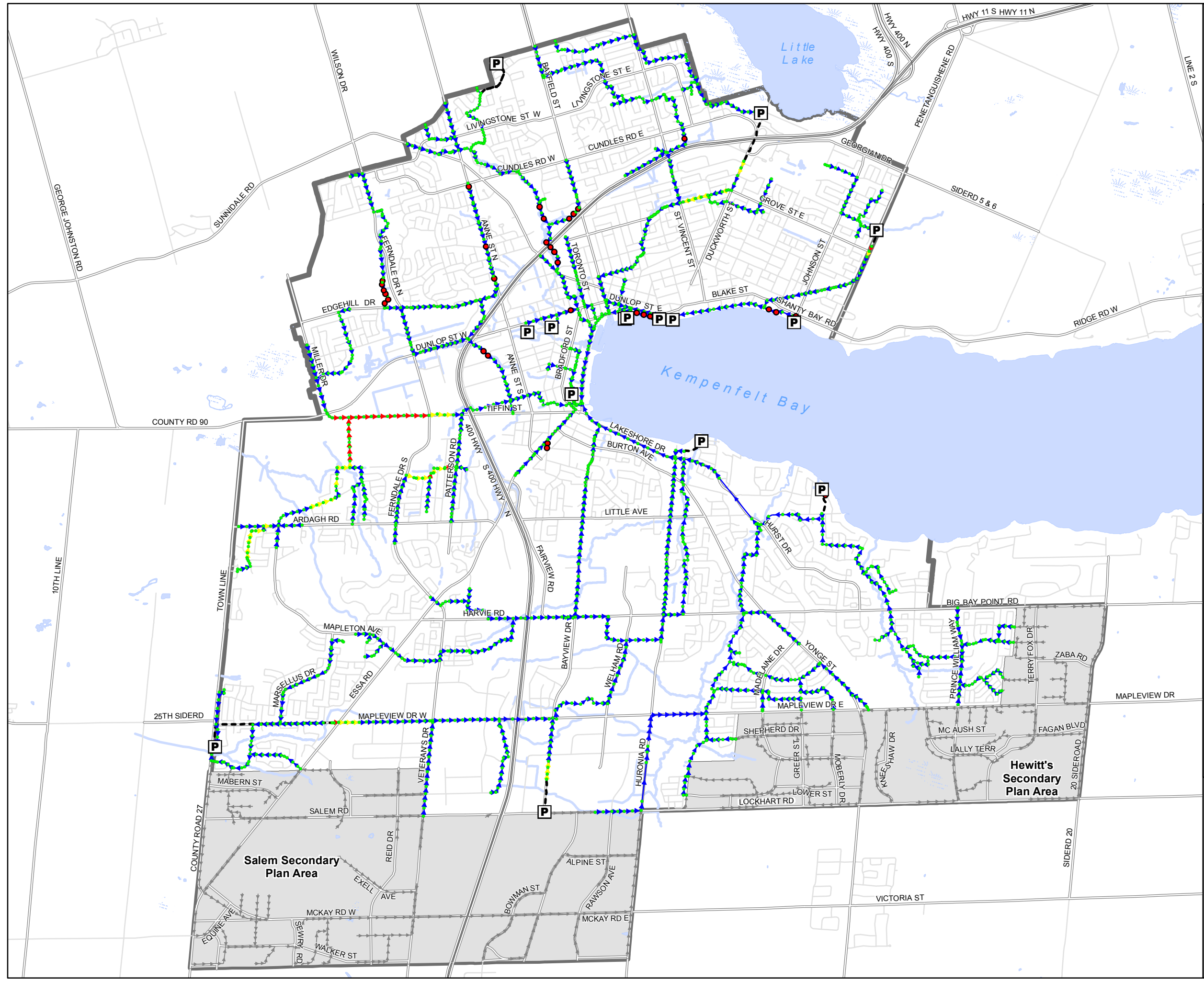


Figure 6-4
City of Barrie
Existing Wastewater
Collection System Performance
Wet Weather Flow Conditions
(5 Year Storm Event)



- P** Pumping Stations
- Secondary Plan Boundaries
- City of Barrie Limits
- Sanitary Collection System**
- Forcemain
- Future Annexed Lands Sewers
- Freeboard (m)**
- $\leq 1.8\text{m}$
- $> 1.8\text{m}$
- Peak Conduit Depth Ratio (d/D)**
- < 0.5
- $0.5 - 0.7$
- > 0.7

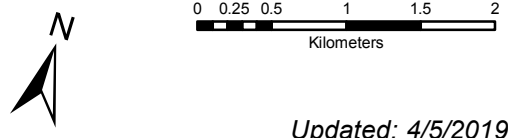


Figure 6-5
City of Barrie
Existing Wastewater
Collection System Performance
Wet Weather Flow Conditions
(10 Year Storm Event)

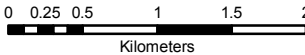
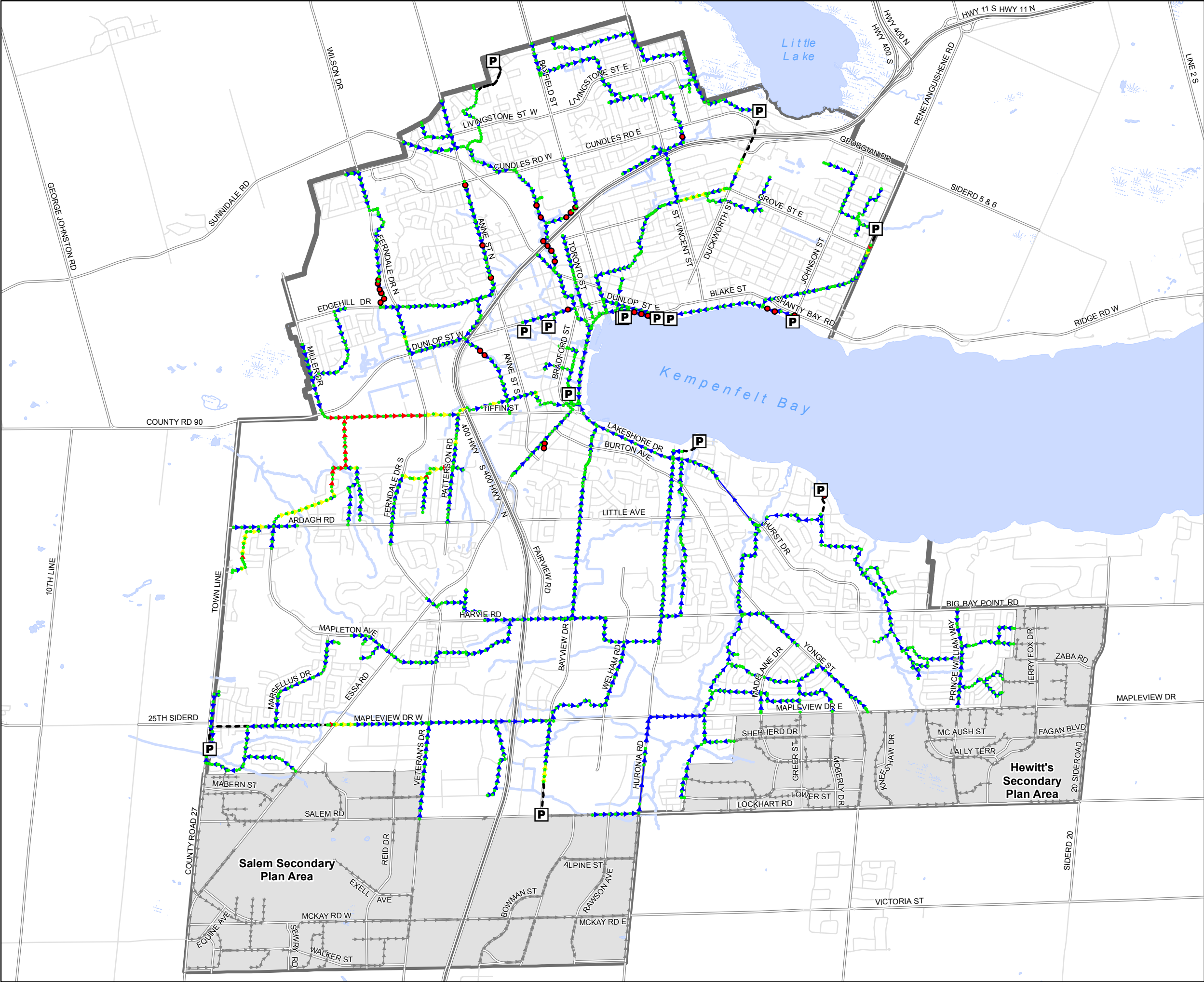
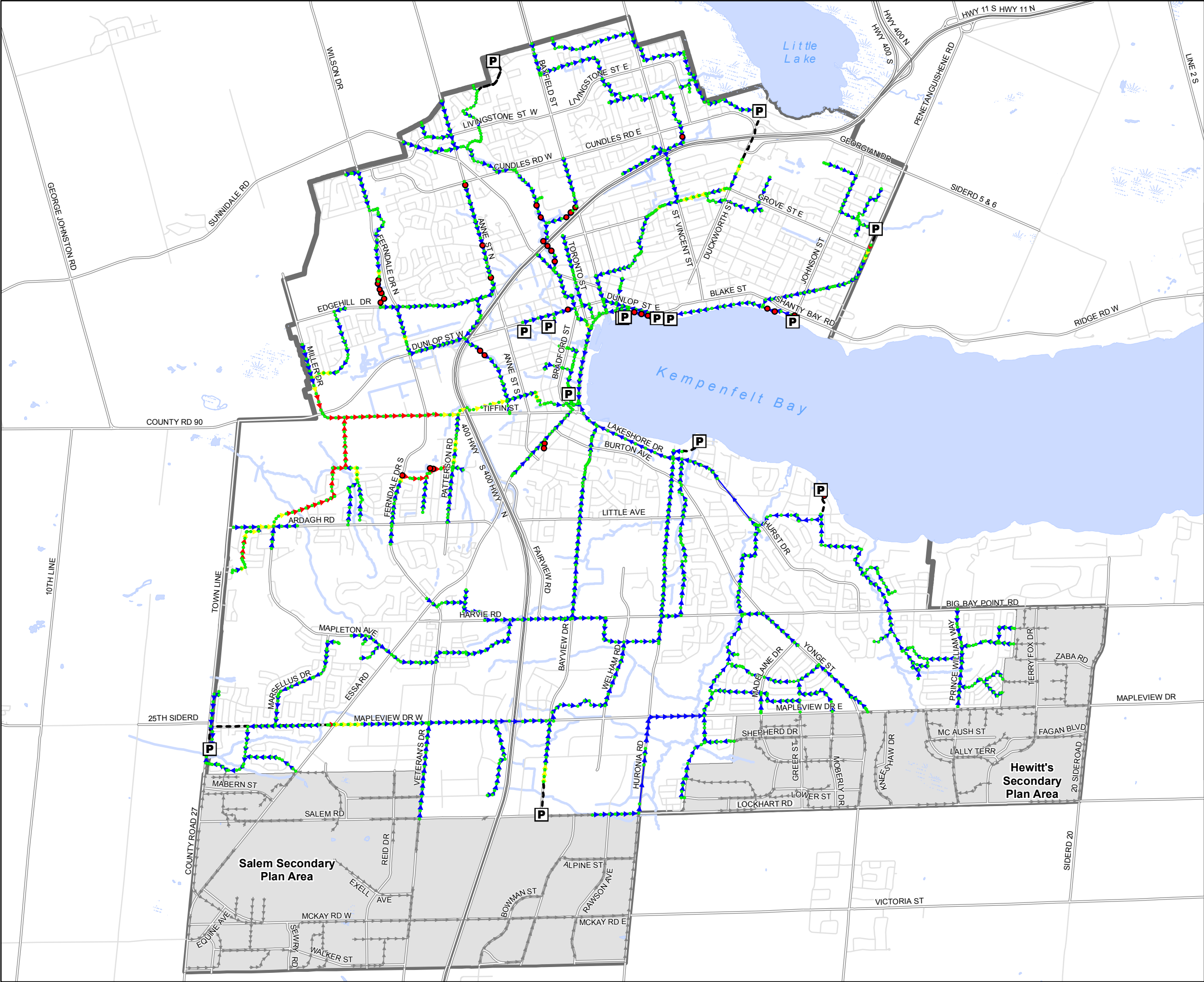
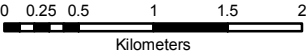


Figure 6-6
City of Barrie
Existing Wastewater
Collection System Performance
Wet Weather Flow Conditions
(25 Year Storm Event)



- P** Pumping Stations
- Forcemain
- Secondary Plan Boundaries
- City of Barrie Limits
- Future Annexed Lands Sewers
- NewFreebrd25yr**
 - $\leq 1.8\text{m}$
 - $> 1.8\text{m}$
- NewMaxFullDpth25yr**
 - < 0.5
 - $0.5 - 0.7$
 - > 0.7



**RESIDENTIAL TOWNHOUSE DEVELOPMENT
829 ESSA ROAD – CITY OF BARRIE
STORM WATER MANAGEMENT & FUNCTIONAL SERVICING REPORT**

APPENDIX C

Drawings



SITE LOCATION

829 ESSA ROAD

RESIDENTIAL TOWNHOUSE DEVELOPMENT



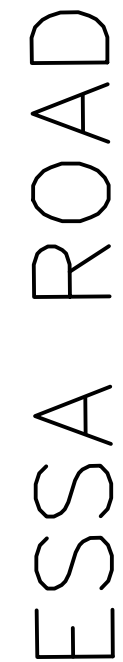
LIST OF DRAWINGS

RM-1
ESC-1
SERV-1
GP-1
DET-1

EXISTING CONDITIONS AND REMOVALS PLAN
EROSION AND SEDIMENT CONTROL PLAN
SITE SERVICING PLAN
SITE GRADING PLAN
STANDARD NOTES AND DETAILS PLAN

PROJECT NO.18-11409B

FOR APPROVAL
DECEMBER 2020



HORIZONTAL CONTROL MONUMENTS
U.T.M. (ZONE 17) NAD 83

MONUMENT # 03120040050
THE HORIZONTAL MONUMENT IS LOCATED AT THE CURB LINE ON THE NORTH SIDE OF ESSA ROAD, APPROXIMATELY 0.3km NORTH OF FERNDALE DRIVE SOUTH ON THE WEST SIDE OF ENTRANCE TO #379 ESSA ROAD.

N = N4911475.809 E = 603404.438

MONUMENT # 01019860430
THE HORIZONTAL MONUMENT IS LOCATED ON THE EAST SIDE OF ESSA ROAD, APPROXIMATELY 15m NORTH OF BEACON ROAD.

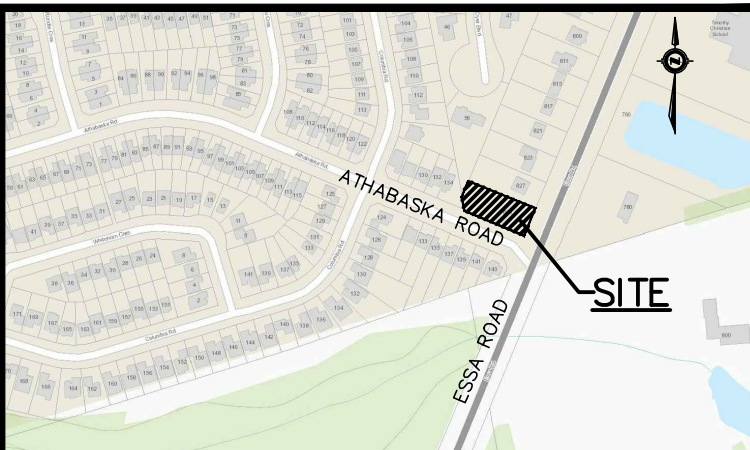
N = 4911746.645 E = 603629.996

VERTICAL CONTROL MONUMENTS

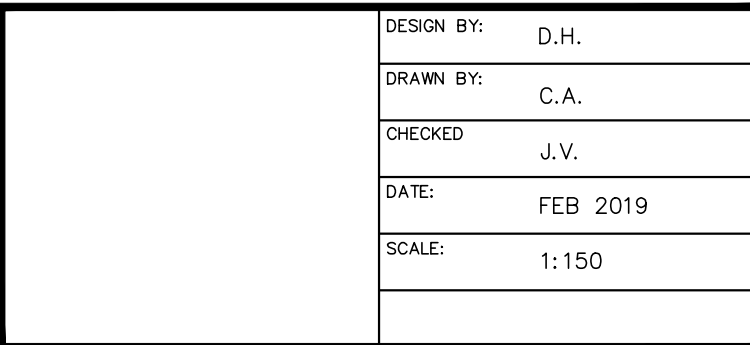
BENCHMARK:

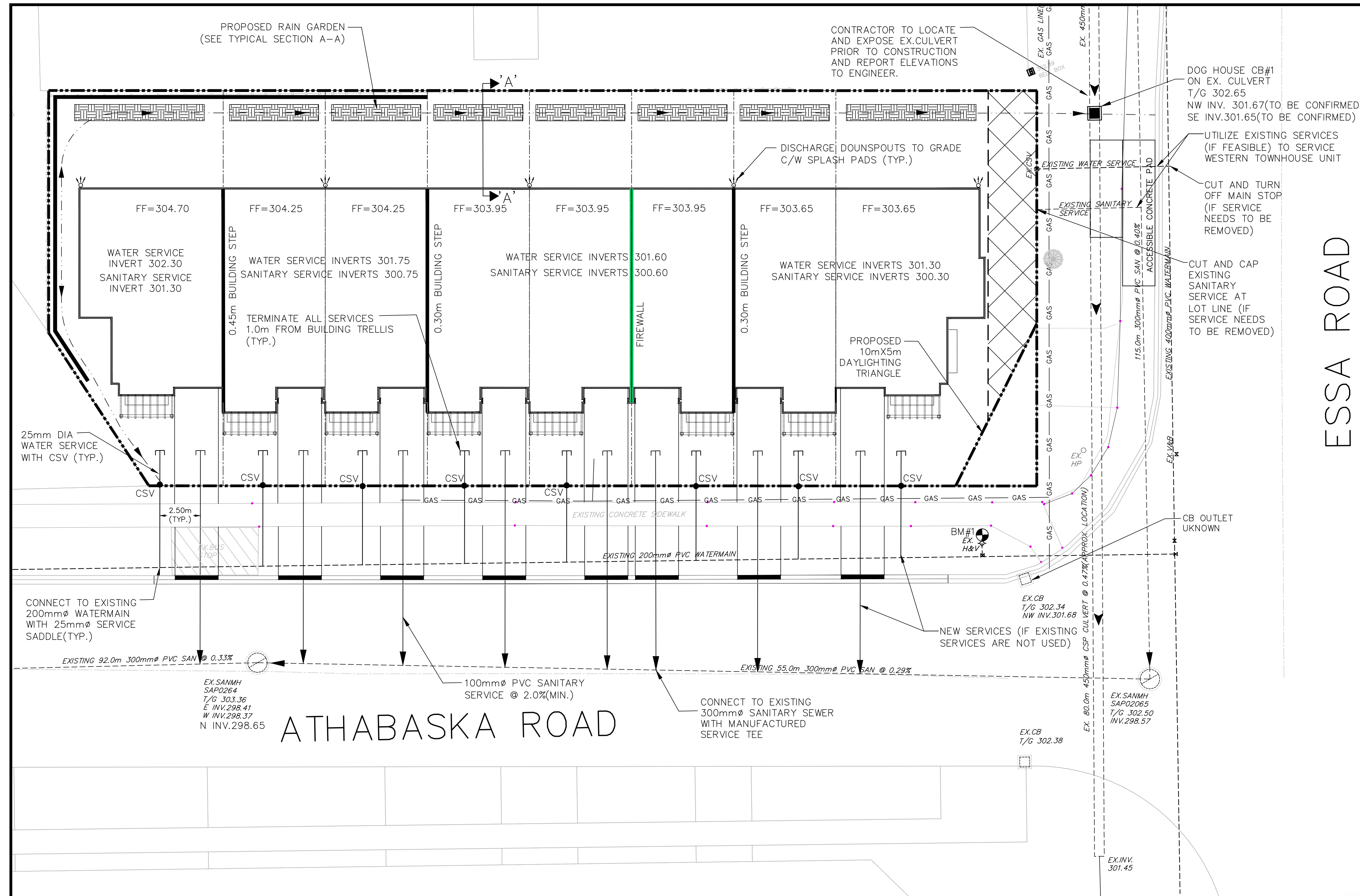
MONUMENT # 03120030023
ELEVATION: 303.862
WATER RESERVOIR ON HARVIE RD 0.5km EAST OF VETERANS DRIVE
THE VERTICAL CONTROL MONUMENT IS SET FLUSH IN THE TOP OF THE EAST WEST WALL TO THE WATER RESERVOIR. THE TABLET IS CENTERED IN THE WALL AND 90 CM BELOW THE TOP OF THE WALL.

MONUMENT # 03120030016
ELEVATION: 314.967
HOLLY MEADOWS ELEMENTARY SCHOOL - 151 MAPLETON AVE
THE VERTICAL CONTROL MONUMENT IS SET FLUSH IN THE NORTH WALL OF THE SOUTHERLY WALL TO SCHOOL. THE TABLET IS 80mm BELOW THE BRICK WORK AND 79cm WEST OF THE NORTH-EAST CORNER.

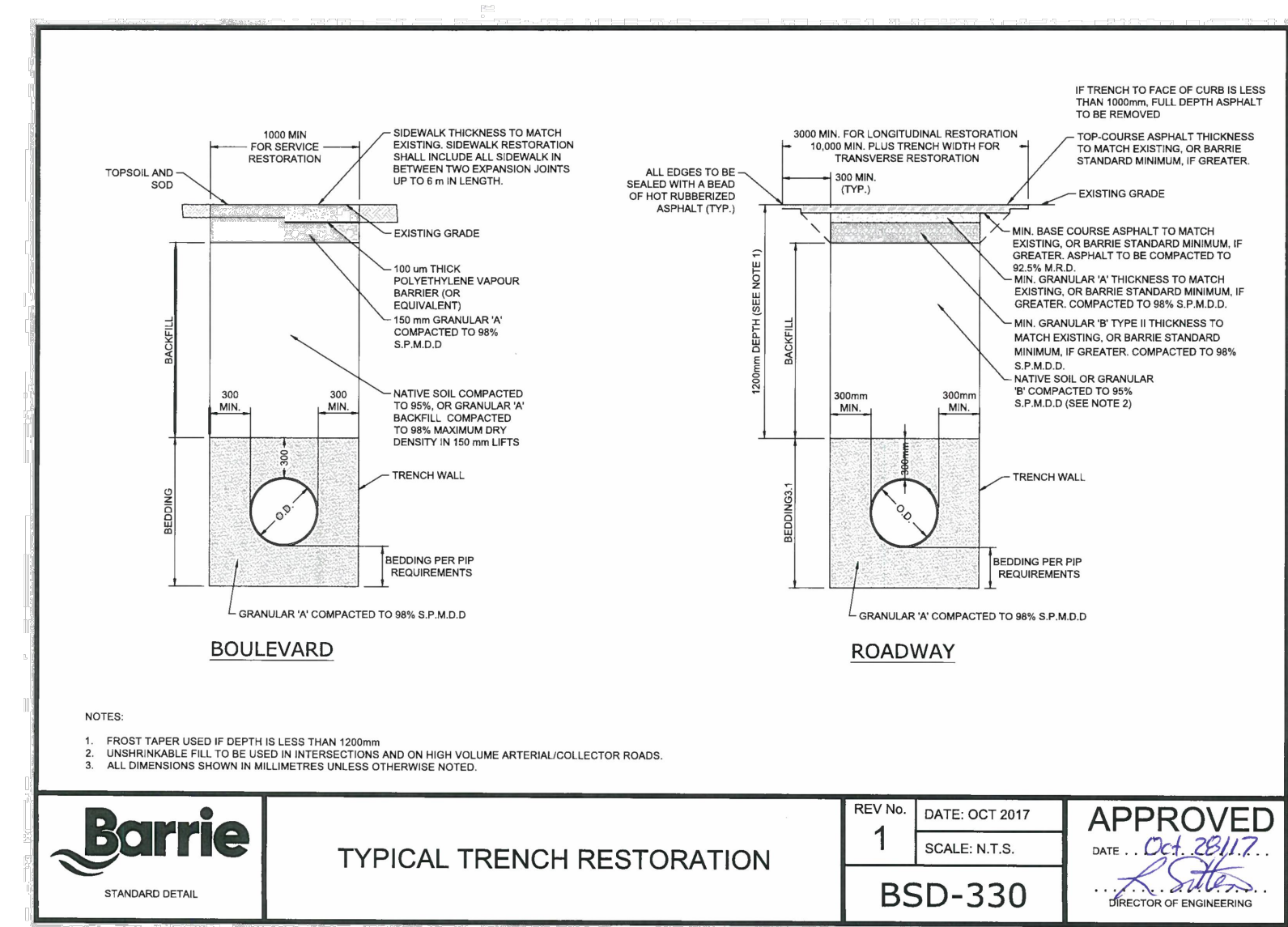
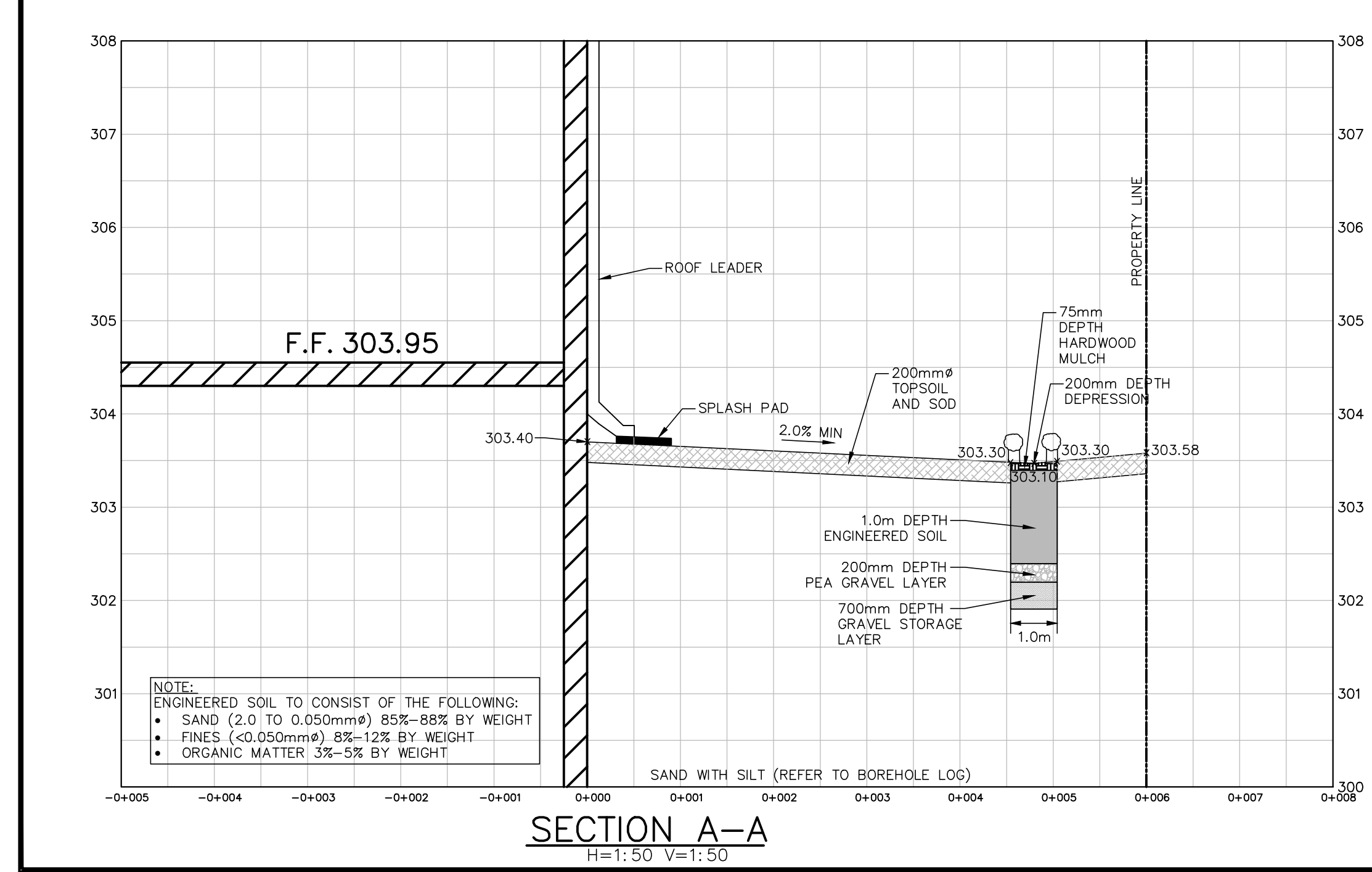


BENCHMARK			
BM #1			
TOP OPERATING NUT OF EXISTING FIRE HYDRANT #39			
ON NORTH SIDE OF ATHABASKA ROAD			
ELEVATION 303.36			
2	20.12.15	REVISED BUILDING LAYOUT	J.V.
1	20.07.30	SITE PLAN REVISION	C.A.
NO.	YY.MM.DD	REVISION	BY





ESSA ROAD



HORIZONTAL CONTROL MONUMENTS
U.T.M. (ZONE 17) NAD 83

MONUMENT # 03120040050
THE HORIZONTAL MONUMENT IS LOCATED AT THE CURB LINE ON THE NORTH SIDE OF ESSA ROAD, APPROXIMATELY 0.3km NORTH OF FERDALE DRIVE SOUTH ON THE WEST SIDE OF ENTRANCE TO #379 ESSA ROAD.
N = N4911475.809 E = 603404.438

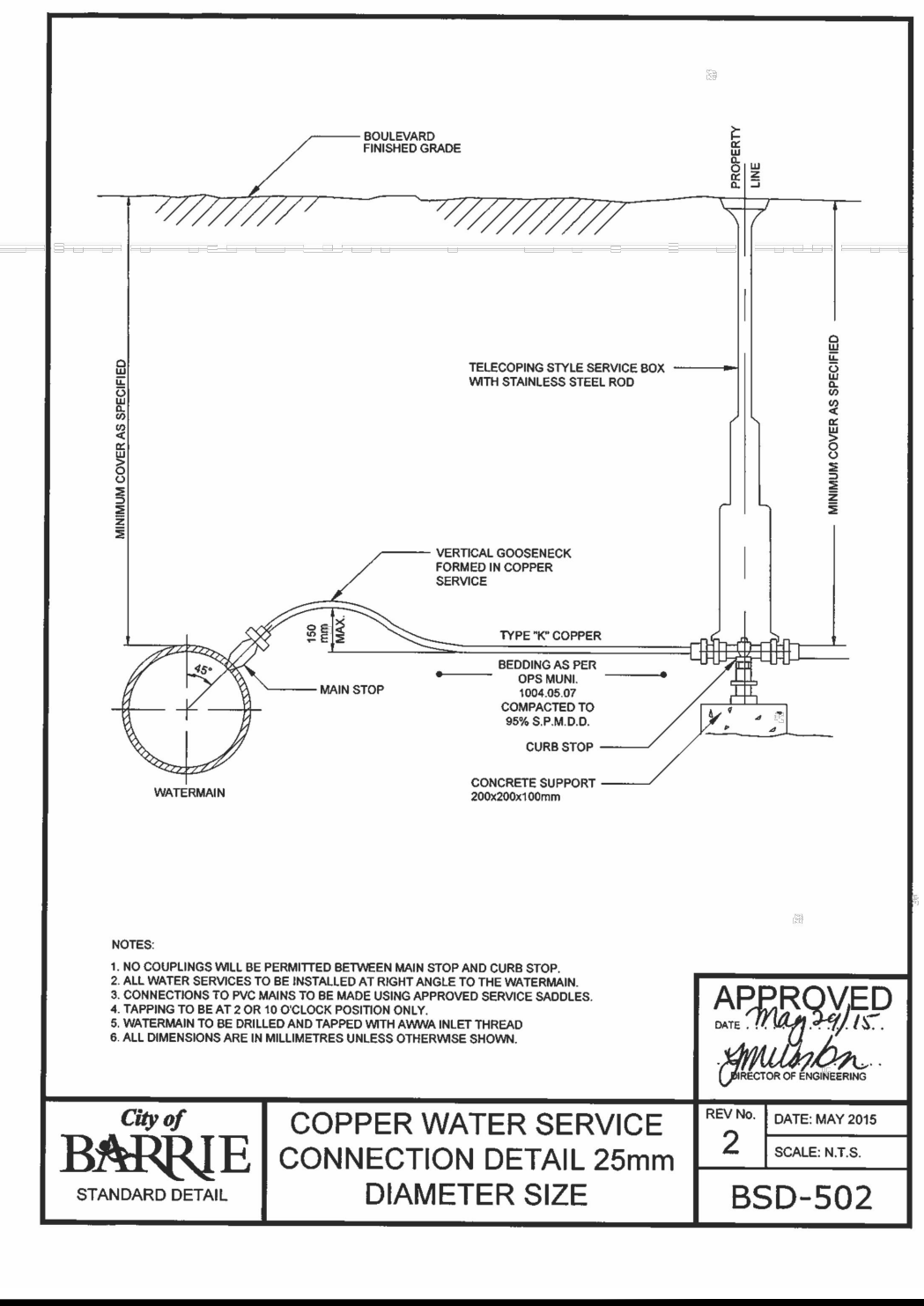
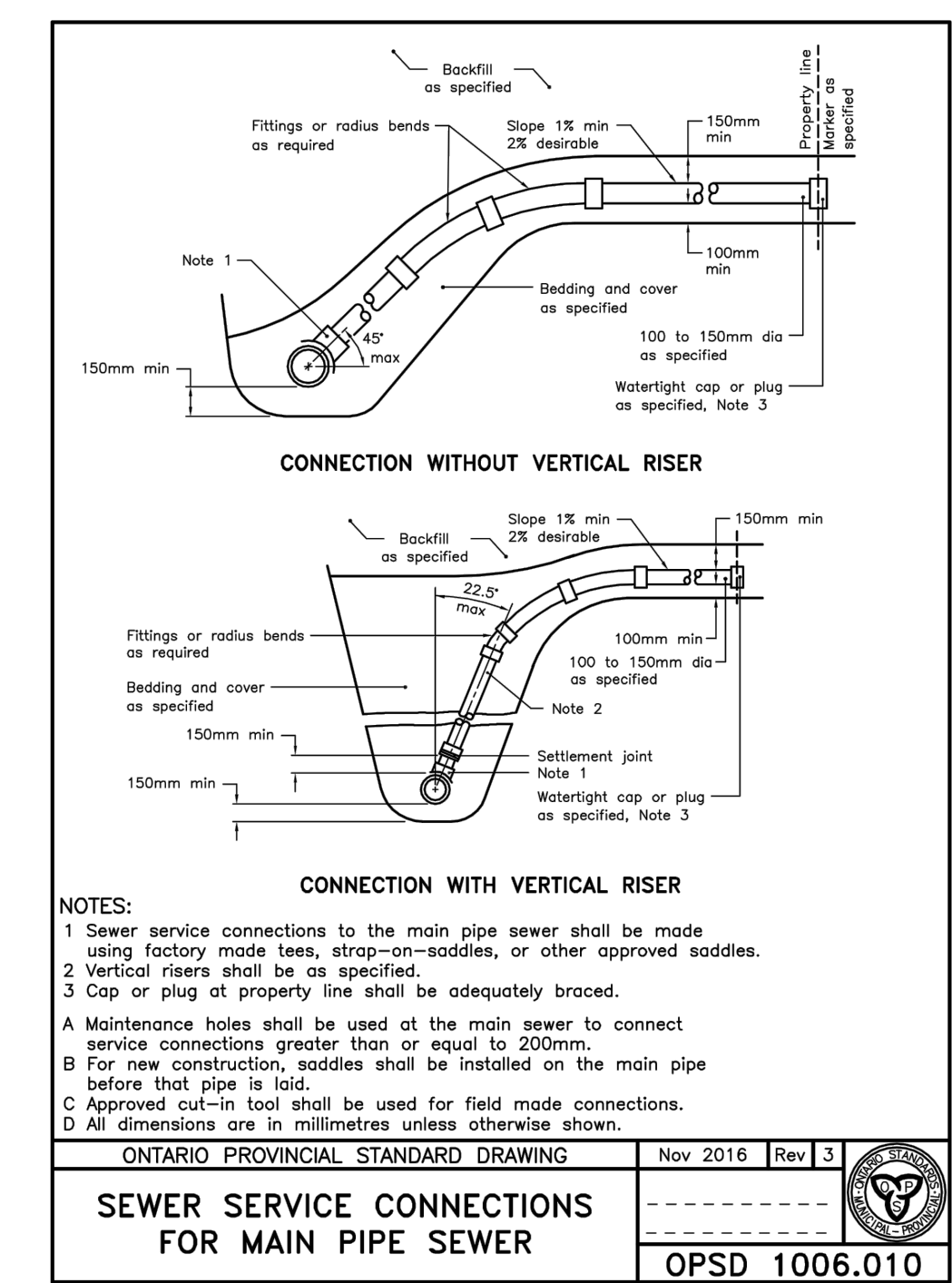
MONUMENT # 01019860430
THE HORIZONTAL MONUMENT IS LOCATED ON THE EAST SIDE OF ESSA ROAD, APPROXIMATELY 15m NORTH OF BEACON ROAD.
N = 4911746.645 E = 603629.996

VERTICAL CONTROL MONUMENTS

BENCHMARK:
MONUMENT # 03120030023
ELEVATION: 303.862
WATER RESERVOIR ON HARVE RD 0.5km EAST OF VETERANS DRIVE
THE VERTICAL CONTROL MONUMENT IS SET FLUSH IN THE SOUTH END OF THE EAST WING WALL TO THE WATER RESERVOIR. THE TABLET IS CENTERED IN THE WALL AND 90 CM BELOW THE TOP OF THE WALL.

MONUMENT # 03120030016
ELEVATION: 314.967
HOLLY MEADOWS ELEMENTARY SCHOOL - 151 MAPLETON AVE THE VERTICAL CONTROL MONUMENT IS SET FLUSH IN THE CORNER OF THE MOST NORTHERLY WALL TO SCHOOL. THE TABLET IS 80mm BELOW THE BRICK WORK AND 79cm WEST OF THE NORTH-EAST CORNER.

BENCHMARK
BM#1
TOP OPERATING NUT OF EXISTING FIRE HYDRANT #39 ON NORTH SIDE OF ATHABASKA ROAD
ELEVATION 303.36



PEL
PINESTONE ENGINEERING LIMITED | www.pel.ca

The position of existing above ground and underground utilities and facilities are not necessarily shown on the drawings, and where shown, the accuracy of the position of such utilities and facilities is not guaranteed. Before starting work, the contractor shall confirm the exact location of all existing utilities and facilities, and shall assume all liability for damage to them.

Drawings shall not be used for construction unless sealed and signed. All work to be performed in accordance with the Occupational Health & Safety Act 1990.

Any errors and/or omissions shall be reported to Pinestone Engineering Ltd. without delay.

KEY MAP

NOTES

- TOPOGRAPHIC SURVEY AND LEGAL INFORMATION COMPLETED BY JOE TOPO DECEMBER 18 2018.
- SITE PLAN PREPARED BY INNOVATIVE PLANNING SOLUTIONS.
- CURB STOP VALVES ARE NOT PERMITTED IN DRIVEWAYS.

LEGEND

- EX. HP/LS
- EX. CATCHBASIN
- EX. HYDRANT
- EX. VALVE AND BOX
- EX. SANITARY MANHOLE
- EX. STORM MANHOLE
- EX. SIGN
- CONTOUR
- OVERHEAD HYDRO WIRE
- DITCH CENTERLINE
- PROPOSED BIO-RETENTION/RAIN GARDEN
- PROPOSED 25mm WATER SERVICE WITH CURB STOP VALVE
- PROPOSED 100mm PVC SANITARY SERVICE

NO.	YY.MM.DD	REVISION	BY
2	20.12.15	REVISED BUILDING LAYOUT	J.V.
1	20.07.30	SITE PLAN REVISION	C.A.

NORTH ARROW

DESIGN BY:	D.H.
DRAWN BY:	C.A.
CHECKED:	J.V.
DATE:	JAN 2019
SCALE:	1:150

CLIENT/PROJECT

829 ESSA ROAD
RESIDENTIAL DEVELOPMENT
CITY OF BARRIE

DRAWING TITLE

SITE SERVICING PLAN

PROJECT NO.	DRAWING NO.	REVISION
18-11409B	SERV-1	2

DWG NAME

ALL WORK SHALL BE IN ACCORDANCE WITH CURRENT CITY OF BARRIE STANDARD DRAWINGS (BSD) AND ONTARIO PROVINCIAL STANDARD DRAWINGS.

ORDER OF PRECEDENCE OF STANDARD DRAWINGS FIRSTLY CITY OF BARRIE STANDARD DRAWINGS (BSD) AND SECONDLY ONTARIO PROVINCIAL STANDARD DRAWINGS (OPSD)

LOCATION OF EXISTING SERVICES ARE NOT GUARANTEED. THE CONTRACTOR IS REQUIRED TO NOTIFY THE VARIOUS UTILITIES COMPANIES 48 HOURS PRIOR TO THE COMMENCEMENT OF ANY WORK.

A RIGHT OF WAY ACTIVITY PERMIT IS REQUIRED FROM THE CITY OF BARRIE OPERATIONS DEPARTMENT PRIOR TO THE COMMENCEMENT OF ANY WORKS ON THE SITE.

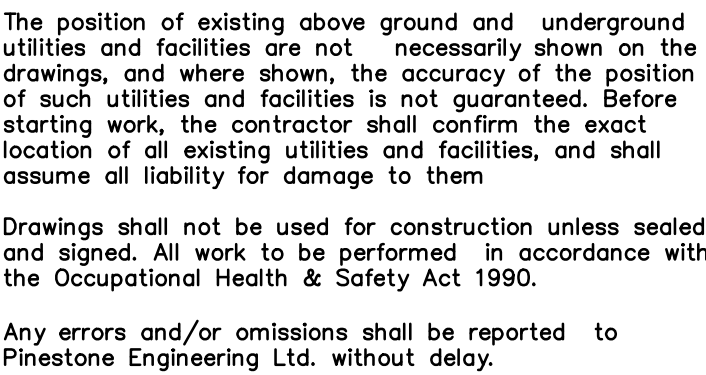
A SITE ALTERATION PERMIT IS REQUIRED FROM THE ENGINEERING DEPARTMENT PRIOR TO THE COMMENCEMENT OF ANY EARTHWORKS ON THE SITE. (IF ALTERATION IS GREATER THEN 0.5ho)

NATIVE MATERIAL SUITABLE FOR BACKFILL SHALL BE COMPACTED TO 95% STANDARD PROCTOR MAXIMUM DENSITY.

GRANULAR MATERIAL USED FOR BACKFILL, SHALL E PLACED IN LAYERS 150MM IN DEPTH MAXIMUM AND COMPACTED TO 100% STANDARD PROCTOR MAXIMUM DRY DENSITY.

ALL DISTURBED AREAS ARE TO BE REINSTATED TO THEIR ORIGINAL CONDITION OR BETTER, AS DETERMINED BY THE CITY ENGINEERING DEPARTMENT.

ALL SILT CONTROL AND EROSION PROTECTION DEVICES ARE TO BE IN PLACE PRIOR TO THE COMMENCEMENT OF CONSTRUCTION AND SHALL REMAIN IN PLACE AND BE MAINTAINED BY THE CONTRACTOR UNTIL CONSTRUCTION IS COMPLETE AND THE GRASS HAS ESTABLISHED GROWTH, SUBJECT TO APPROVAL BY THE CITY ENGINEERING DEPARTMENT.



1. TOPOGRAPHIC SURVEY AND LEGAL INFORMATION COMPLETED BY JOE TOPO DECEMBER 18 2018.
2. SITE PLAN PREPARED BY INNOVATIVE PLANNING SOLUTIONS.

2	20.12.15	REVISED BUILDING LAYOUT	J.V.
1	20.07.30	SITE PLAN REVISION	C.A.
NO.	YY.MM.DD	REVISION	BY

SEAL 	NORTH ARROW
---	--------------------

	DESIGN BY:	D.H.
	DRAWN BY:	C.A.
	CHECKED	J.V.
	DATE:	FEB 2019
	SCALE:	N.T.S.

CLIENT/PROJECT			
829 ESSA ROAD RESIDENTIAL DEVELOPMENT CITY OF BARRIE			
DRAWING TITLE			
STANDARD NOTES AND DETAILS PLAN			
PROJECT NO. 18-11409B	DRAWING NO. DET-1		REVISION 2